鋼筋混凝土深短連接梁之設計

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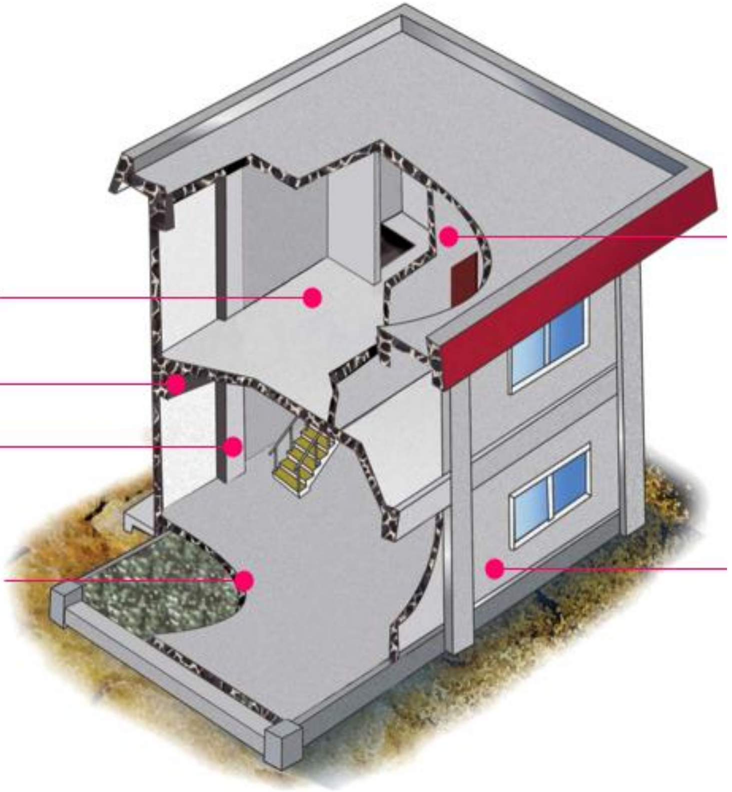
國立臺灣大學
National Taiwan University

鋼筋混凝土牆為台灣建築結構之重要構件



潮濕、多雨

↓

RC外牆

地小人稠、集合住宅

↓

RC內牆

通風、採光、通行

↓

開孔RC牆

建築功能需要，RC牆於建築圖中隨處可見

↓

RC牆勁度高、強度大，強而有力的結構桿件

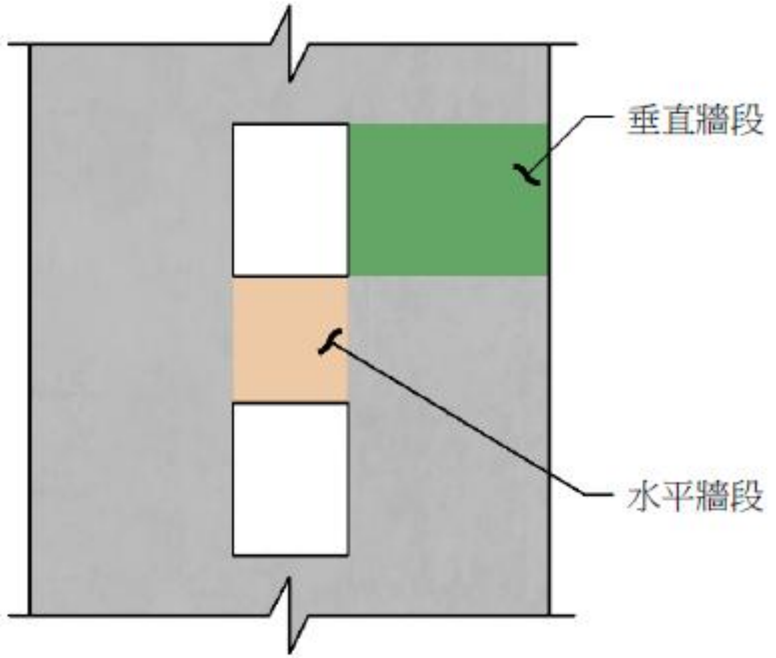
↓

結構功能遭忽略，RC牆於結構圖中從缺

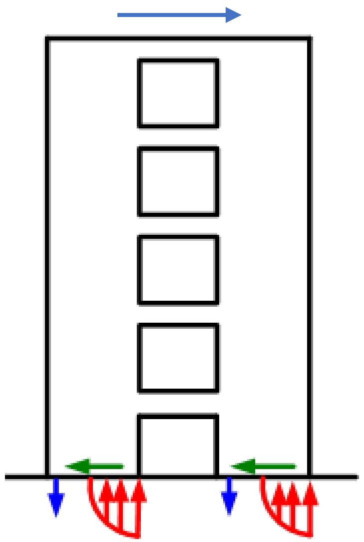
↓

確有不當，應予改善

本次講習會之宗旨

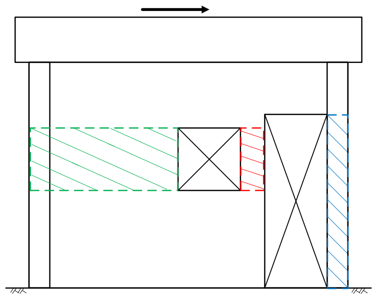


水平牆段為弱區



連接梁設計

垂直牆段為弱區



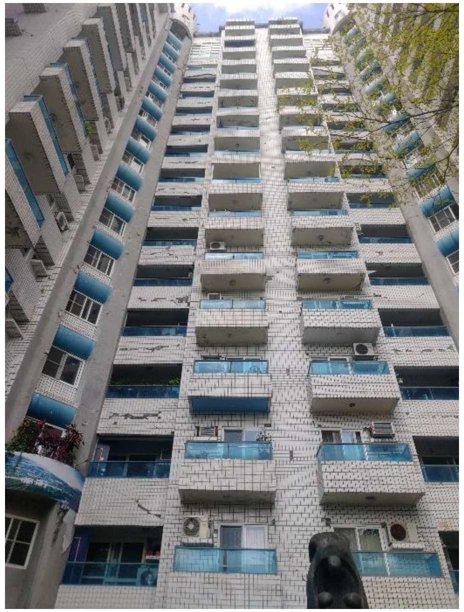
開孔牆設計

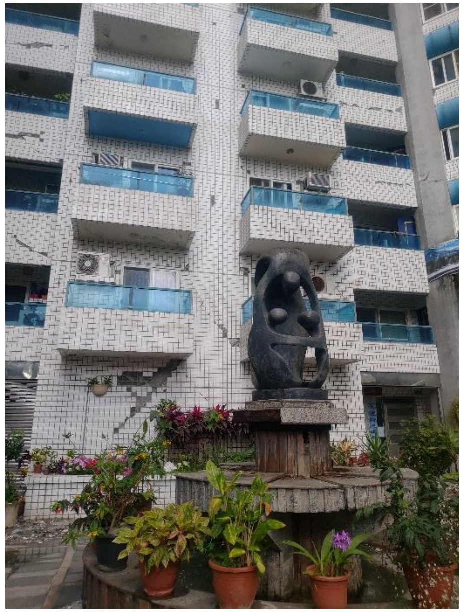
大綱

- 前言
- 對角鋼筋之功能
- 傳力機制與強度預測
- ACI 318規範設計之檢討
- 測試資料庫之探討
- 深短連接梁之設計建議
- 結論

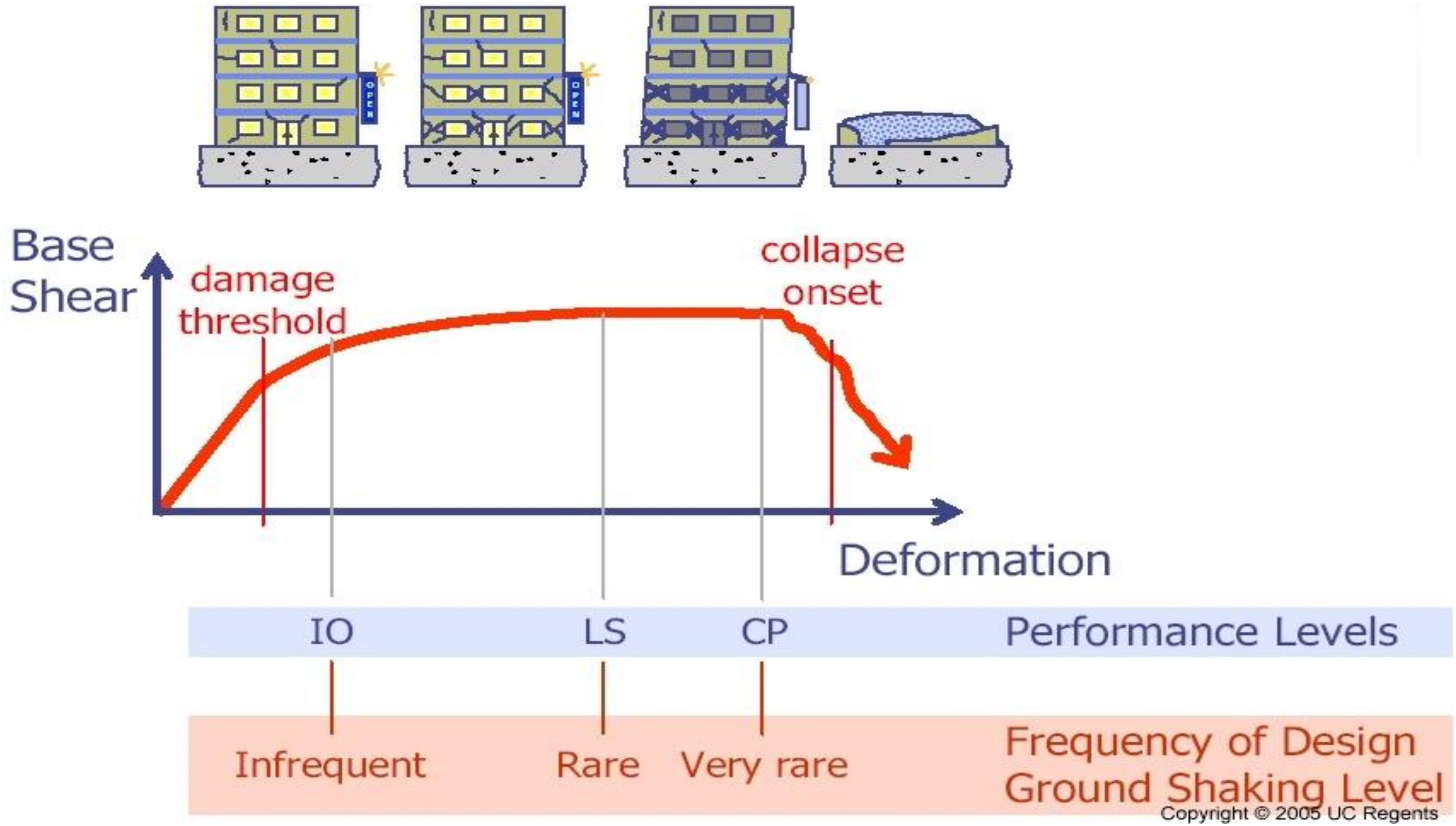
前言

非結構RC牆損壞 (2024-04-03花蓮大地震)





建築損傷控制 - 位移控制



Base Shear
damage threshold
collapse onset
Deformation
IO
LS
CP
Performance Levels
Infrequent
Rare
Very rare
Frequency of Design Ground Shaking Level
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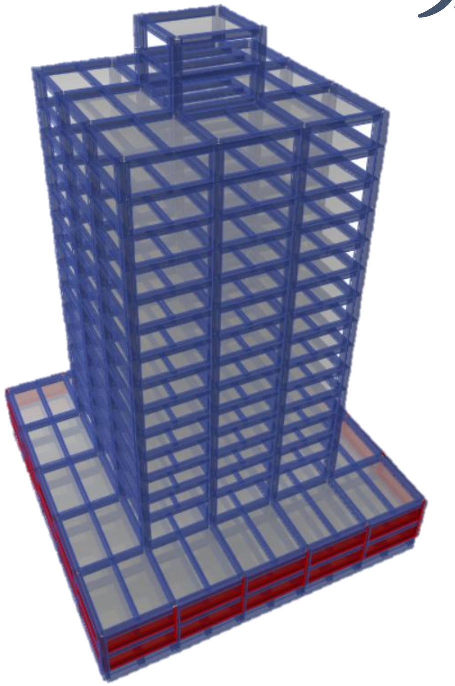
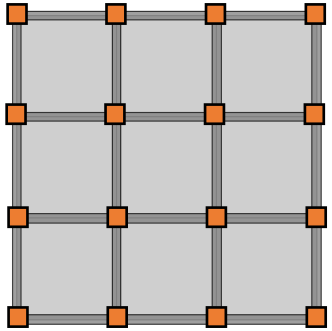
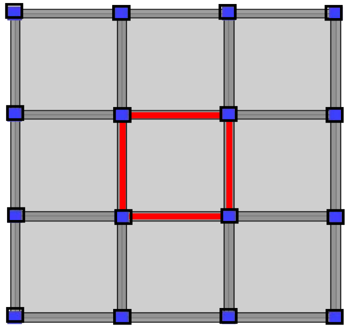
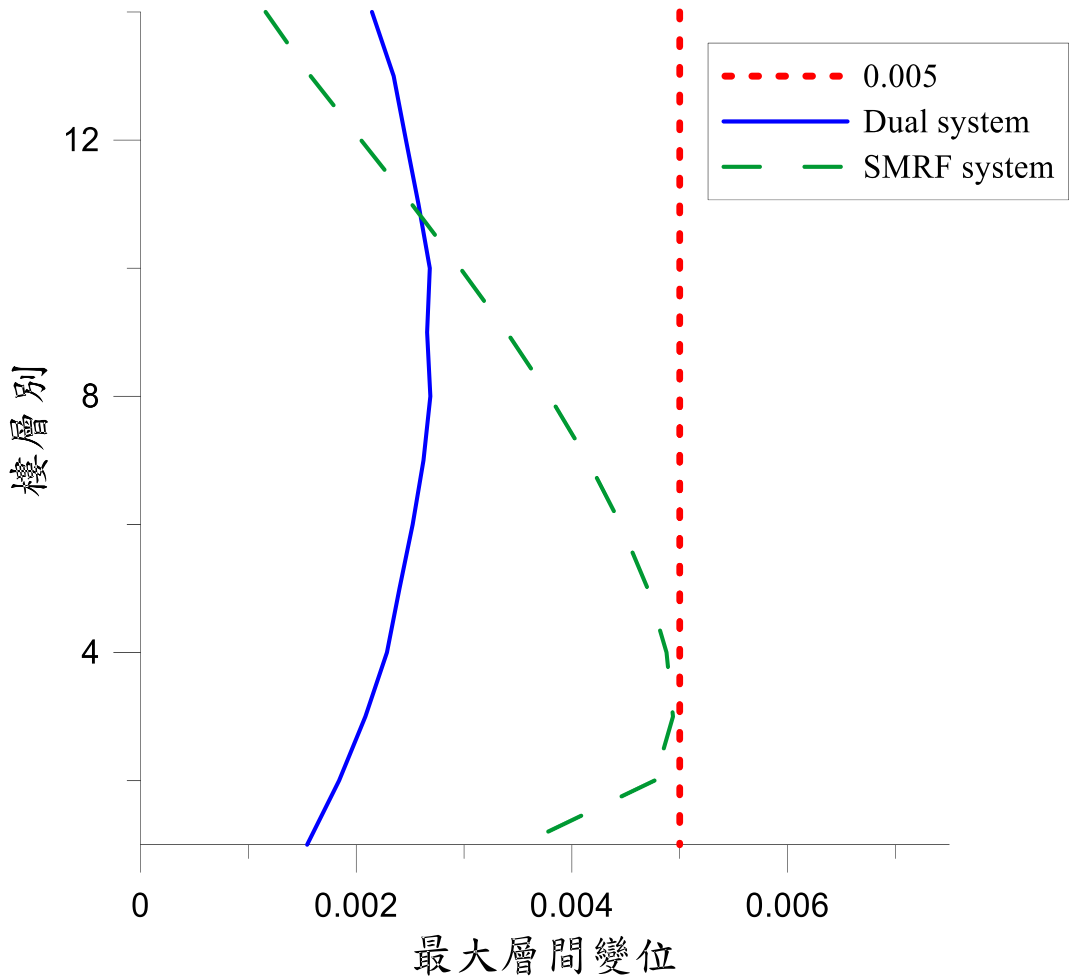
剪力牆系統 - 優良之側位移控制



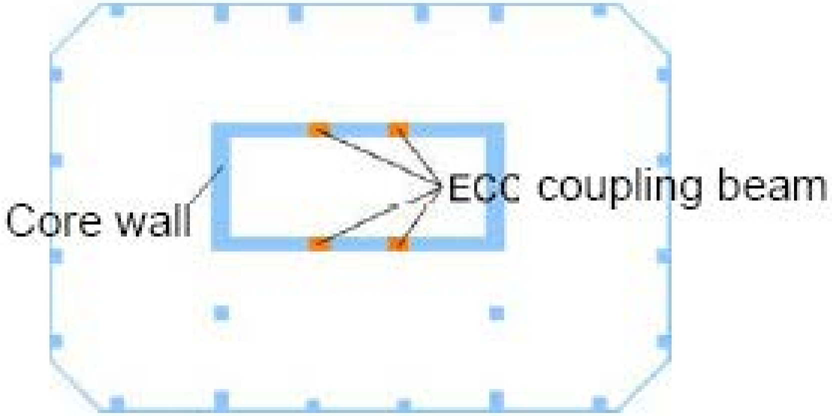


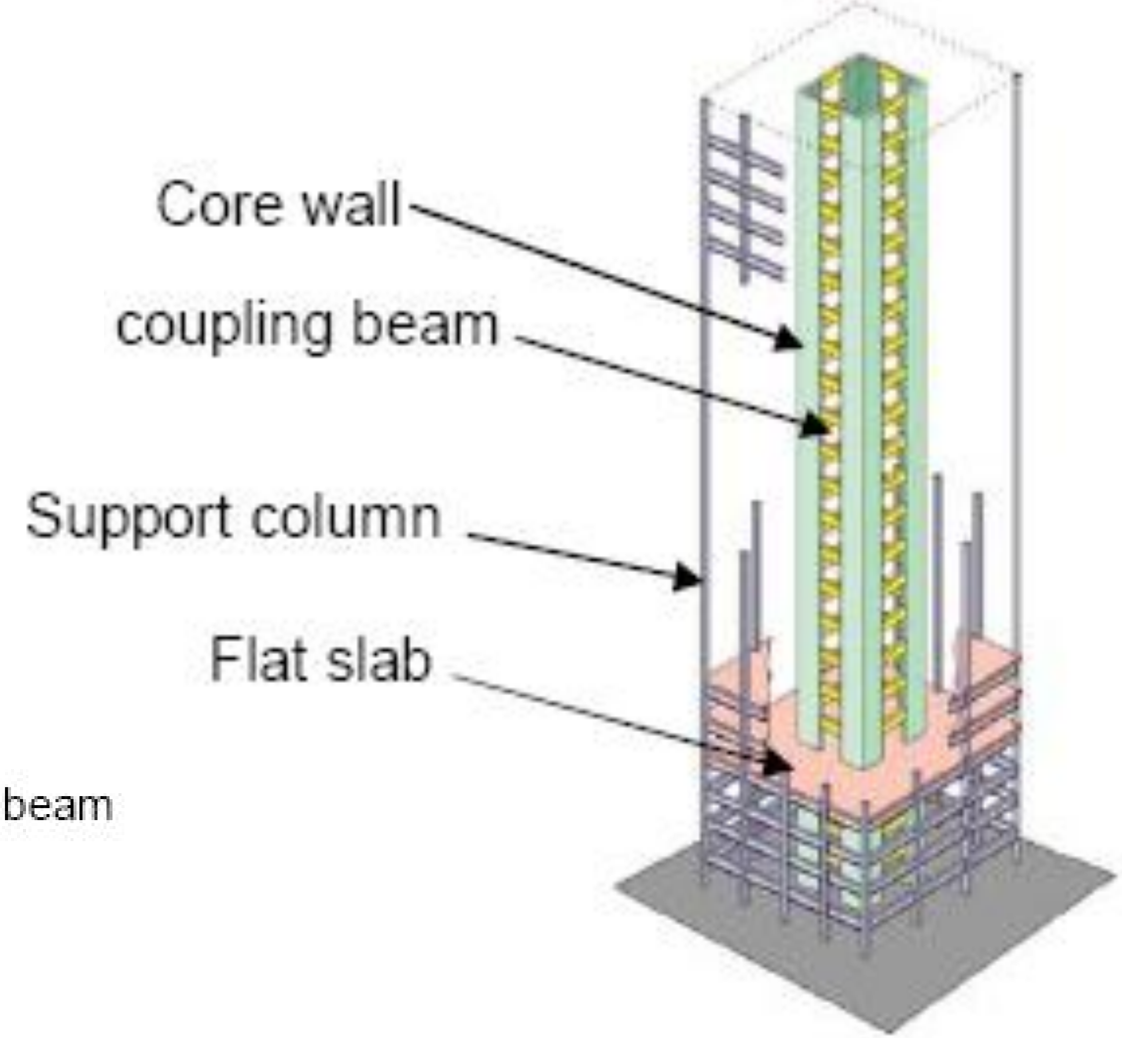




黃世建、黃紹愷、翁樸文、歐昱辰、黃明慧，(2023)「鋼筋混凝土二元系統剪力牆之剪力強度設計」，結構工程，第三十八卷，第四期，第80-105頁。

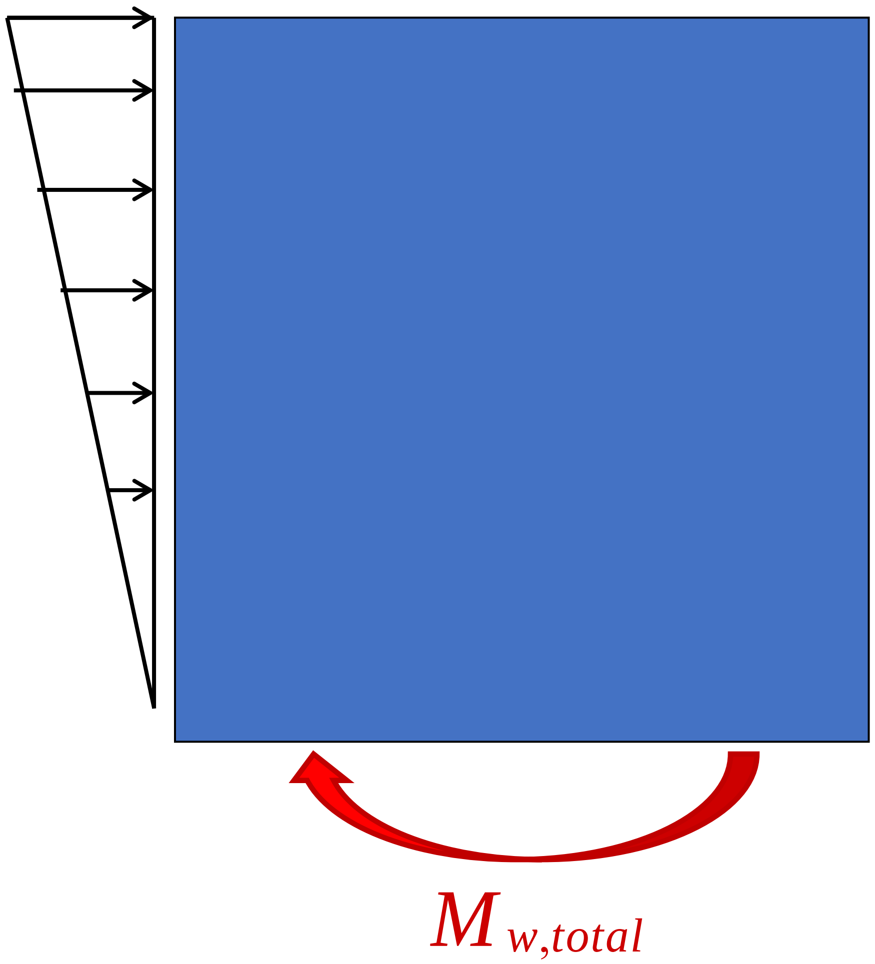
高樓服務核心之耦合牆系統

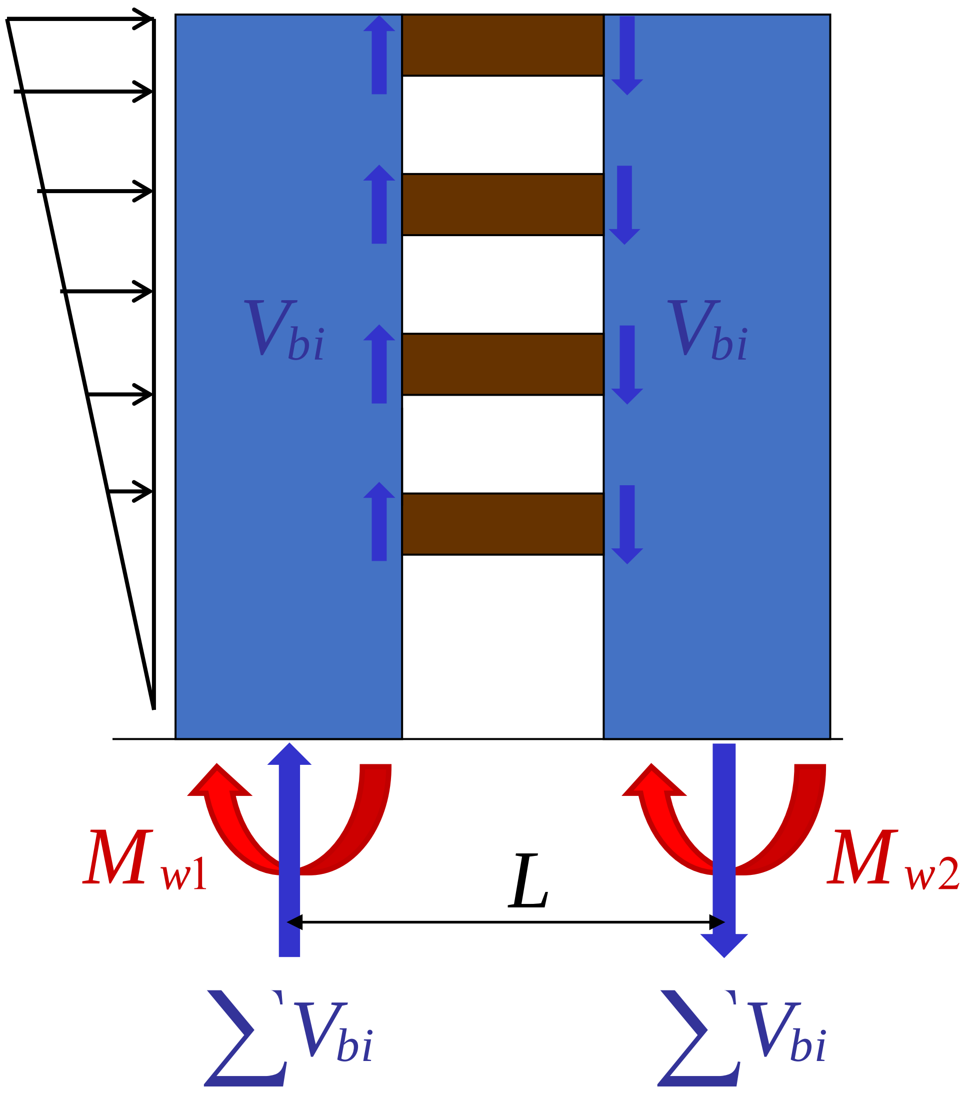




外顯式之耦合牆系統
– Kabuki Hotel @ Japanese Town, SF, California

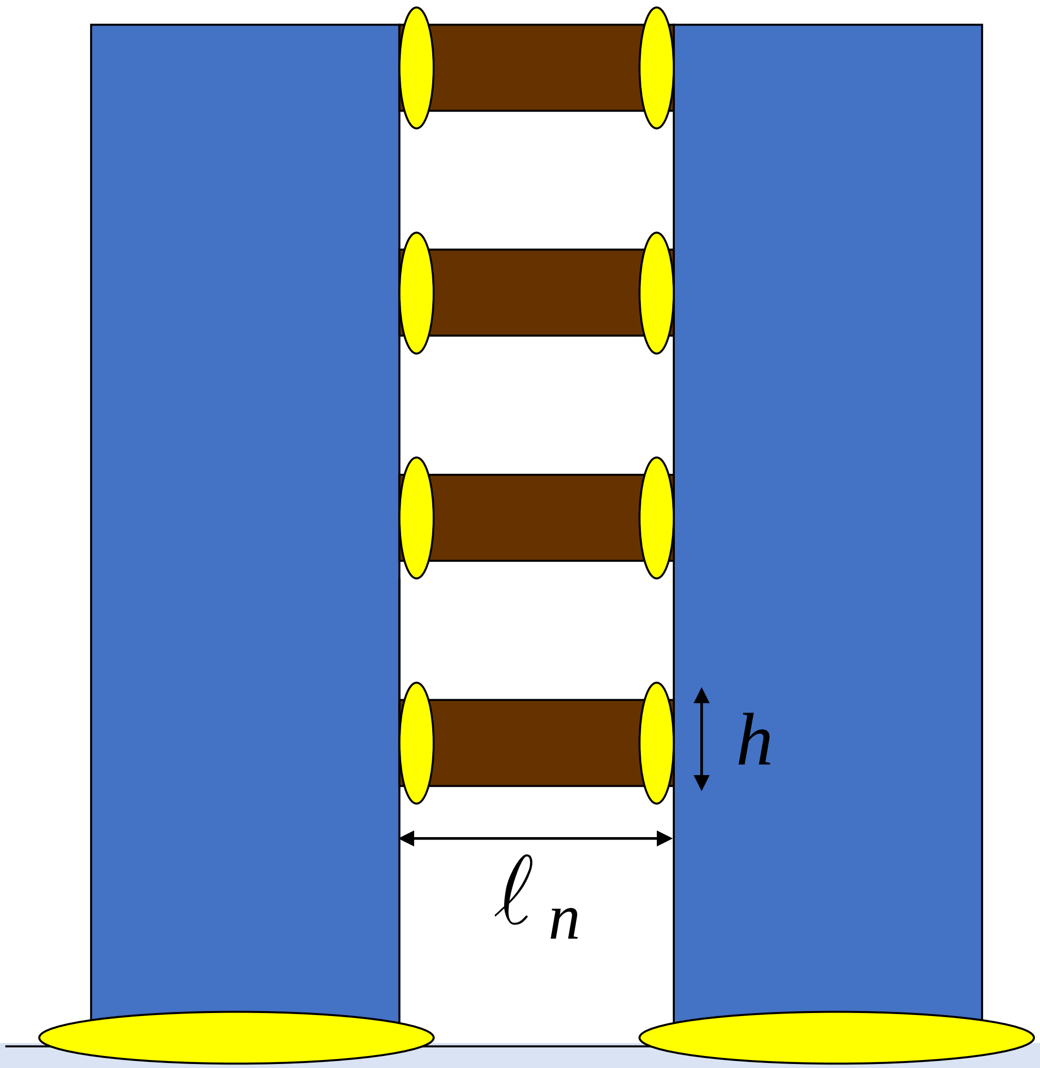
耦合牆作用–提高傾倒彎矩之阻抗





連接梁作用−提高耐震消能機制

較佳能量消散



較佳耐震性能

NTU CEER

國家實驗研究院
國家地震工程研究中心

鋼筋混凝土框架剪力牆與地梁之剪力強度設計講習會

時間		講題	主講人	主持人
114年7月25日(五)	13:30～13:50	報到		
	13:50～14:00	致詞	歐昱辰	黃尹男
	14:00～14:50	框架結構牆之剪力強度設計	黃世建	
	14:50～15:40	框架非結構牆之損傷控制		
	15:40～16:00	休息與茶點		
	16:00～16:50	基礎地梁之剪力強度設計	黃世建	林瑞良
	16:50～17:10	綜合座談		

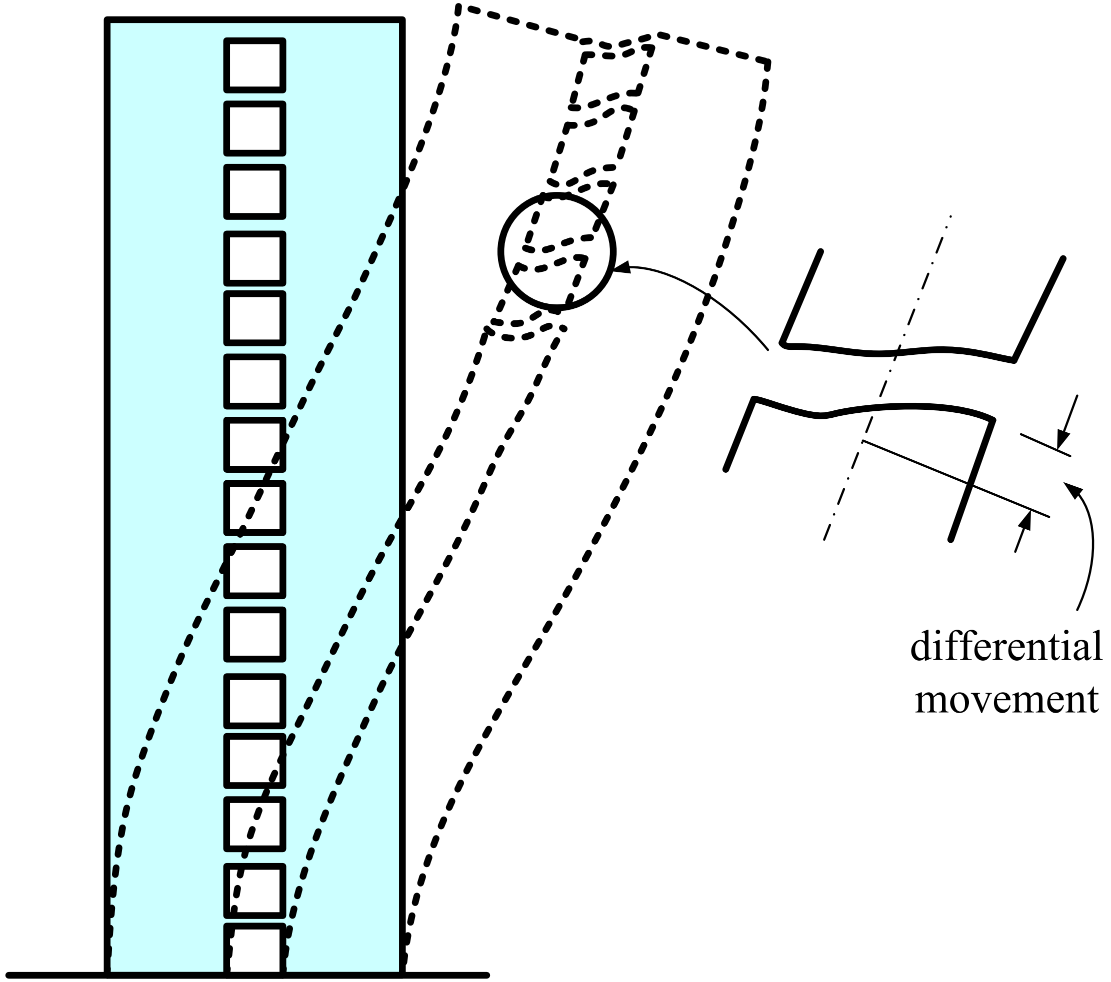
- 講義資料下載

https://conf.ncree.org.tw/download/0-A1140725-會議資料R1.zip

- 錄影檔觀看

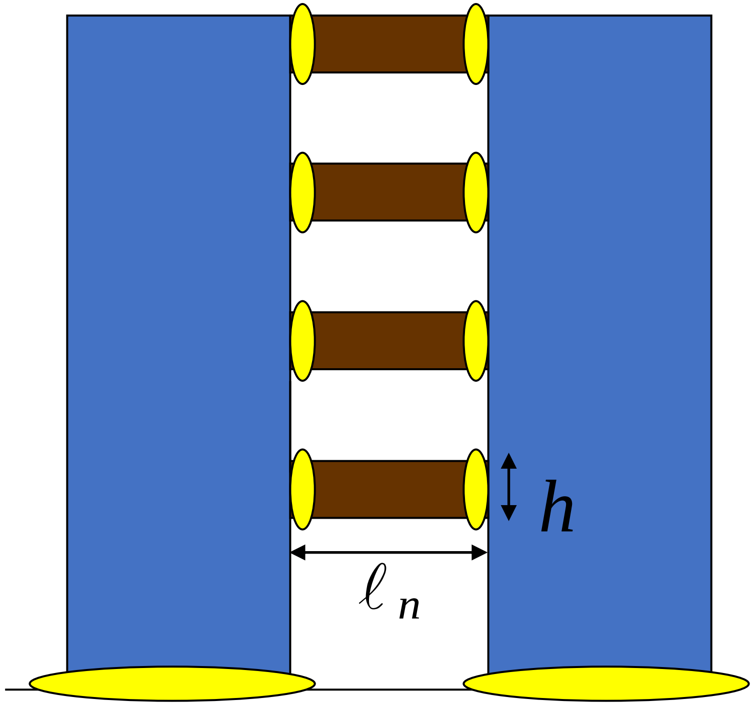
https://youtu.be/0bc1ixYgVsg

連接梁需求較高的塑鉸轉動能力



連接梁之耐震設計理念

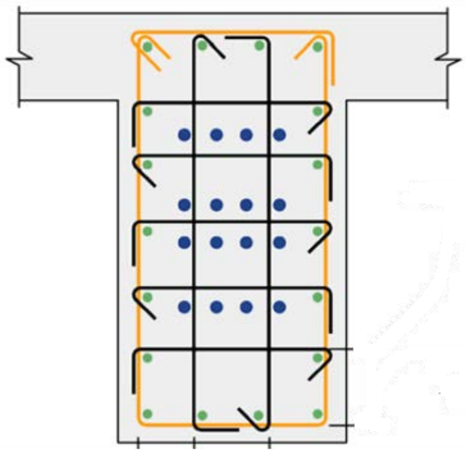
講習宗旨：
深短連接梁剪力設計
之探討



- 設計梁兩端產生撓曲塑鉸

$$\phi V_{n@DBE} \geq \frac{2M_n}{\ell_n}$$

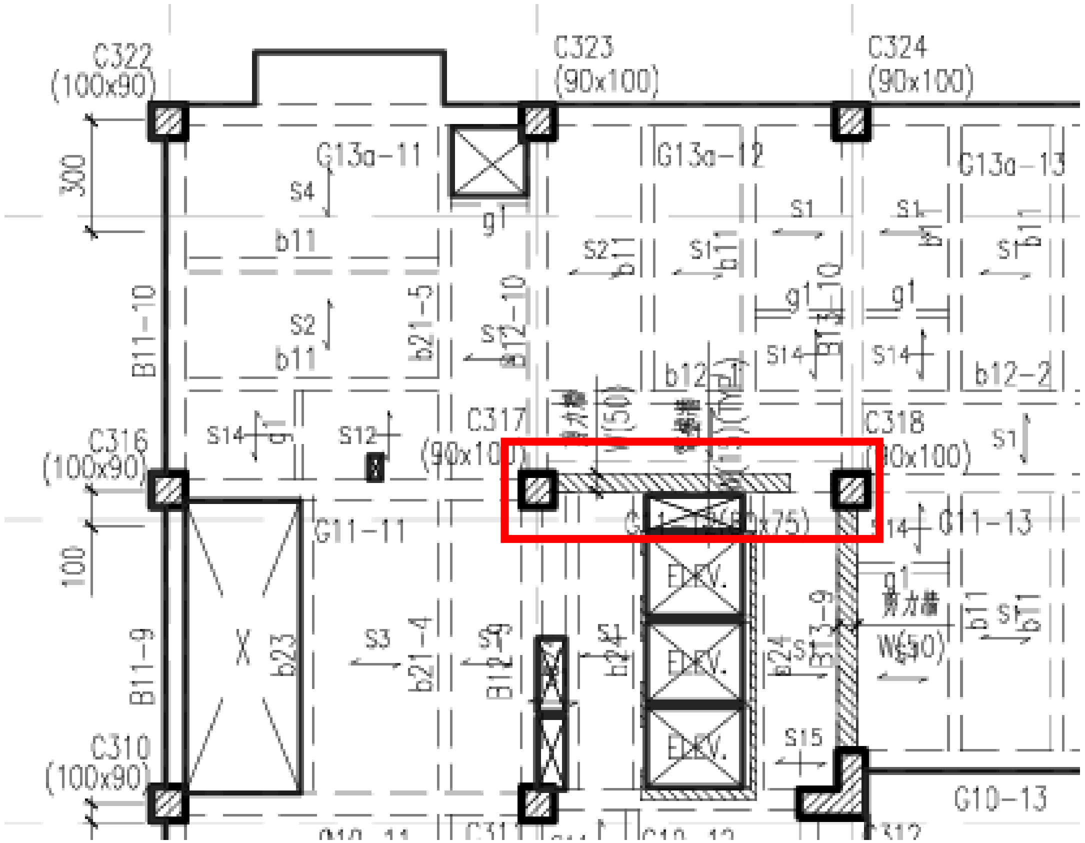
- 緊密箍筋配置維持塑鉸消能

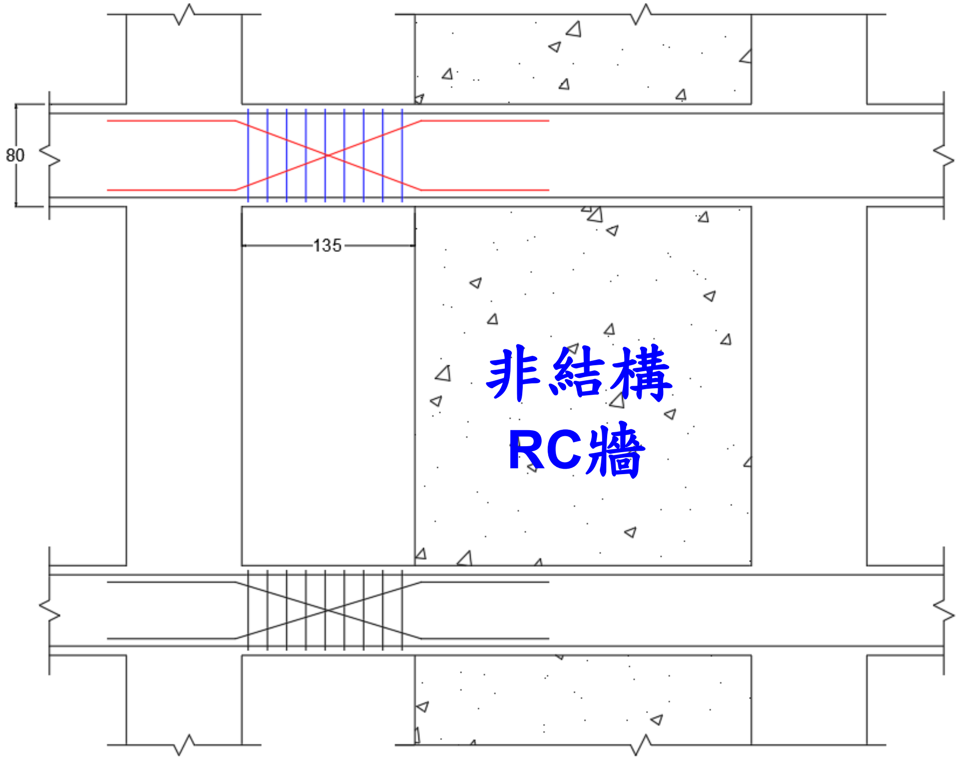


- 全面排除剪力破壞

$$\phi V_{n@MCE} \geq \frac{2M_{pr}}{\ell_n}$$

非結構RC牆所造成之深梁亦適用

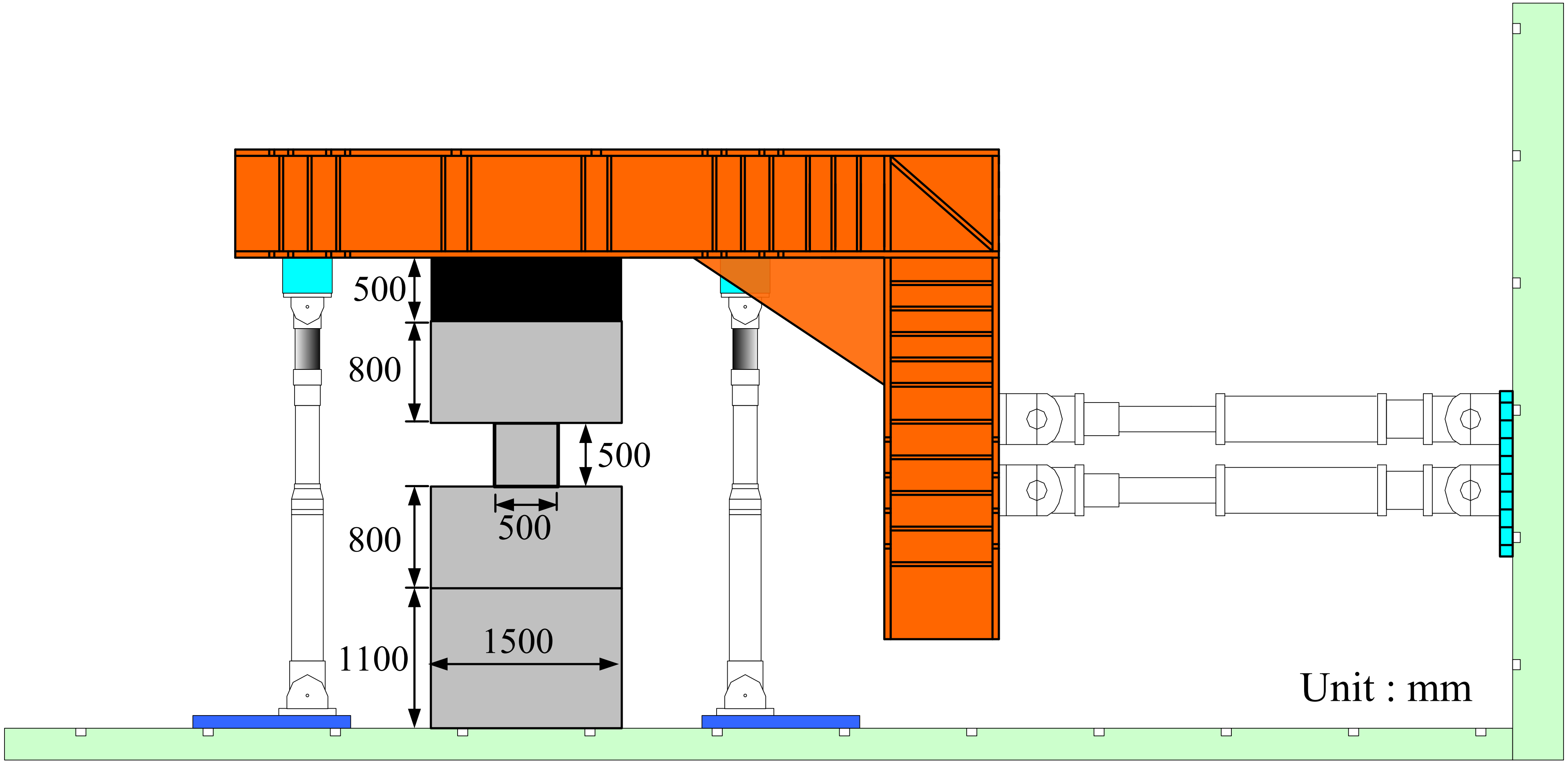




對角鋼筋之功能

測試布置

$$\frac{\ell_n}{h} = 1.0$$



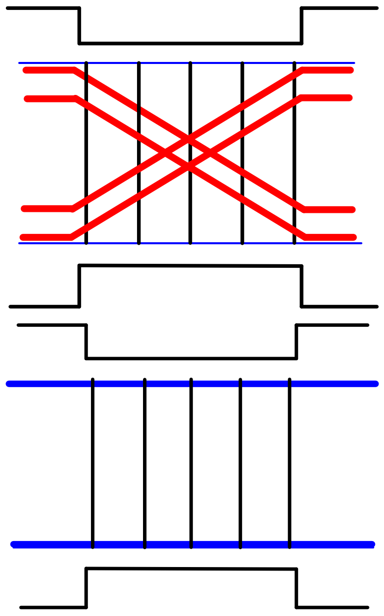
Lim, E., Hwang, S. J., Wang, T. W., and Chang, Y. H. (2016). "An Investigation on the Seismic Behavior of Deep Reinforced Concrete Coupling Beams," ACI Structural Journal, V. 113, No. 2, March-April, pp. 217-226.

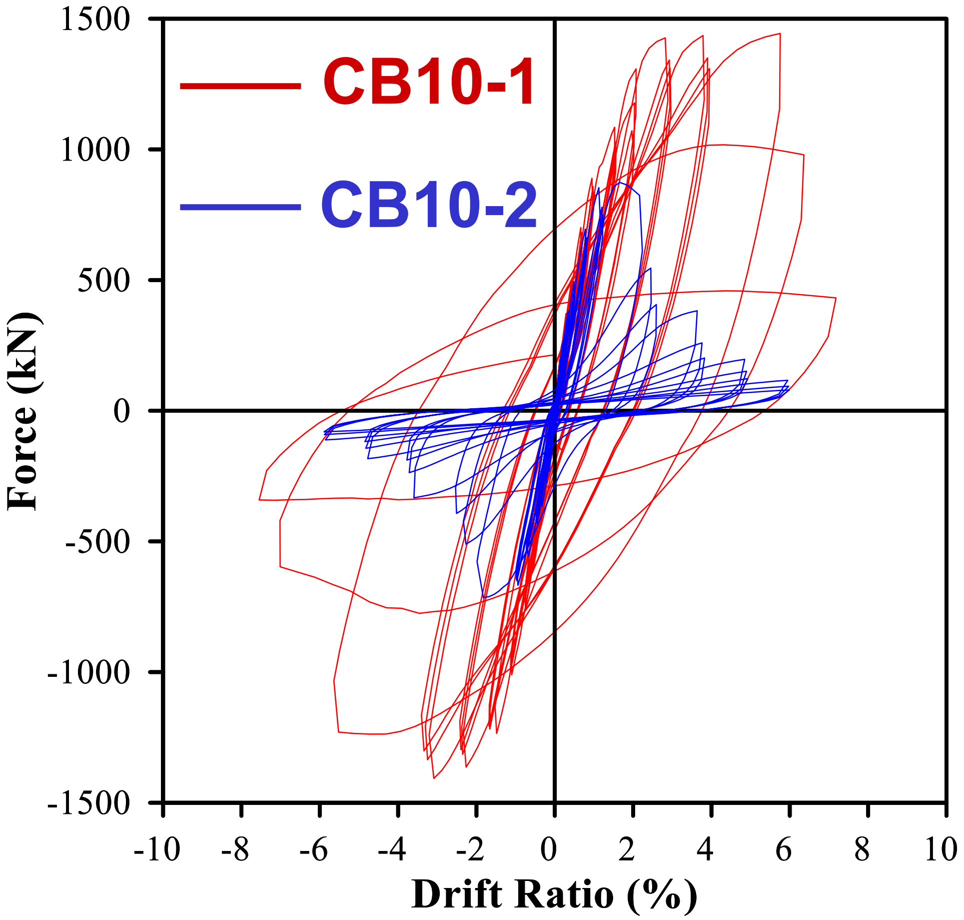
行為比較

$$\frac{\ell_n}{h} = 1.0$$

CB10-1

CB10-2

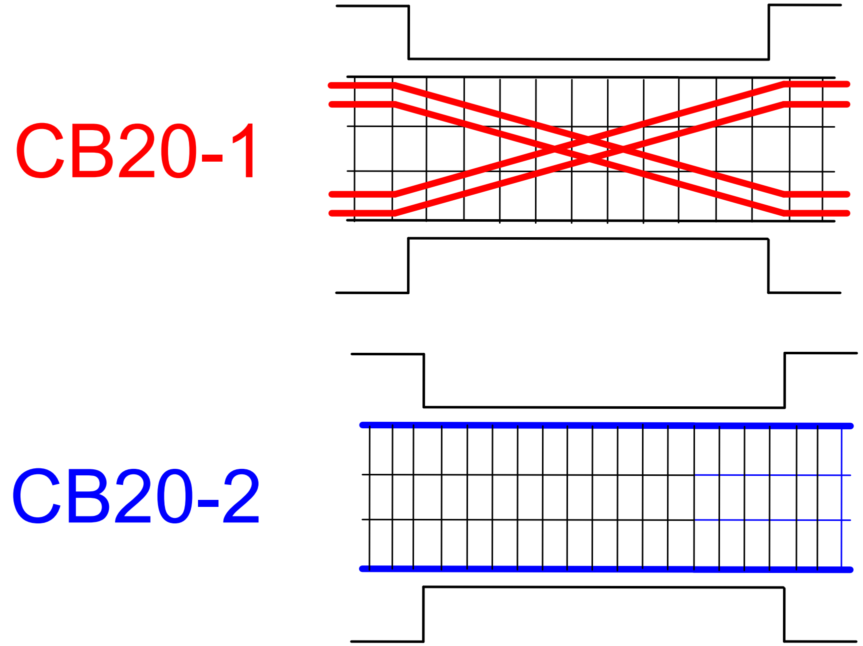


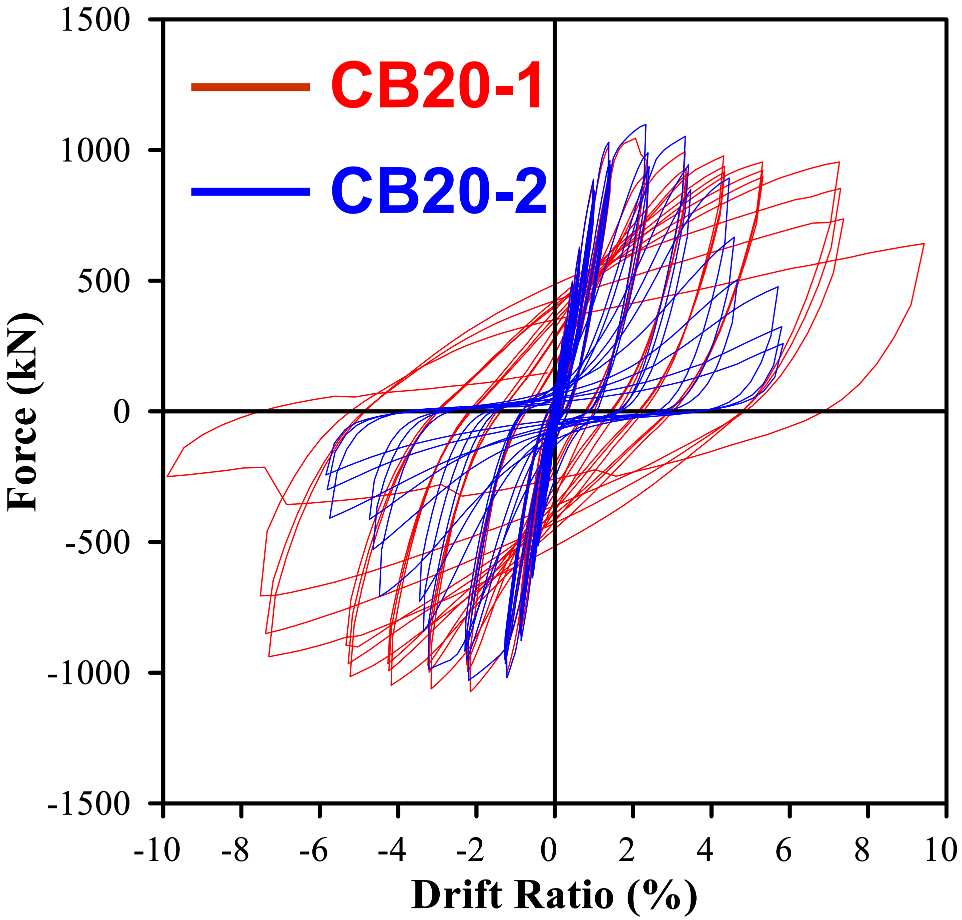


	Failure Mode	$\upsilon_{max}[MPa]$	Code Limit
CB10-1	Flexural Shear	$2.6\sqrt{f'_c}$	$\leq 0.83\sqrt{f'_c}$
CB10-2	Shear	$1.2\sqrt{f'_c}$	

行為比較

$$\frac{\ell_n}{h} = 2.0$$



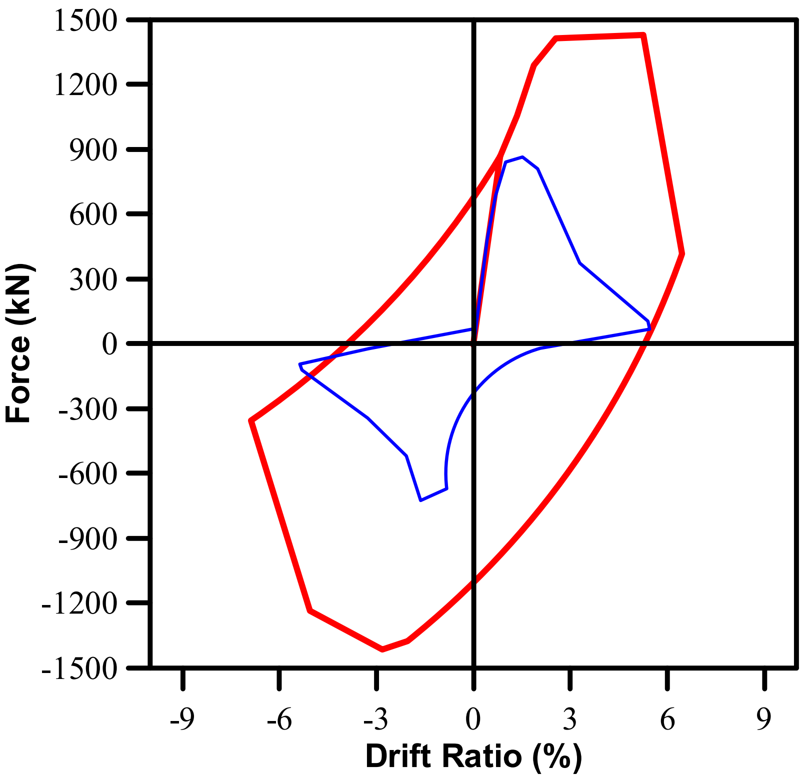


	Failure Mode	Ult. DR	Qual. DR	$\upsilon_{max}[MPa]$
CB20-1	Flexure	8.1%	5.3%	$1.22\sqrt{f_c'}$
CB20-2	Flexural Shear	4.3%	3.5%	$1.17\sqrt{f_c'}$

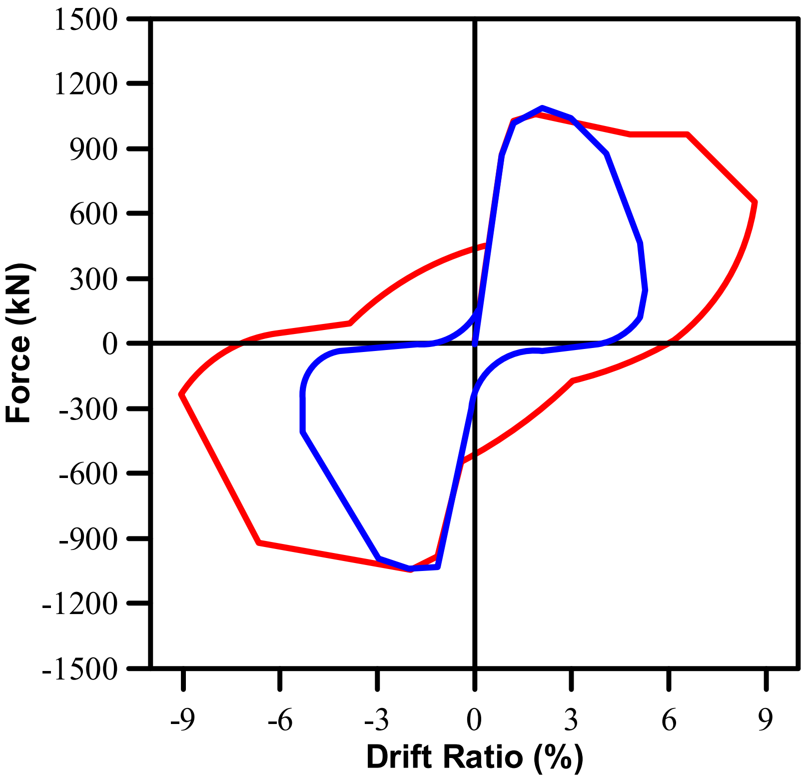
對角鋼筋之功能

— 對角鋼筋
— 傳統配筋

$$\frac{\ell_n}{h} = 1.0$$

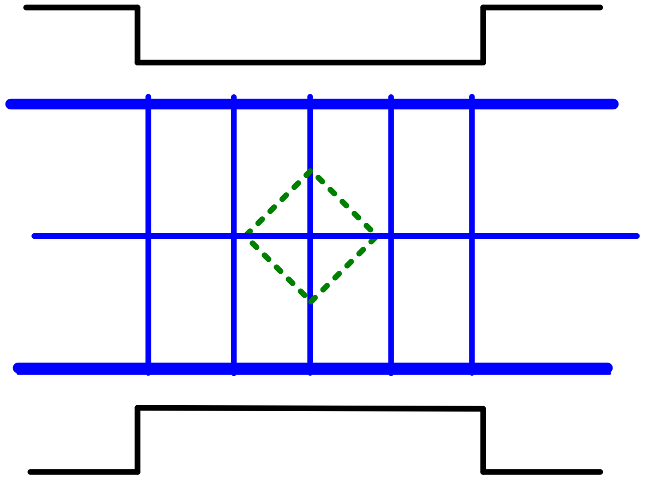


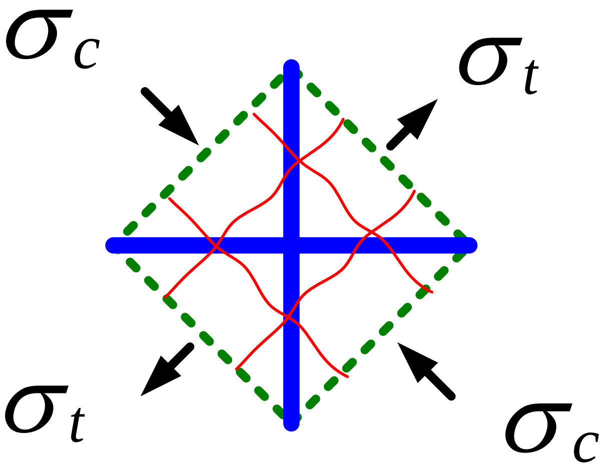
$$\frac{\ell_n}{h} = 2.0$$

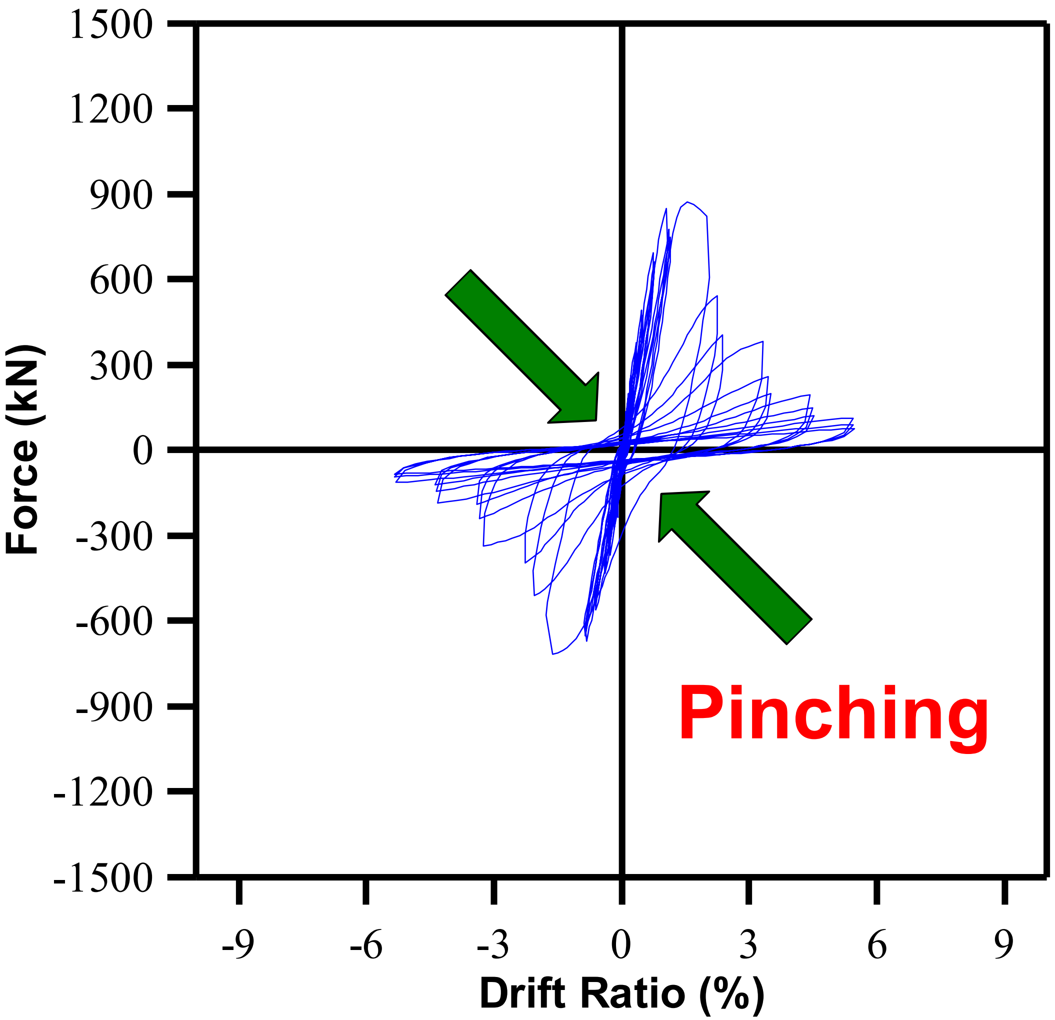


- 避免剪力束縮效應
- 提高剪力強度
- 增加變形能力

剪力束縮效應 - 傳統鋼筋配置

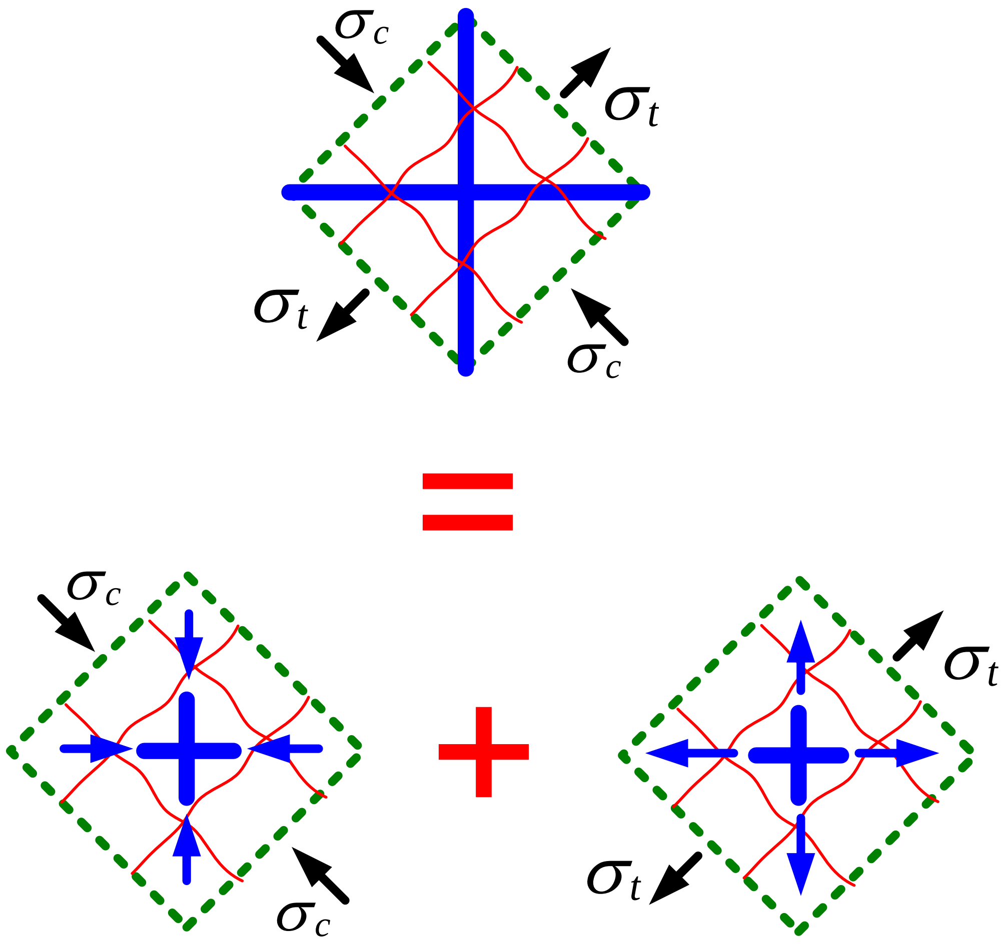




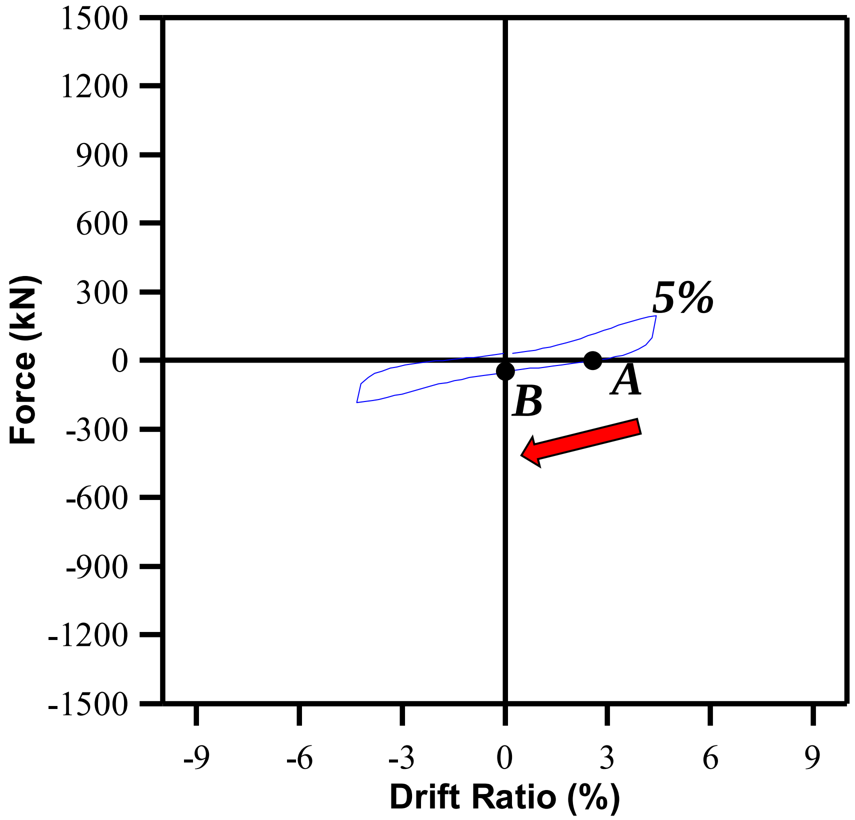


嚴重剪力束縮效應

(Hsu and Mo 2010)

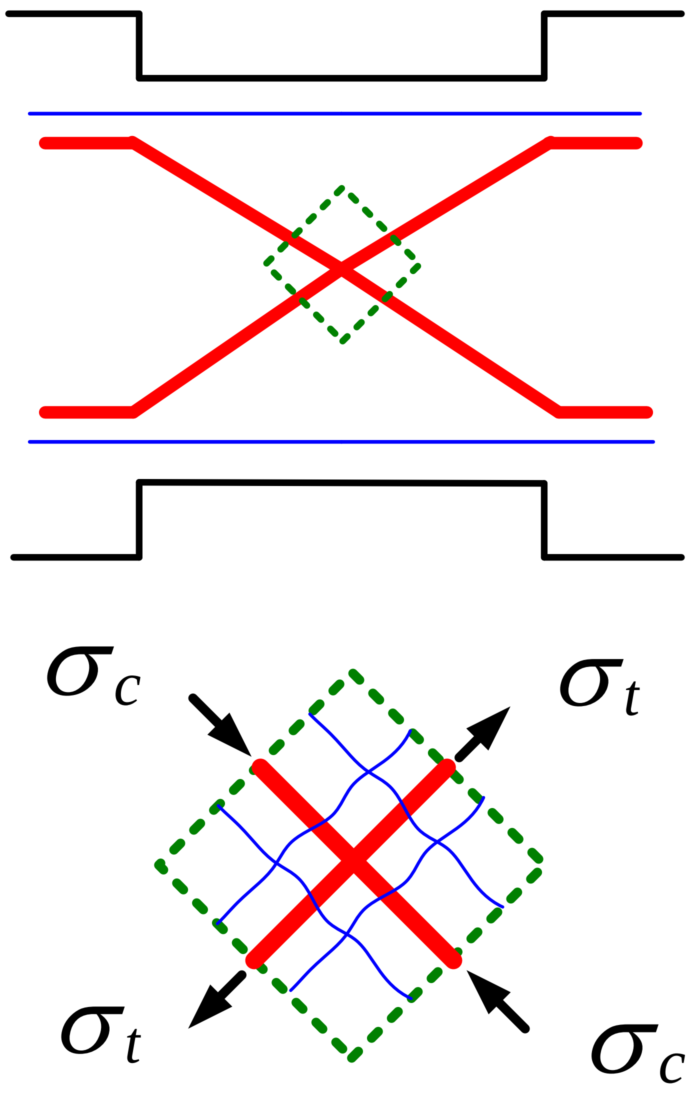


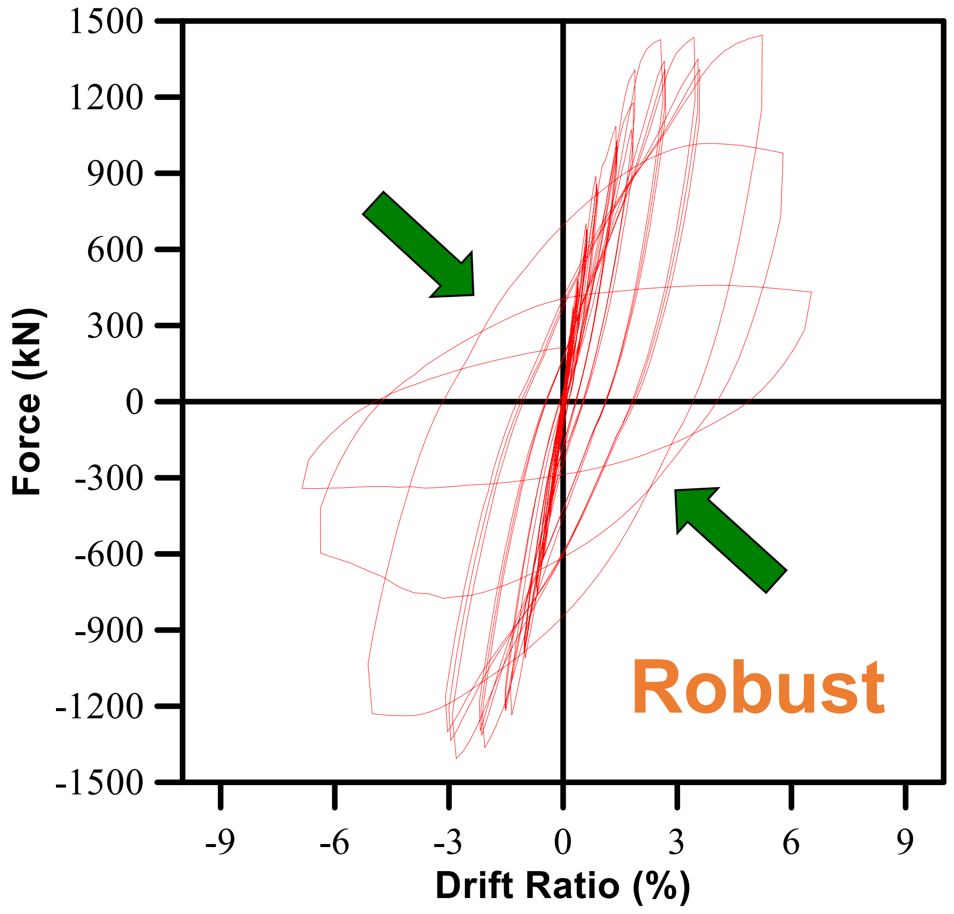
- **Large deformation:** Significant strain due to closing of cracks



- **Zero resistance:** No contribution from steel due to cancelation of steel stresses

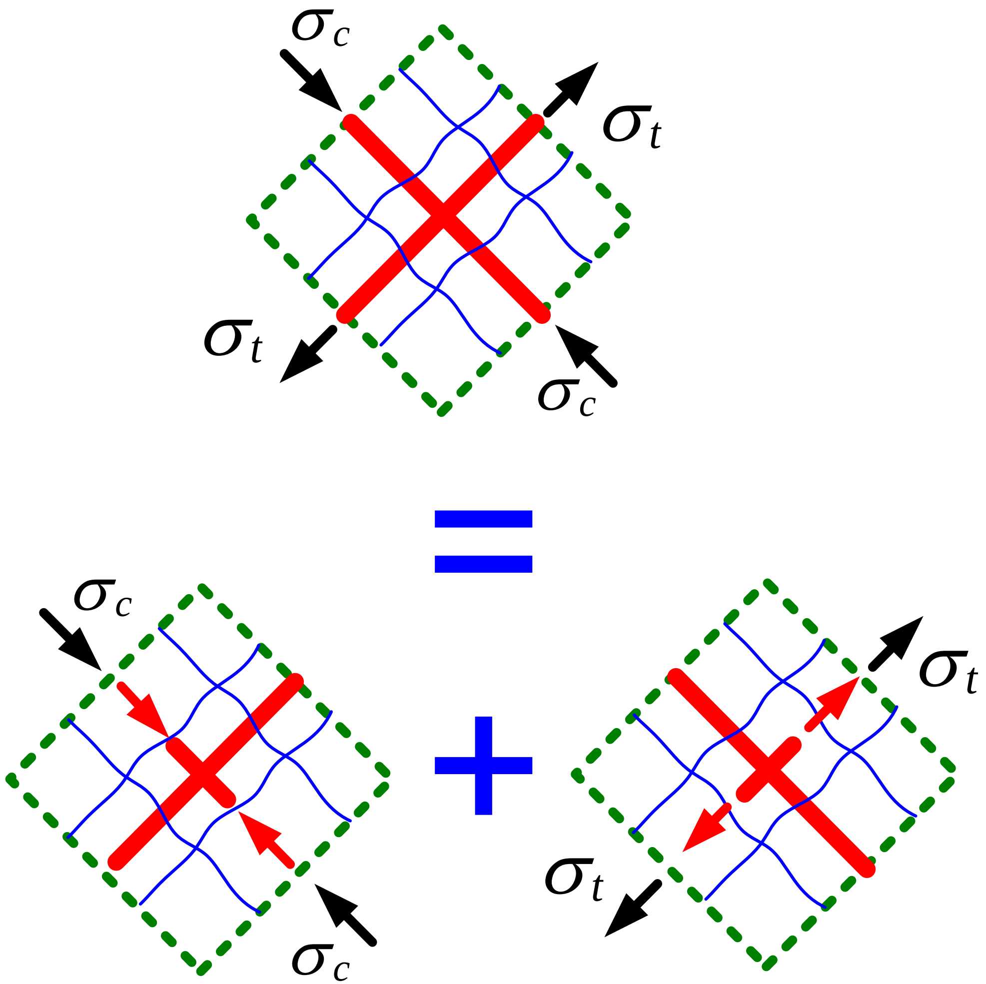
剪力束縮效應 - 對角鋼筋配置

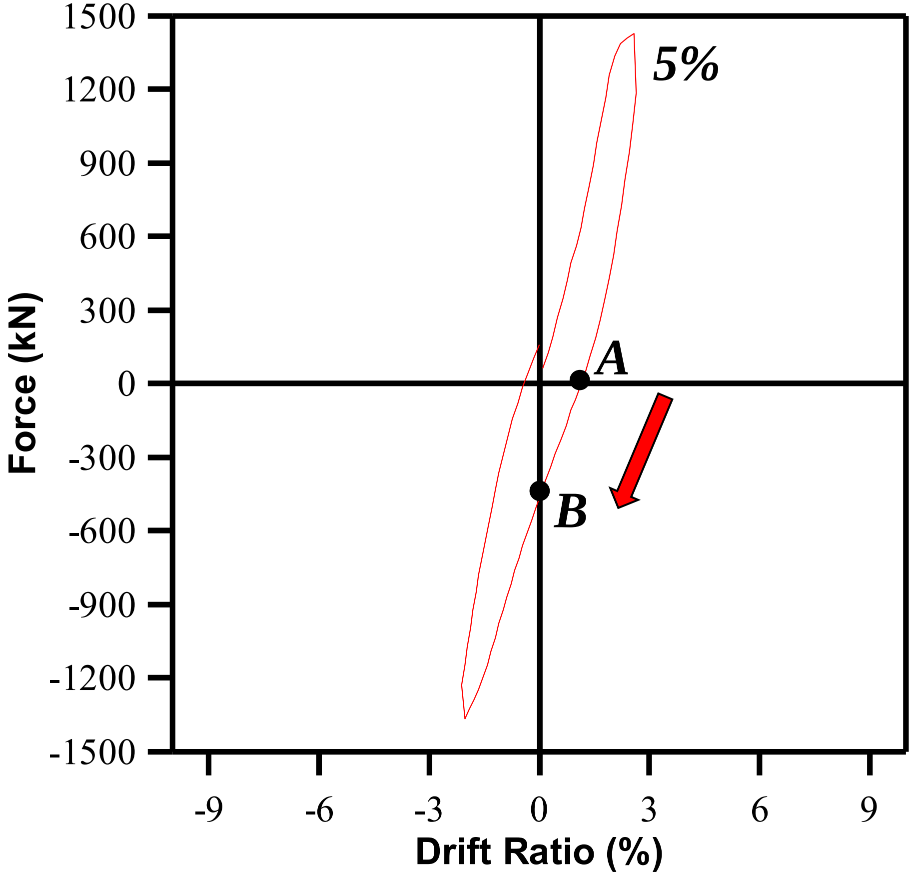




避免剪力束縮效應

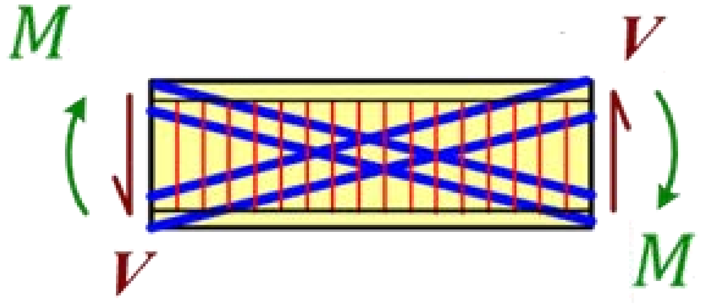
(Hsu and Mo 2010)

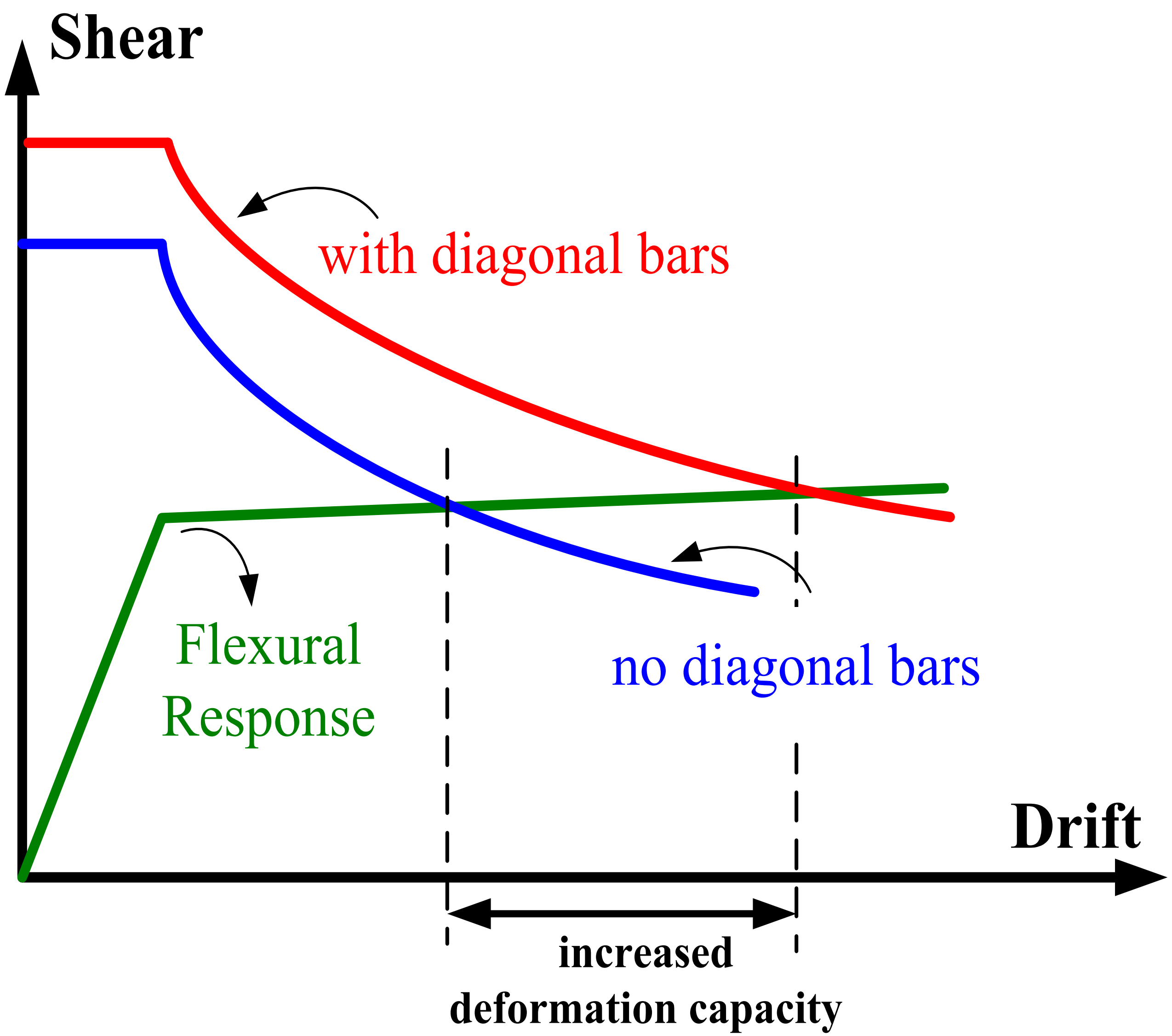




Resistance provided by diagonal bars even before closing of cracks

對角鋼筋→提高剪力強度→增加變形能力

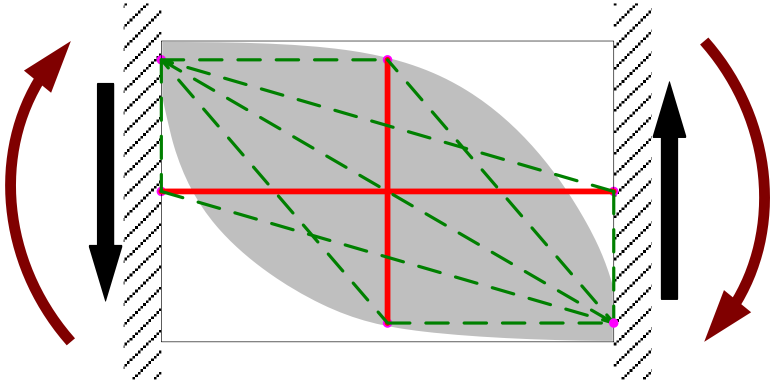




傳力機制與強度預測

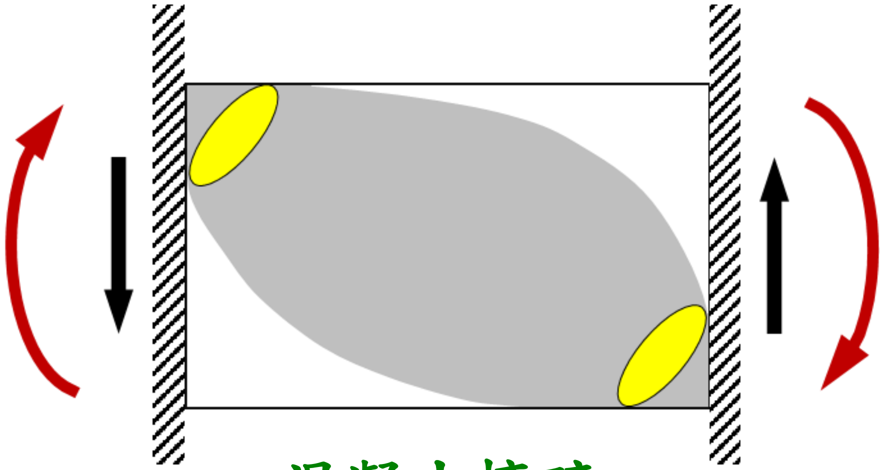
深短連接梁 - 傳統鋼筋配置

跨深比 $\frac{\ell_n}{h} \le 2$



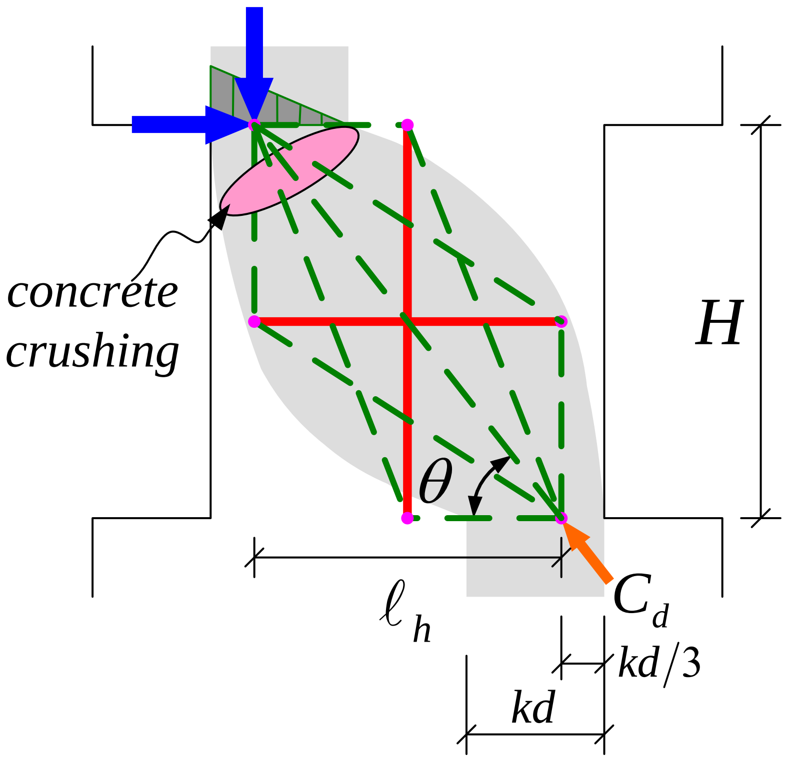
D區域

瓶狀壓桿



混凝土擠碎

軟化壓拉桿模型 – Softened Strut-and-Tie (SST) Model



對角壓力流

↓

力平衡

↓

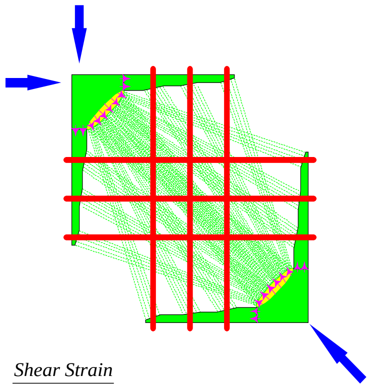
位移相容

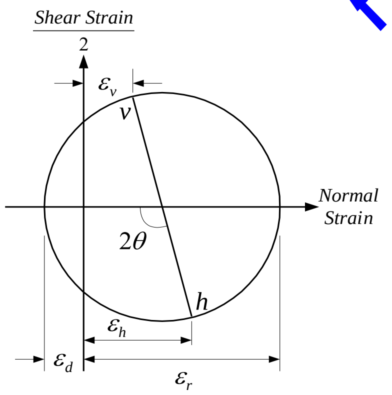
↓

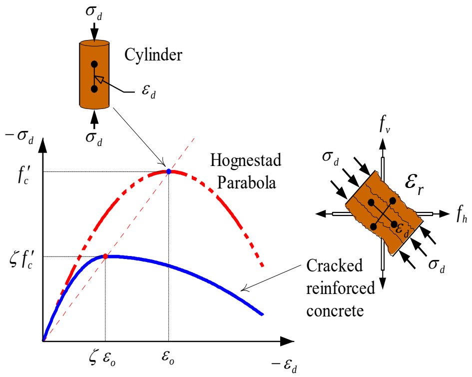
軟化組成率

↓

對角壓力破壞







- Hwang, S. J., and Lee, H. J., (2002) “Strength Prediction for Discontinuity Regions by Softened Strut-and-Tie Model,” Journal of Structural Engineering, ASCE, Vol. 128, No. 12, pp. 1519-1526.
- Hwang, S. J., Tsai, R. J., Lam, W. K., and Moehle, J. P. (2017). “Simplification of Softened Strut-and-Tie Model for Strength Prediction of Discontinuity Regions,” ACI Structural Journal, V. 114, No. 5, September-October, pp. 1239-1248.

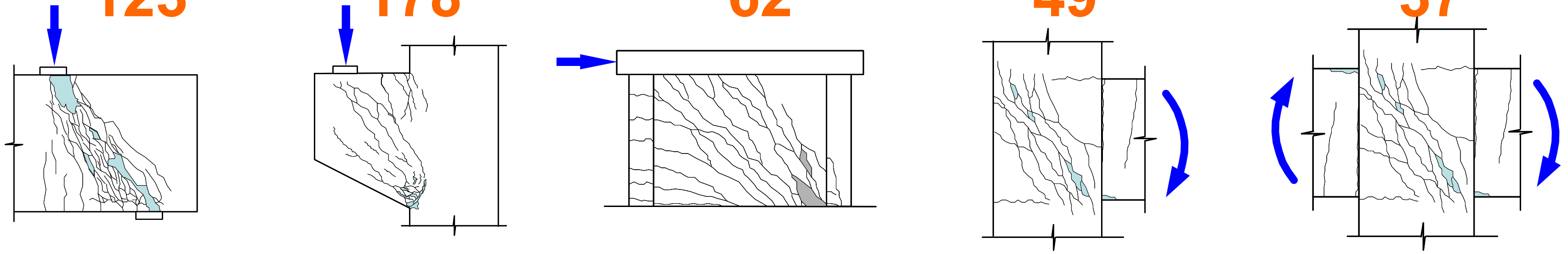
測試驗證

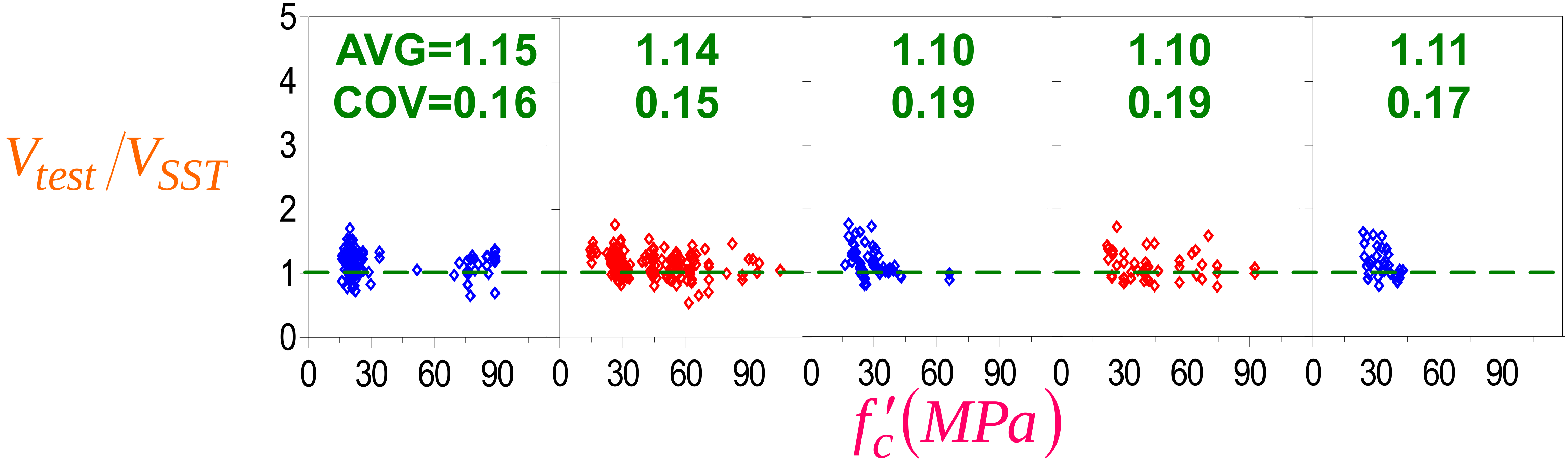
(449 specimens)

AVG = 1.14

COV=0.17

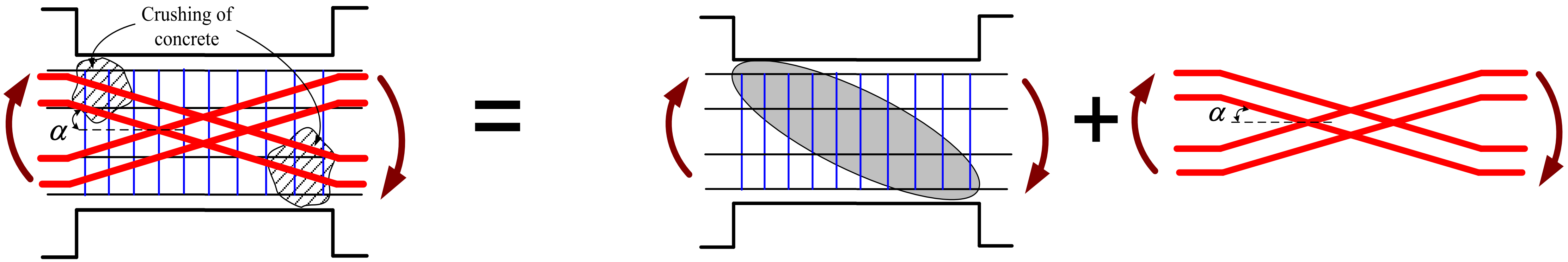
深梁	托架	剪力牆	外接頭	內接頭
123	178	62	49	37





深短連接梁 - 對角鋼筋配置

跨深比 $\frac{\ell_n}{h} \leq 2$



對角壓桿混凝土擠碎破壞

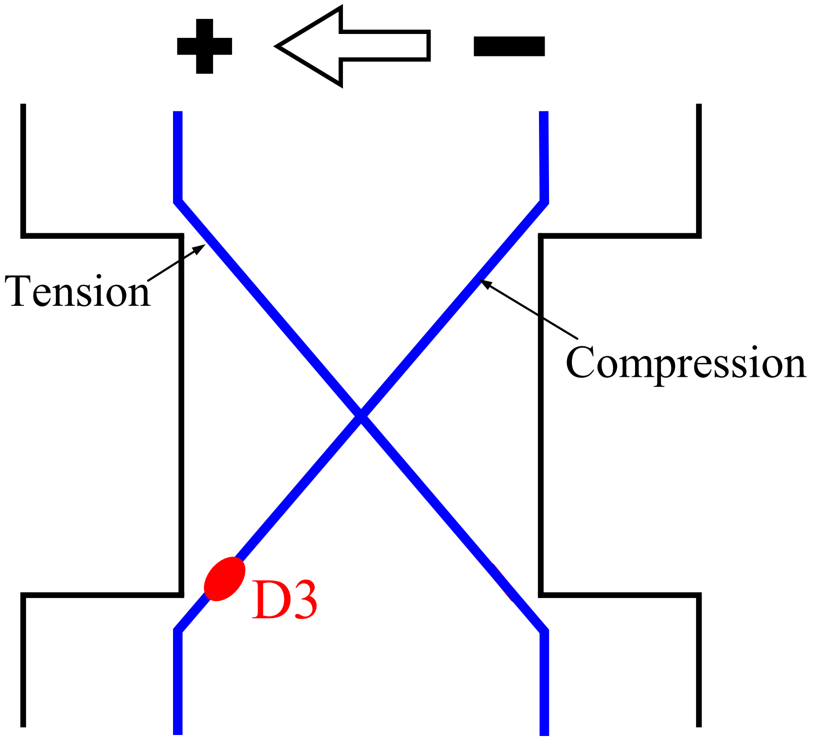
混凝土對角壓桿

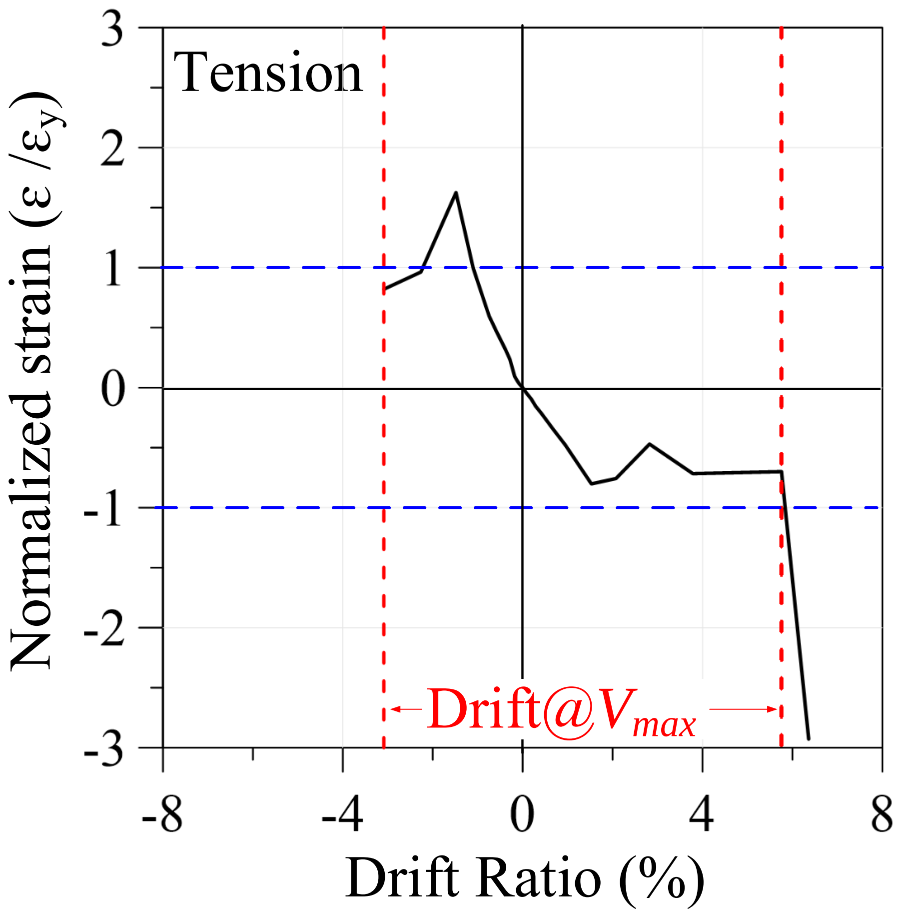
對角鋼筋

備註：對角鋼筋分擔作用剪力，延緩混凝土壓桿擠碎破壞的產生，故而提高剪力強度。

$\ell_n / h = 1.0$ **CB10-1**

對角鋼筋之應變硬化

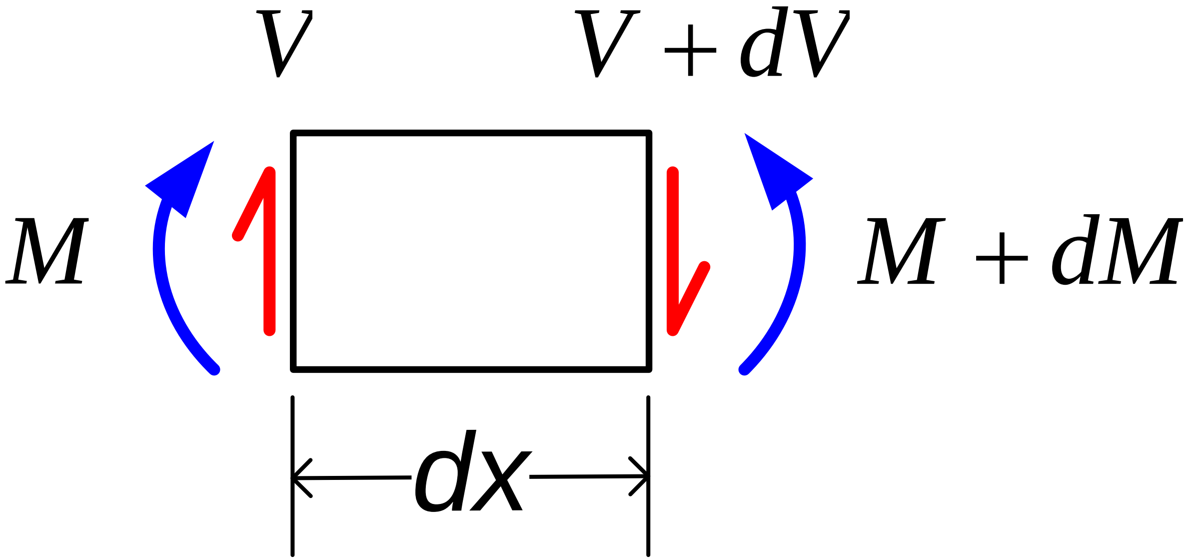




拉力鋼筋與壓力鋼筋均出現應變硬化之現象

王亭惟，(2011) 「鋼筋混凝土剪力連接梁耐震鋼筋配置之探討」，碩士論文，國立台灣大學土木工程學系，臺北，臺灣。

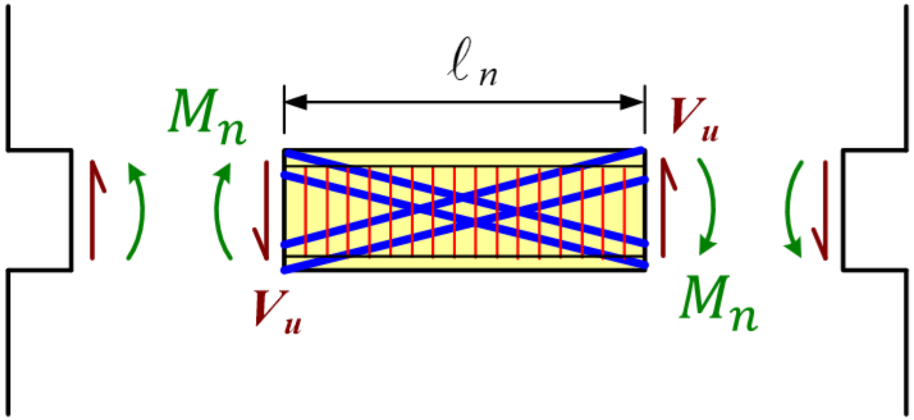
彎矩與剪力交互影響

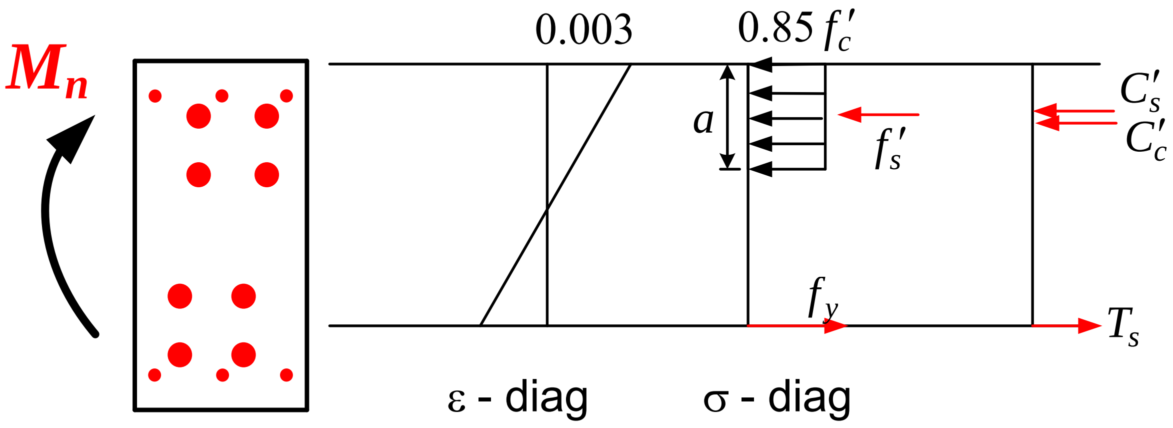


$$V = \frac{dM}{dx}$$

剪力設計必須考慮對應之彎矩行為

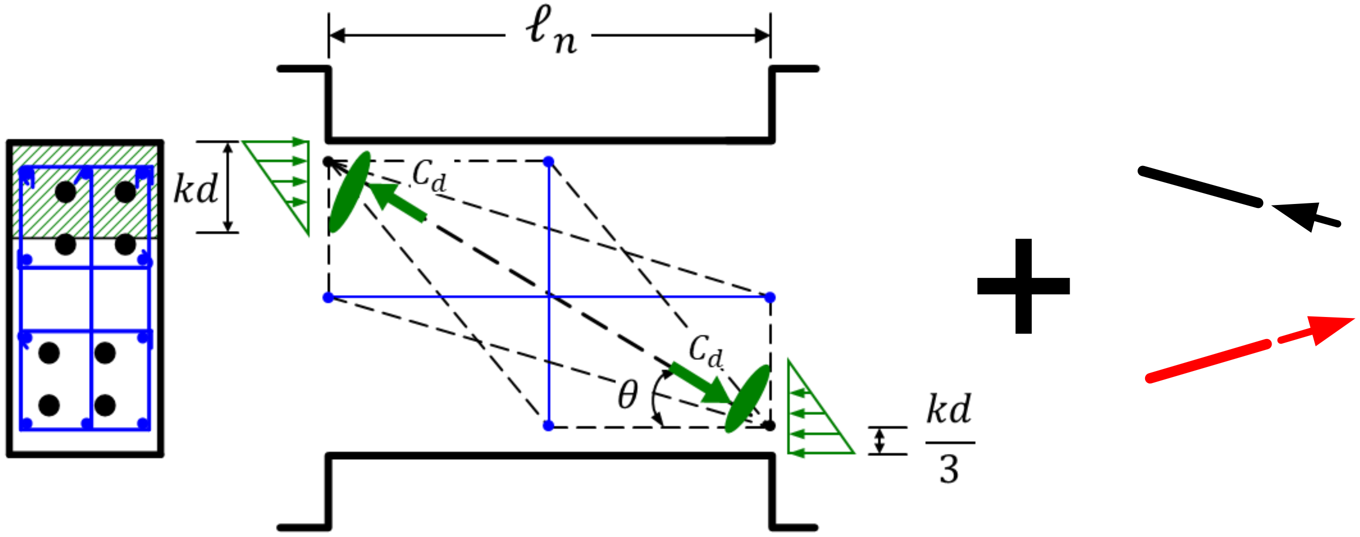
撓曲強度評估





- 斷面分析
- 平面維持平面
- $f_s = 1.0 f_y$ @DBE　　主筋降伏
- $f_s = 1.25 f_y$ @MCE　　主筋應變硬化

剪力強度評估 – Design Based Earthquake (DBE)



$$C_s = 1.0\, f_{yd} \times A_{vd}$$

$$T_s = 1.0\, f_{yd} \times A_{vd}$$

$$C_d \sin\theta = K\zeta f_c' A_{str} \sin\theta$$

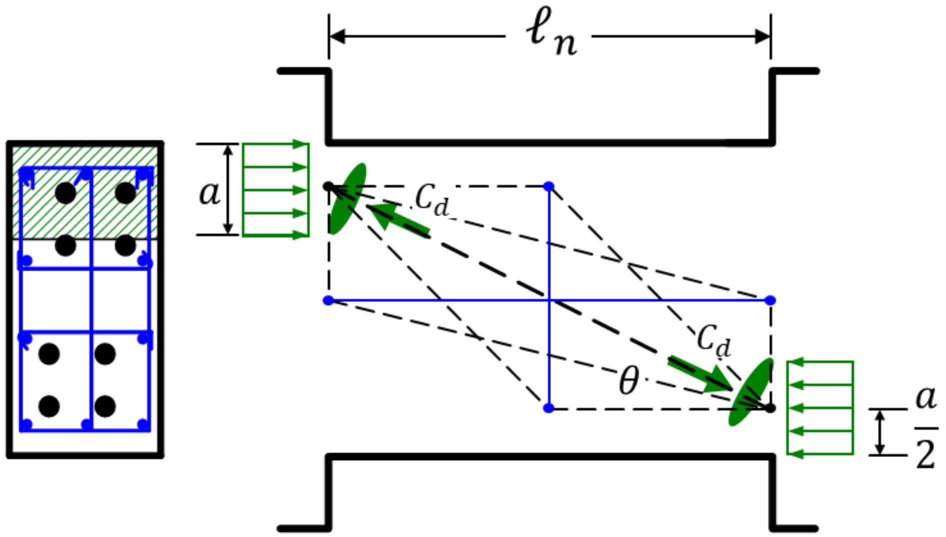
$$A_{str} = b_w \times kd \qquad kd = \left(\sqrt{(\rho_f n)^2 + 2\rho_f n} - \rho_f n\right) d$$

彈性壓力區深度

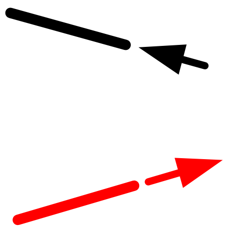
$$V_n = C_d \sin\theta + (1.0 + 1.0) A_{vd} f_{yd} \sin\alpha$$

忽略應變硬化

剪力強度評估 – Maximum Credible Earthquake (MCE)



+



$$C_s = 1.25 f_{yd} \times A_{vd}$$

$$T_s = 1.25 f_{yd} \times A_{vd}$$

$$C_d \sin\theta = K\zeta f_c' A_{str} \sin\theta$$

$$A_{str} = b_w \times a$$

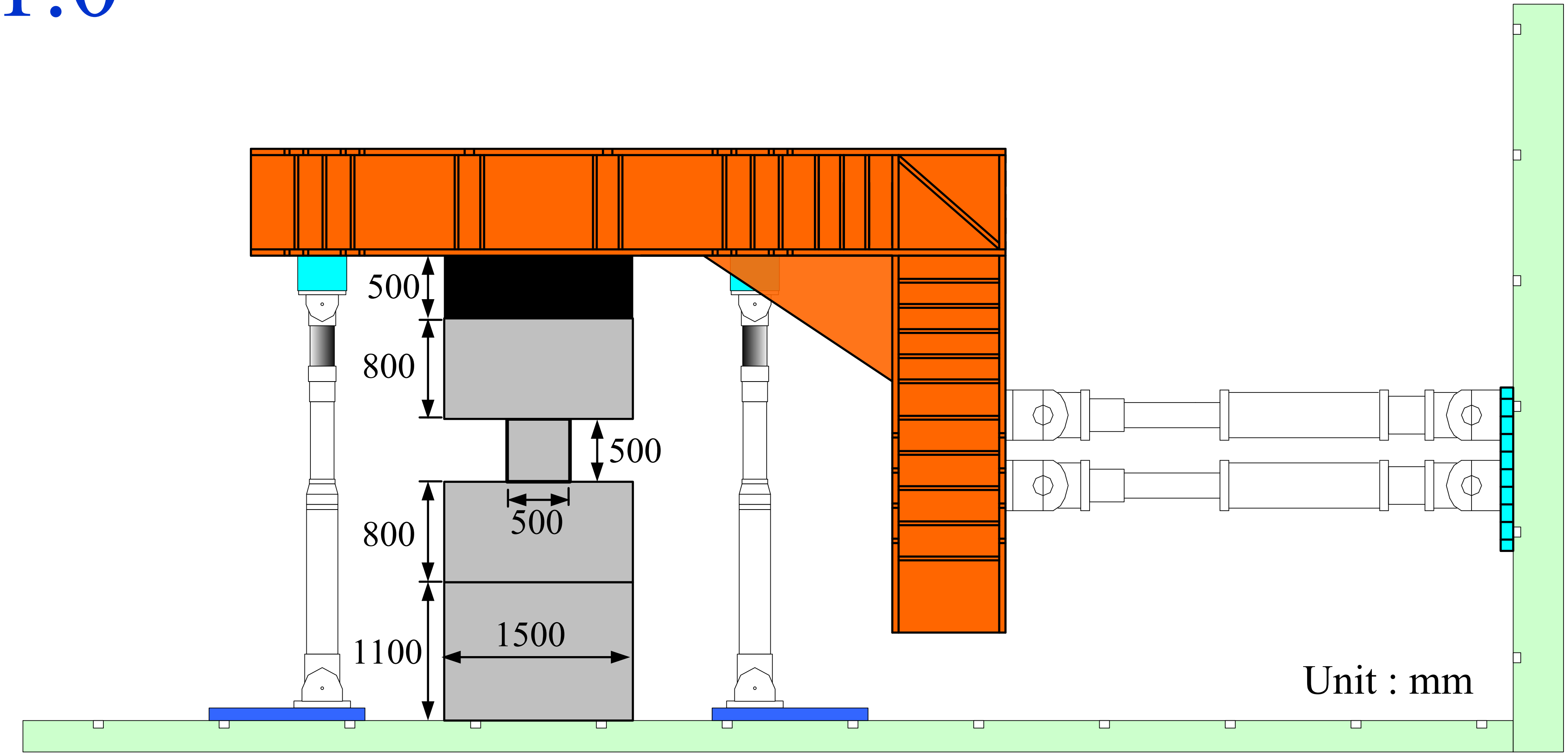
$$a = \frac{1.25 f_{yd} \times A_{vd} \cos\alpha}{0.85 f_c' \times b_w}$$

塑性壓力區深度

$$V_n = C_d \sin\theta + (1.25 + 1.25) A_{vd} f_{yd} \sin\alpha$$

塑鉸應變硬化

$$\frac{\ell_n}{h} = 1.0$$

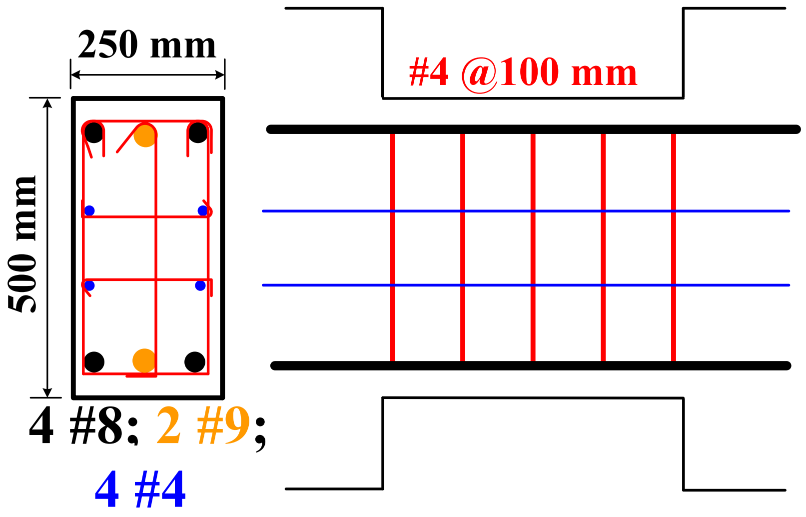


Lim, E., Hwang, S. J., Wang, T. W., and Chang, Y. H. (2016). “An Investigation on the Seismic Behavior of Deep Reinforced Concrete Coupling Beams,” ACI Structural Journal, V. 113, No. 2, March-April, pp. 217-226.

實驗試體

$\ell_n / h = 1.0$

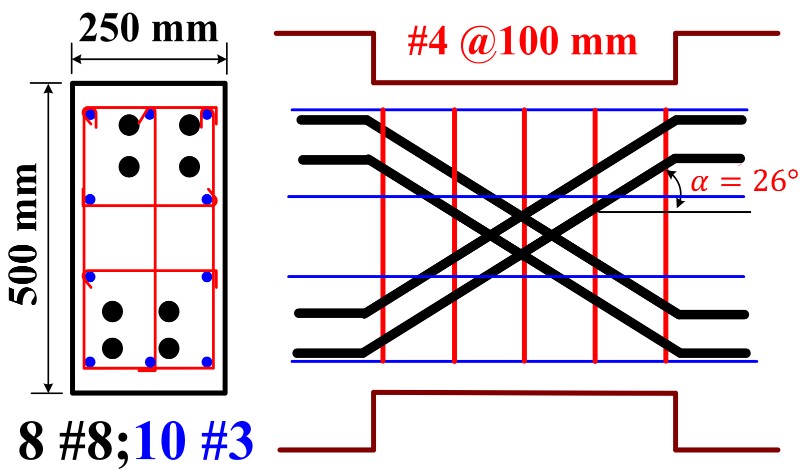
CB10-2



$f'_c = 35.4\ MPa$

$\rho_f = 1.53\%$

CB10-1

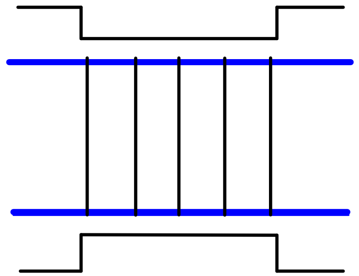


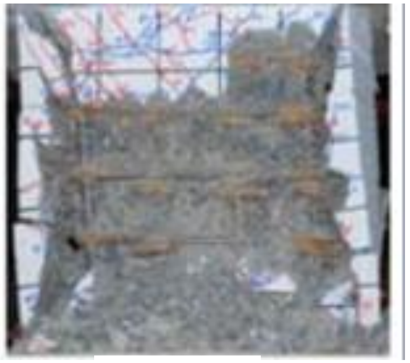
$f'_c = 33.7\ MPa$

$\rho_f = 2.04\%$

試驗結果

CB10-2

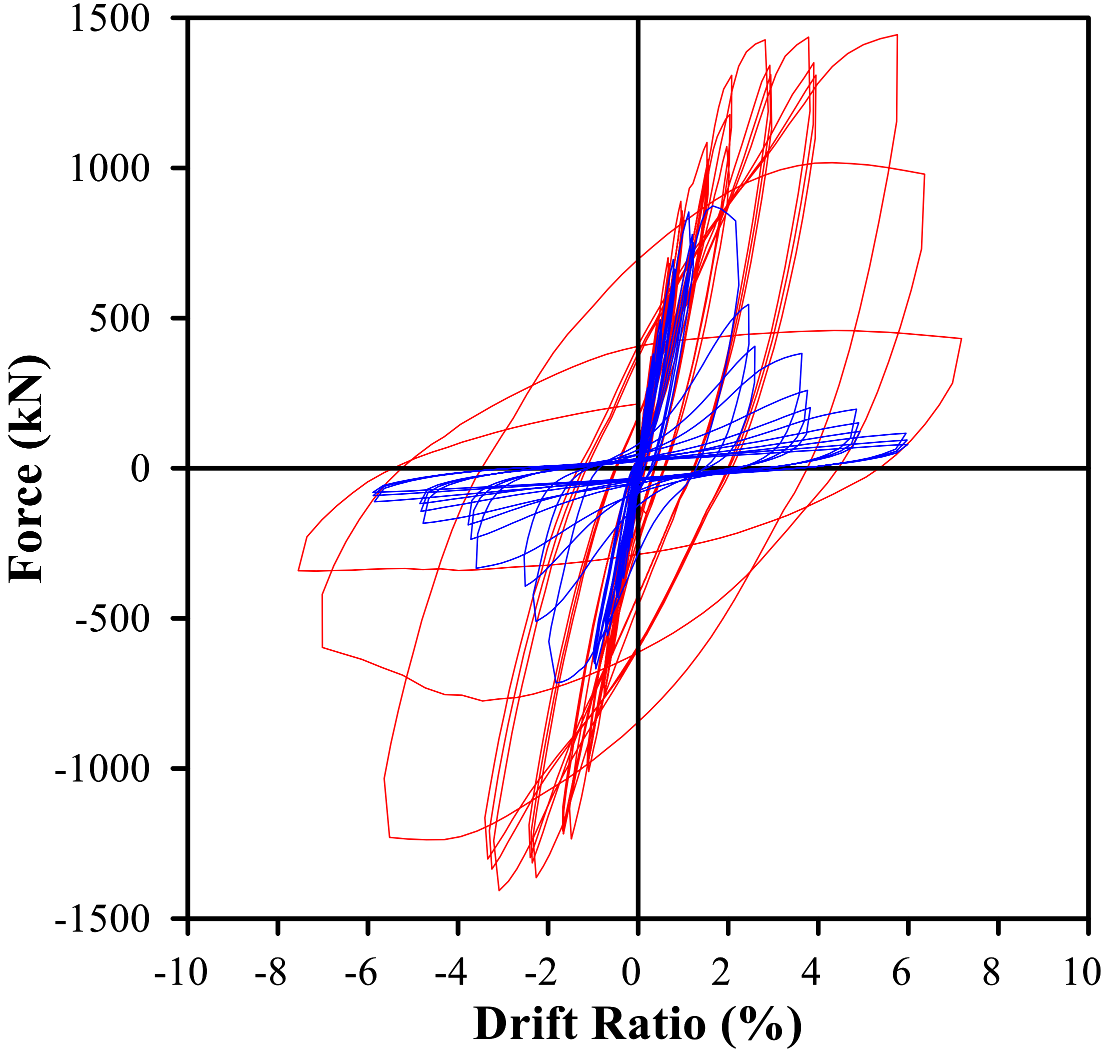




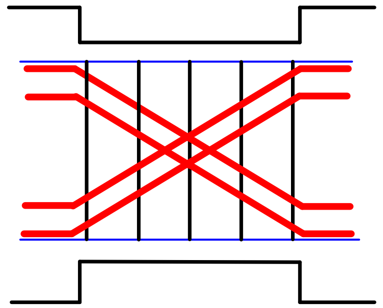
6.0%

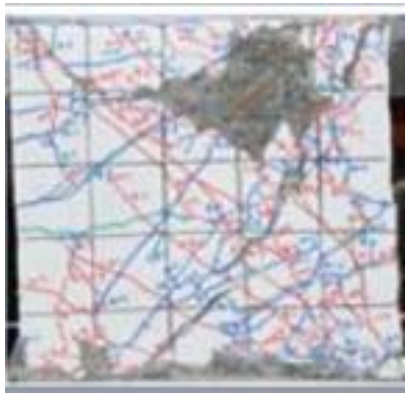
$$\upsilon = 1.34\sqrt{f_c'}$$

UDR = 2.1%



CB10-1





7.2%

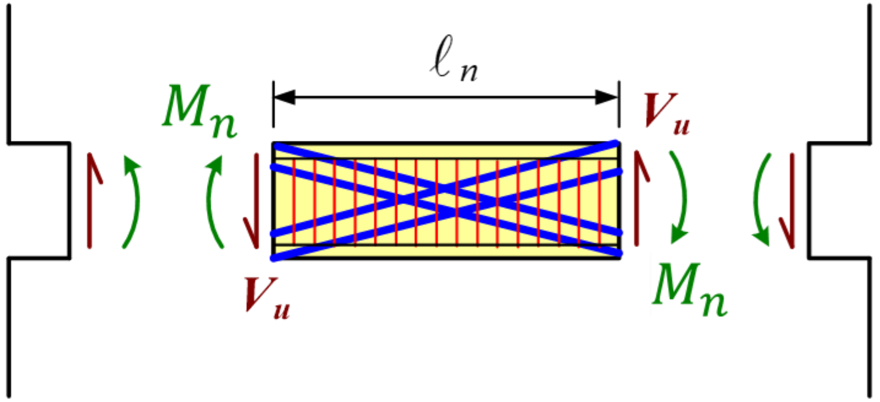
$$\upsilon = 2.64\sqrt{f_c'}$$

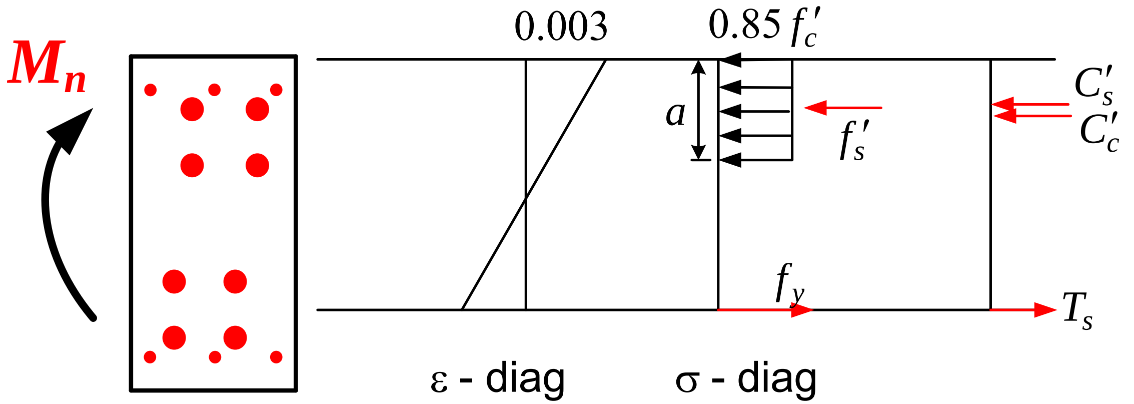
UDR = 5.7%

撓曲強度之計算流程

撓曲強度:

由 **XTRACT** 取得斷面標稱彎矩 $M_n \Rightarrow V_{mn} = \dfrac{2M_n}{\ell_n}$

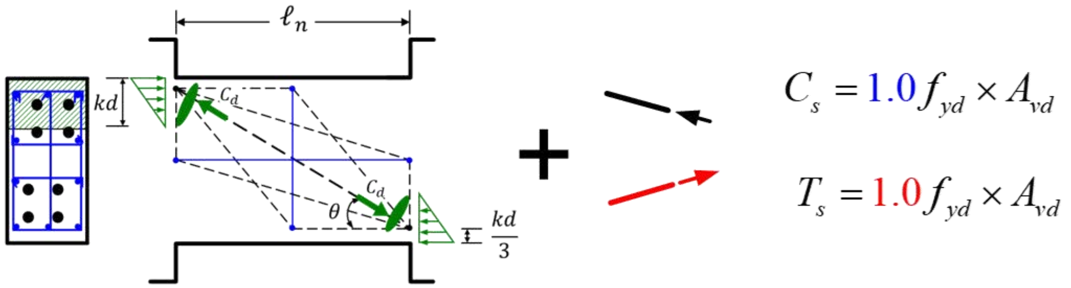




- 斷面分析 - XTRACT
- 平面維持平面

DBE剪力強度之計算流程

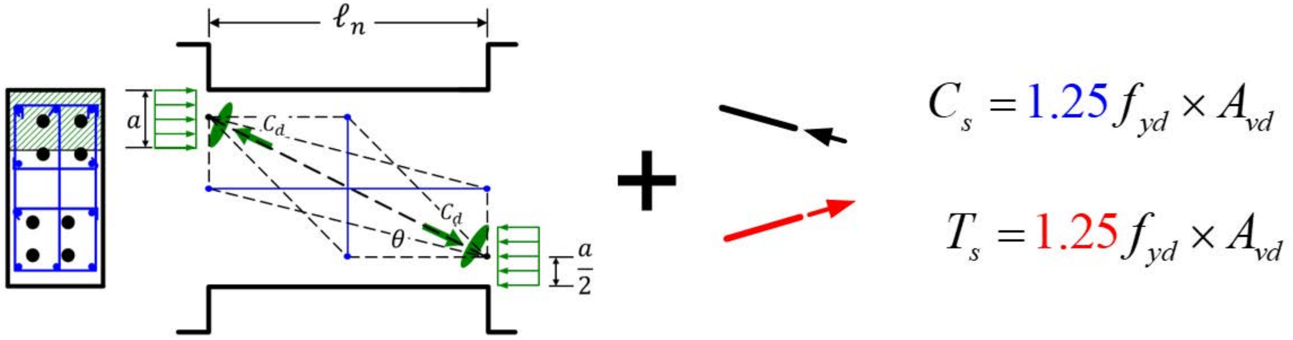
Strength (DBE):



1. 彈性壓力區深度 $kd = \left(\sqrt{(\rho_f n)^2 + 2\rho_f n} - \rho_f n\right) d$
2. 對角壓桿之有效面積 $A_{str} = b_w \times kd$
3. 對角壓桿之水平仰角 $\theta = \tan^{-1}\left(\frac{h - 2\times kd/3}{\ell_n}\right)$
4. 壓拉桿指標 $K = \tan^A \theta + \cot^A \theta - 1 + 0.14B \leq 1.64$
5. 開裂鋼筋混凝土軟化係數 $\zeta = \frac{3.35}{\sqrt{f_c'}} \leq 0.52$
6. 對角壓桿之垂直剪力強度 $C_d \sin\theta = K\zeta f_c' A_{str} \sin\theta$
7. 計算DBE需求下之剪力強度 $V_{n@DBE} = C_d \sin\theta + (1.0 + 1.0) A_{vd} f_{yd} \sin\alpha$

MCE剪力強度之計算流程

Strength (MCE):

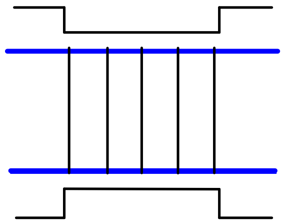


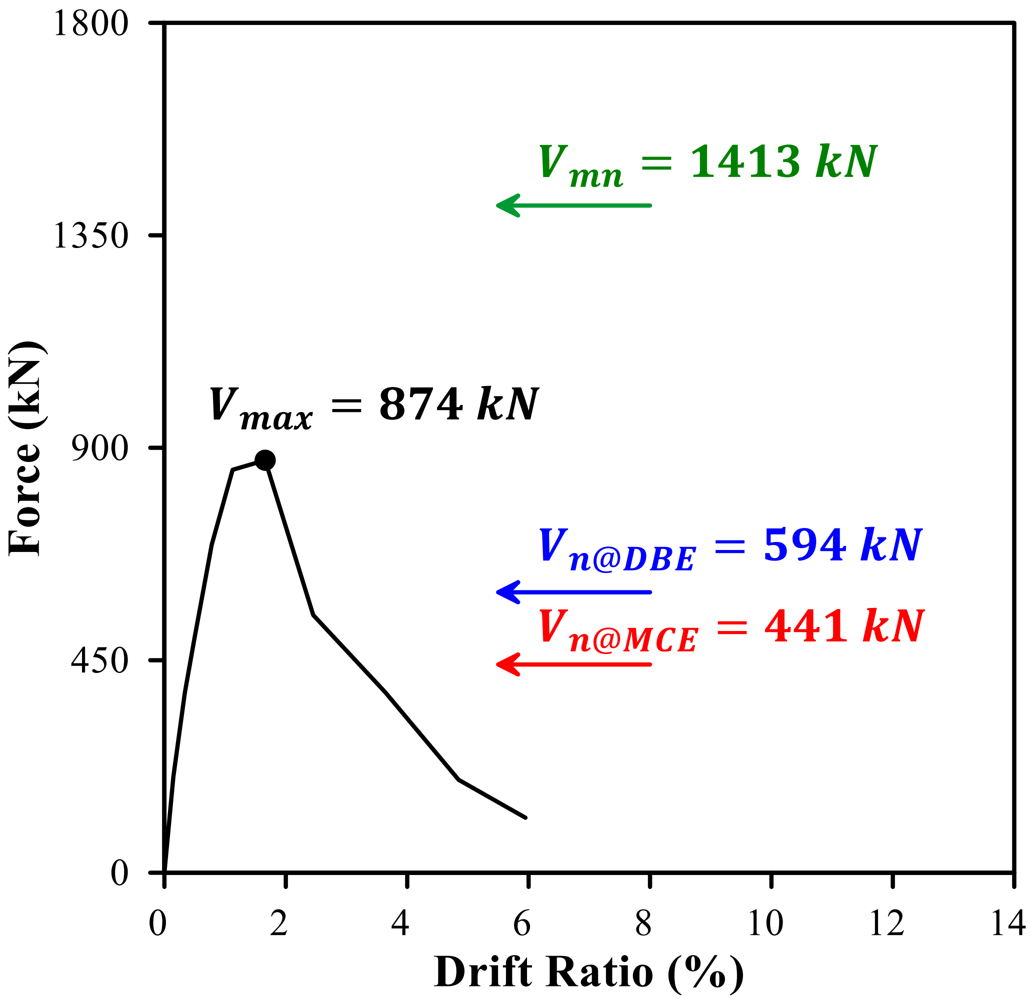
1. 壓力區深度 $a = \frac{1.25 f_{yd} \times A_{vd} \cos\alpha}{0.85 f'_c \times b_w}$
2. 對角壓桿之有效面積 $A_{str} = b_w \times a$
3. 對角壓桿之水平仰角 $\theta = \tan^{-1}\left(\frac{h - 2 \times a/2}{\ell_n}\right)$
4. 壓拉桿指標 $K = \tan^A\theta + \cot^A\theta - 1 + 0.14B \le 1.64$
5. 開裂鋼筋混凝土軟化係數 $\zeta = \frac{3.35}{\sqrt{f'_c}} \le 0.52$
6. 對角壓桿之垂直剪力強度 $C_d \sin\theta = K\zeta f'_c A_{str} \sin\theta$
7. 計算MCE需求下之剪力強度 $V_{n@MCE} = C_d \sin\theta + (1.25 + 1.25) A_{vd} f_{yd} \sin\alpha$

$\ell_n / h = 1.0$

撓曲與剪力強度評估

CB10-2



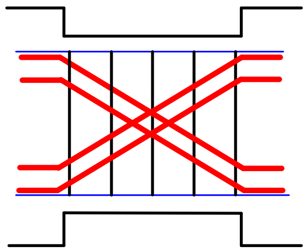


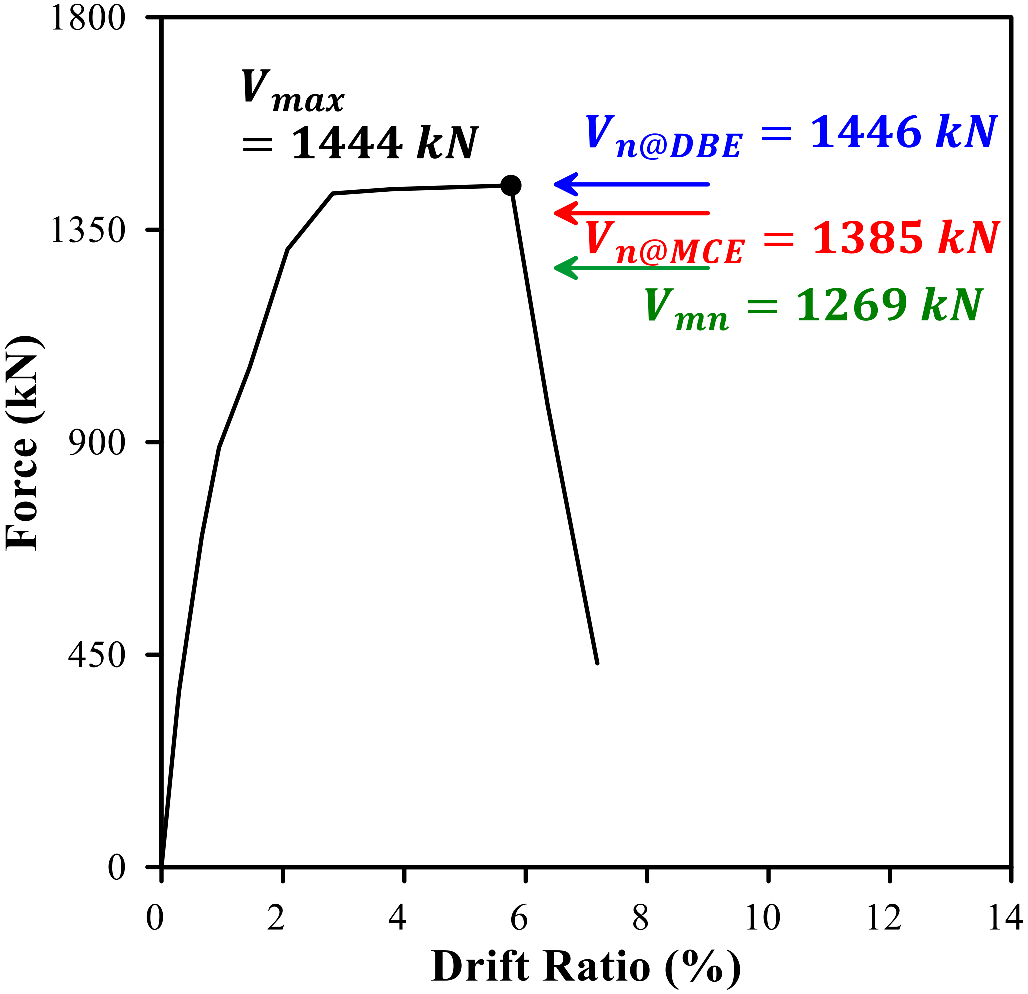
1. $V_{n@DBE} < V_{mn}$
2. 剪力強度主控行為
3. $\frac{V_{max}}{V_{n@DBE}} = 1.47$

$\ell_n / h = 1.0$

撓曲與剪力強度評估

CB10-1



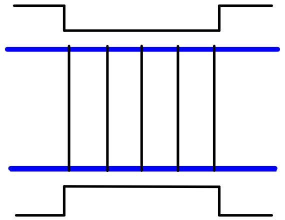


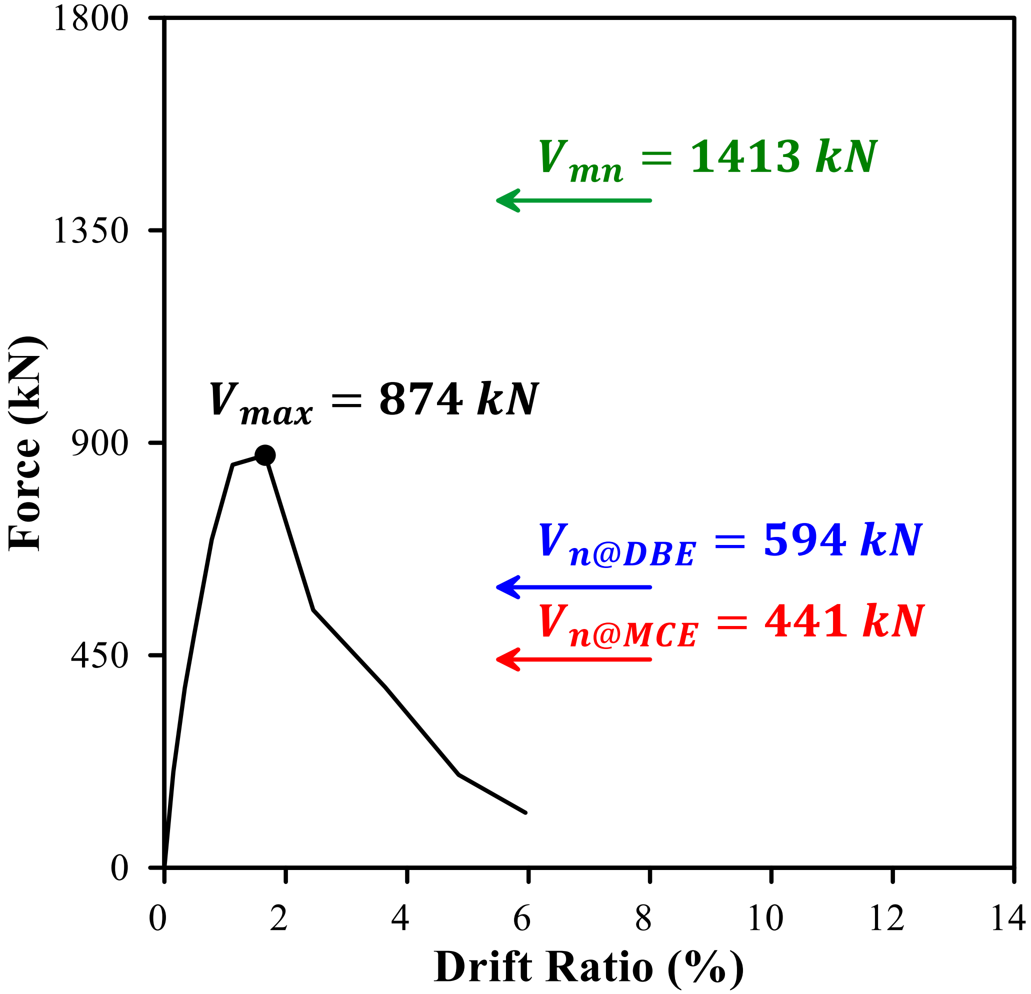
1. $V_{mn} < V_{n@DBE}$
2. 撓曲強度主控行為
3. $\frac{V_{max}}{V_{mn}} = 1.14$
4. $V_{n@MCE} \approx V_{max}$
5. 撓曲塑鉸變形充分發展
6. $\frac{V_{n,@MCE}}{V_{max}} = 0.96$

$\ell_n / h = 1.0$

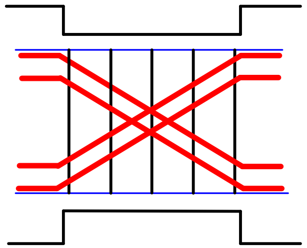
測試行為評估

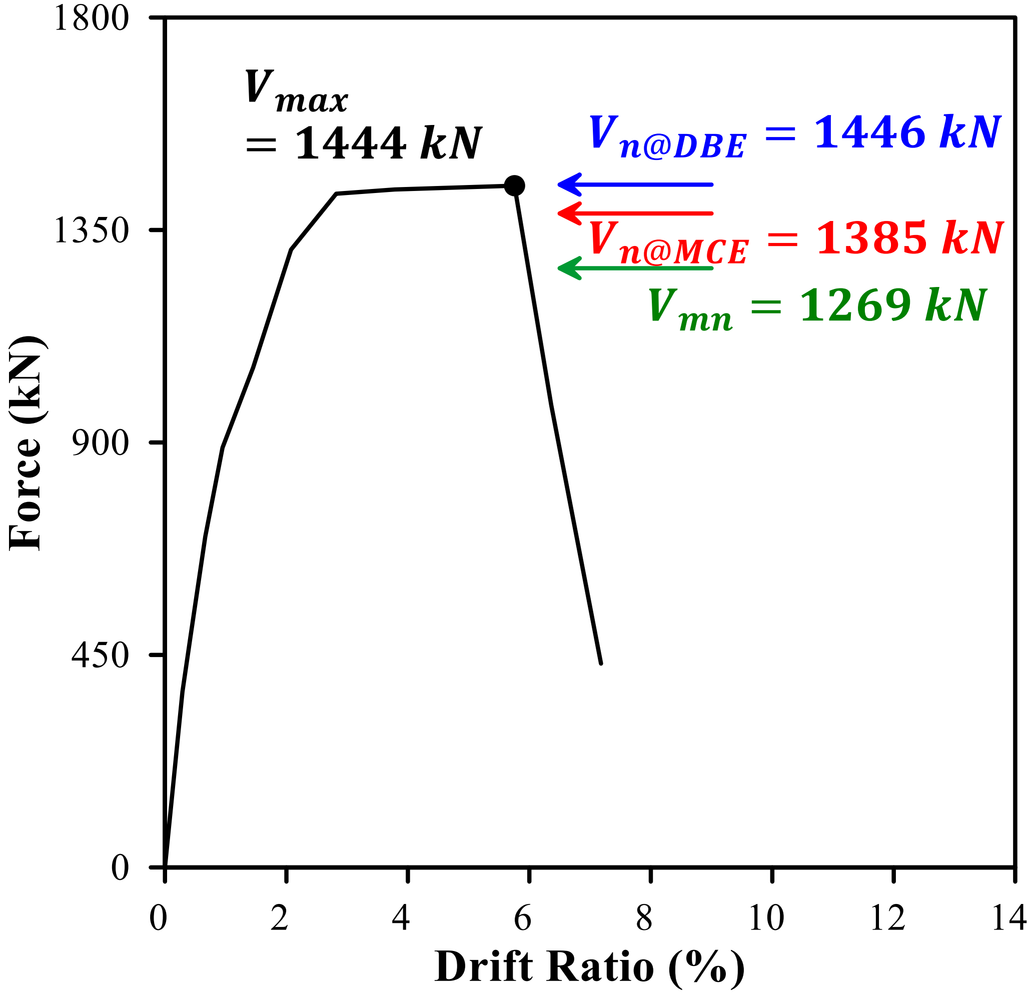
CB10-2



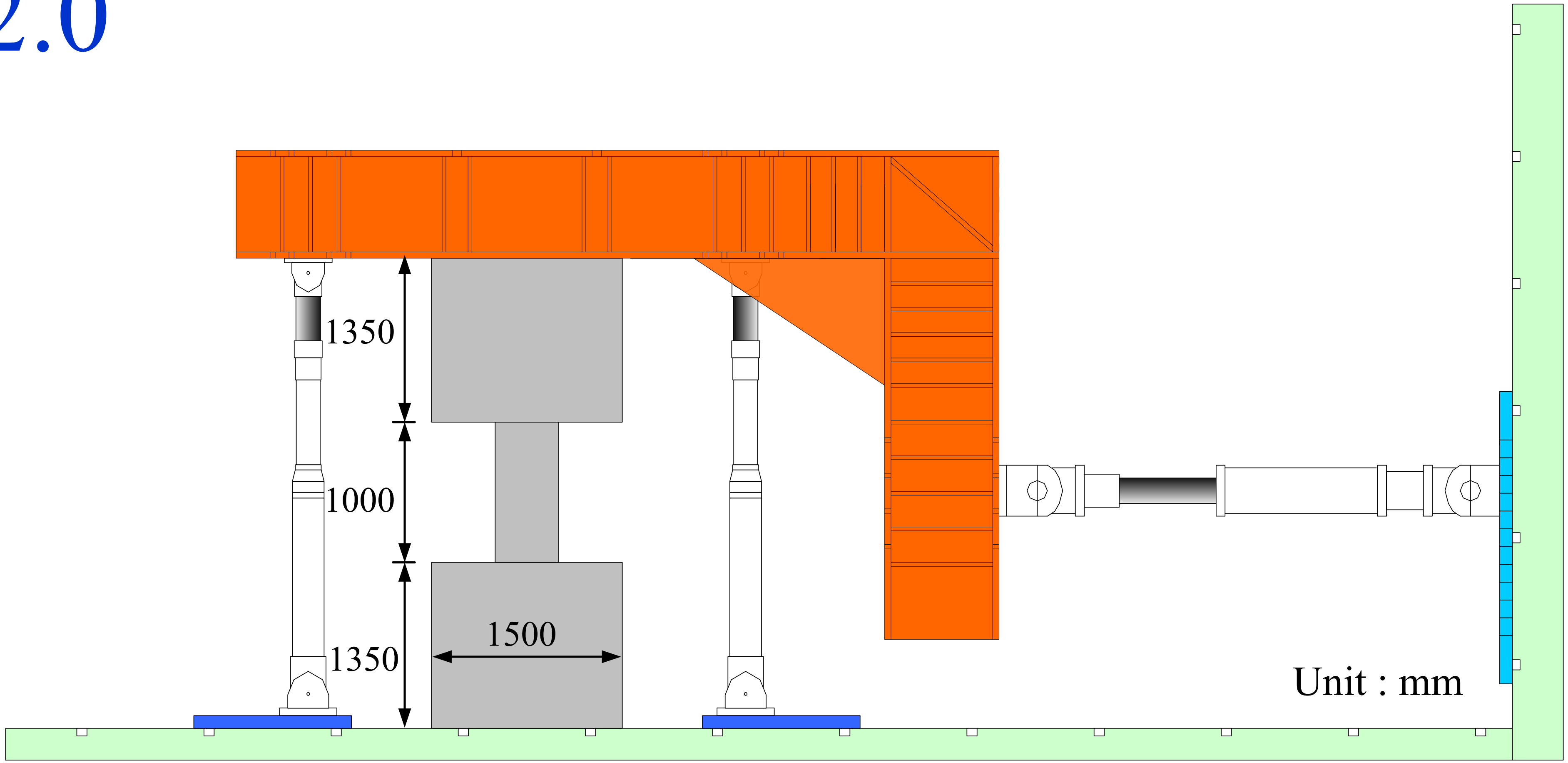


CB10-1





$$\frac{\ell_n}{h} = 2.0$$

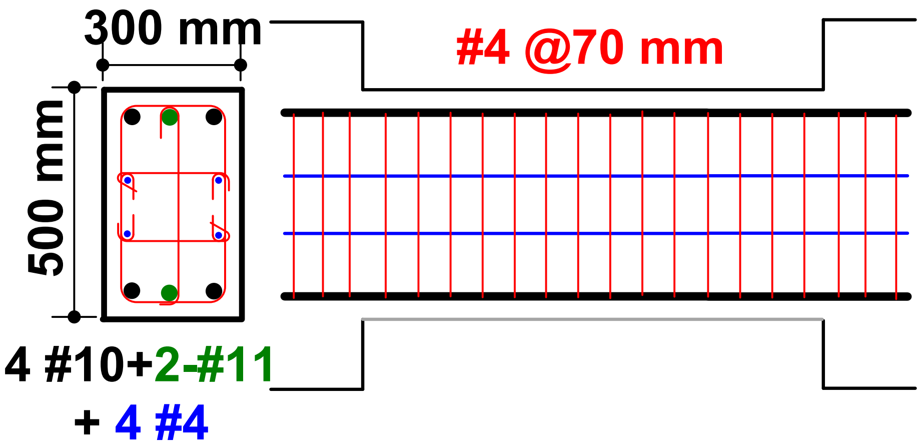


Lim, E., Hwang, S. J., Wang, T. W., and Chang, Y. H. (2016). "An Investigation on the Seismic Behavior of Deep Reinforced Concrete Coupling Beams," ACI Structural Journal, V. 113, No. 2, March-April, pp. 217-226.

$\ell_n / h = 2.0$

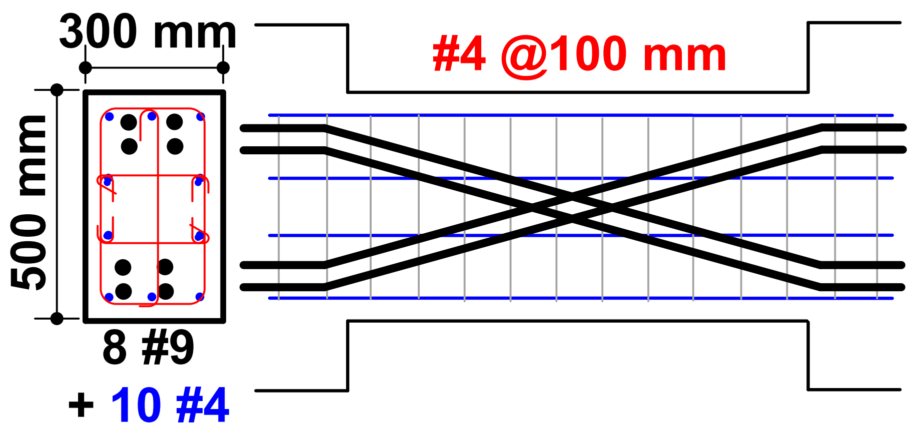
實驗試體

CB20-2



$$f_c' = 54.2MPa$$
$$\rho_f = 2.05\%$$

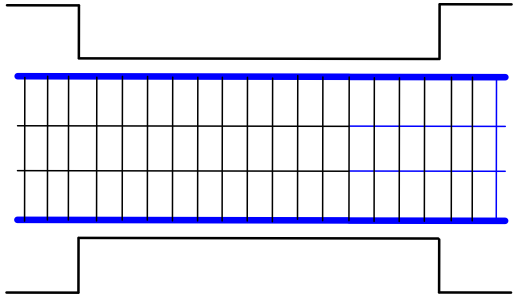
CB20-1



$$f_c' = 52.1\ MPa$$
$$\rho_f = 2.39\%$$

試驗結果

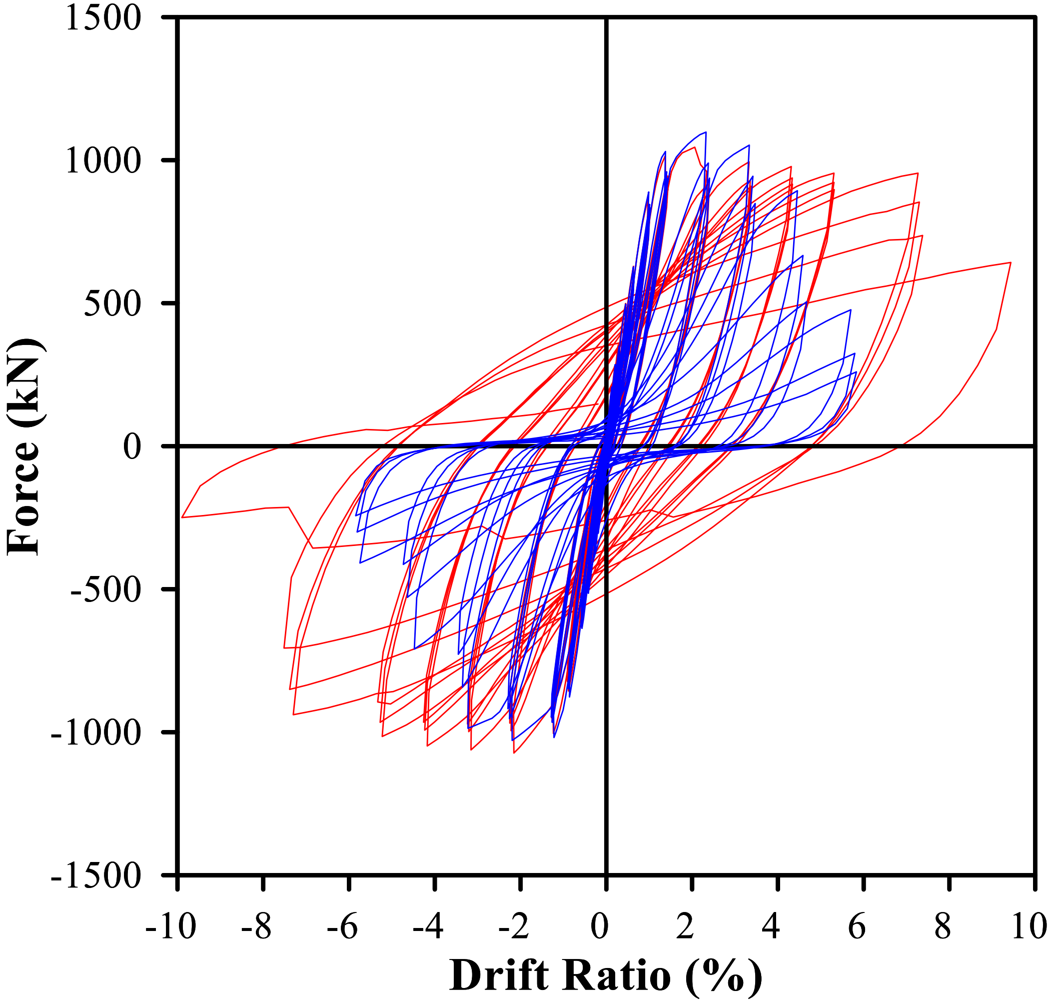
CB20-2



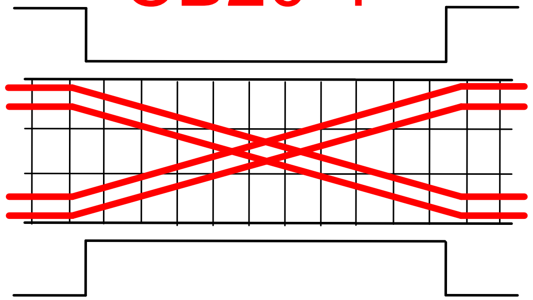
5.7%

$$\upsilon = 1.2\sqrt{f_c'}$$

UDR = 4.4%



CB20-1



7.3%

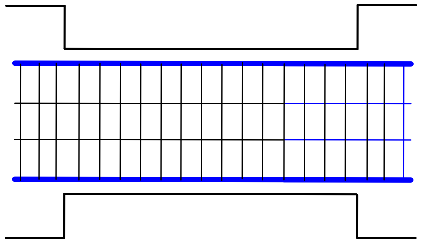
$$\upsilon = 1.3\sqrt{f_c'}$$

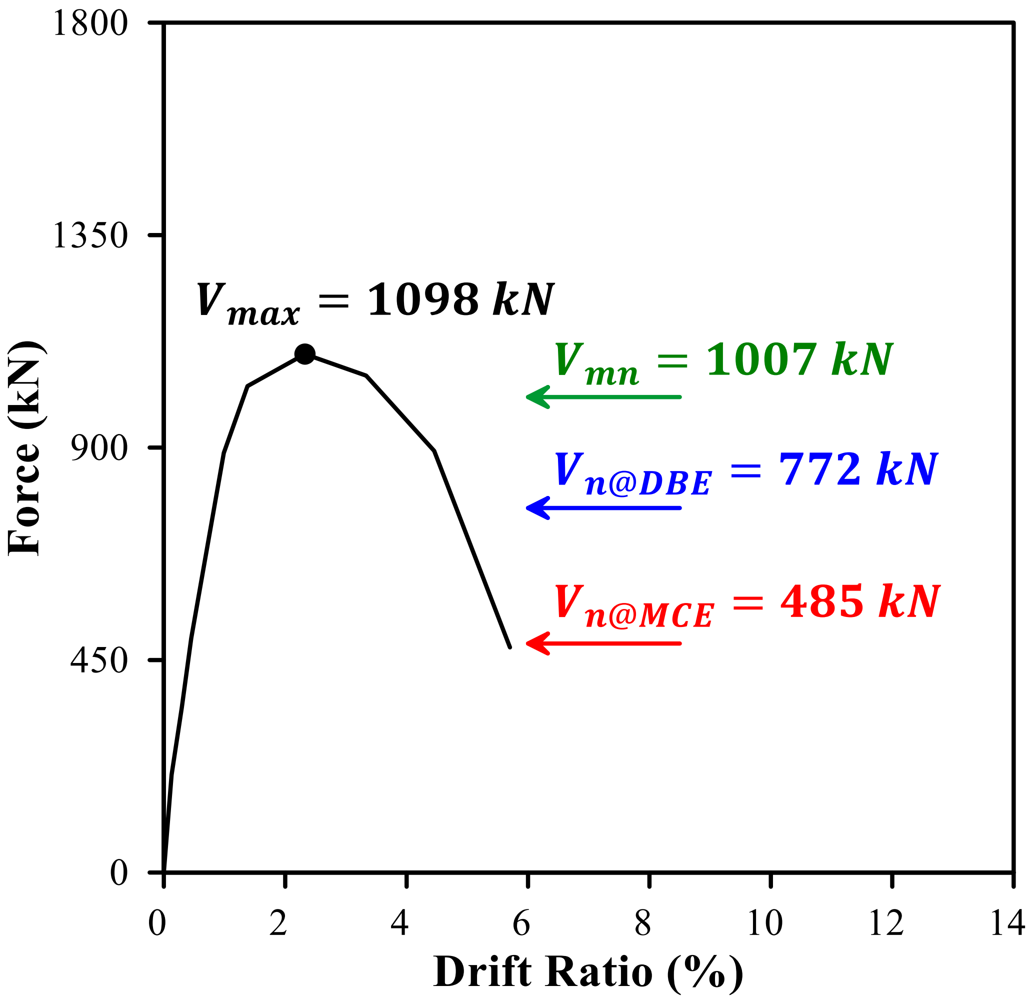
UDR = 7.2%

$\ell_n / h = 2.0$

撓曲與剪力強度評估

CB20-2

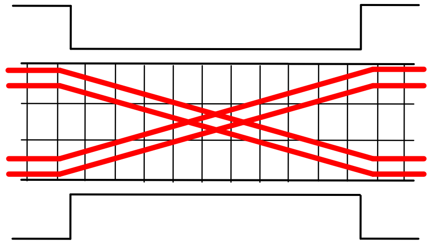


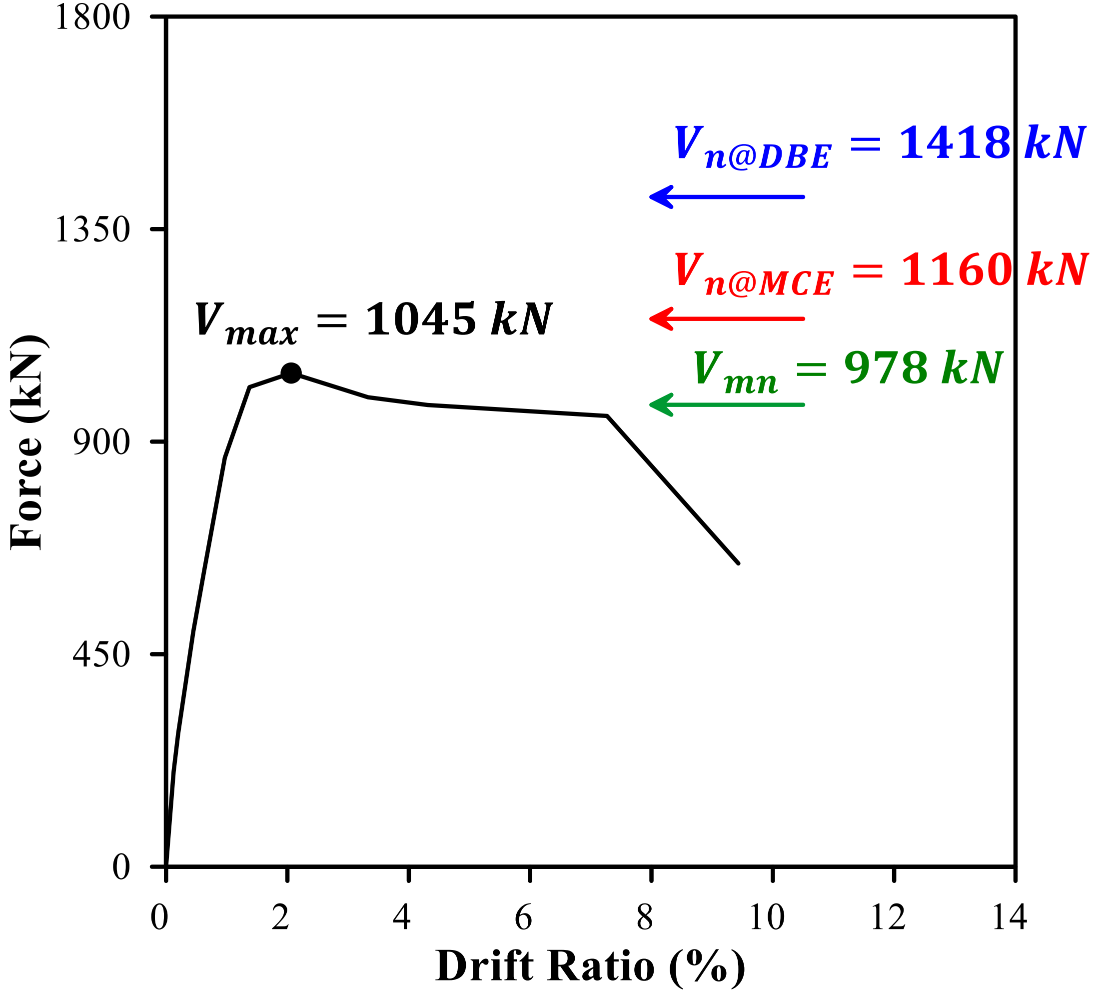


1. $V_{n@DBE} < V_{mn}$
2. 剪力強度主控行為
3. $\frac{V_{max}}{V_{n@DBE}} = 1.42$
4. $V_{n@DBE}$以D-區域評估與DBD-區域評估會有不同，可詳中短連接梁講義

$\ell_n / h = 2.0$

CB20-1





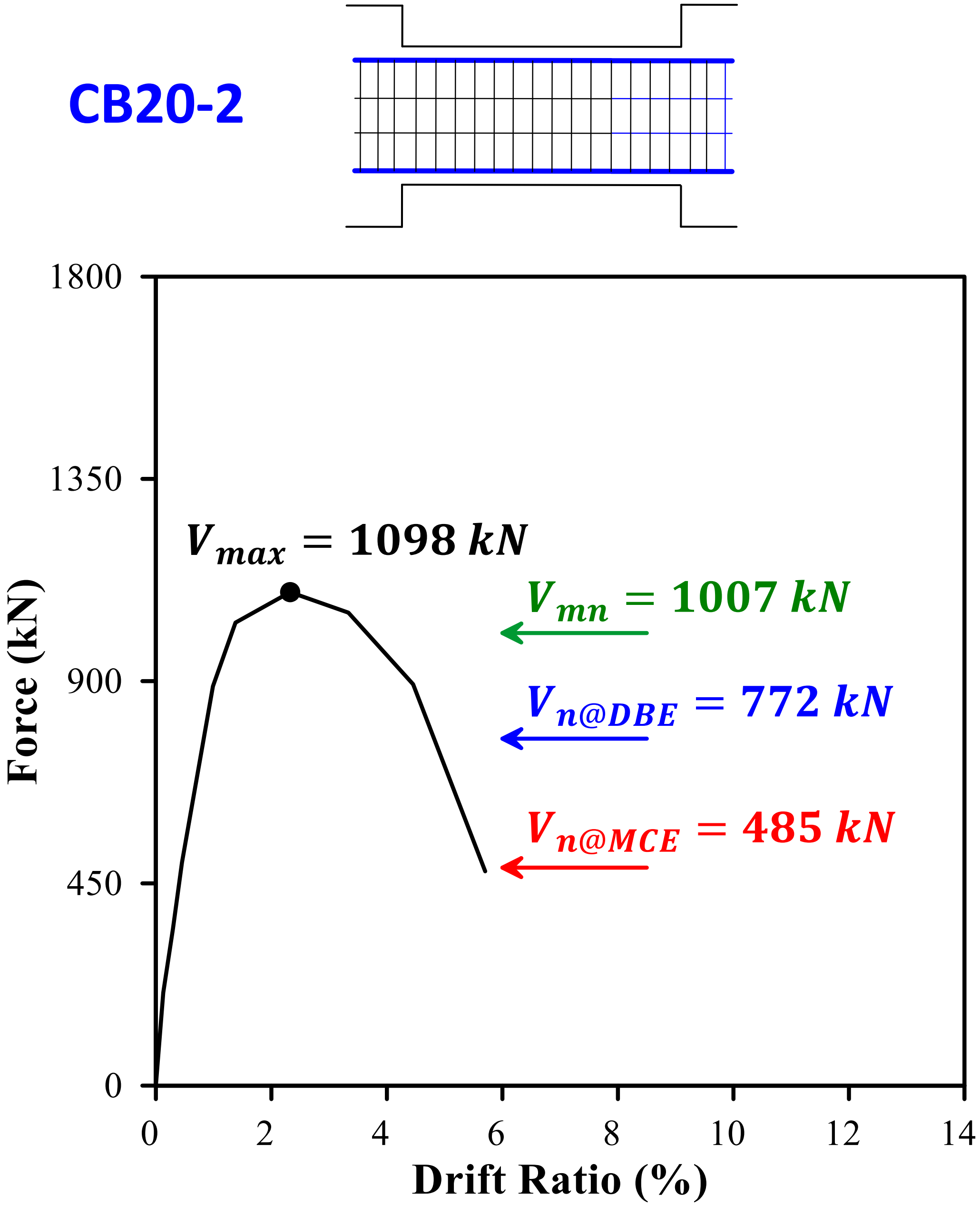
撓曲與剪力強度評估

1. $V_{mn} < V_{n@DBE}$
2. 撓曲強度主控行為
3. $\frac{V_{max}}{V_{mn}} = 1.07$
4. $V_{n@MCE} > V_{max}$
5. 撓曲塑鉸變形充分發展
6. $\frac{V_{n,@MCE}}{V_{max}} = 1.11$

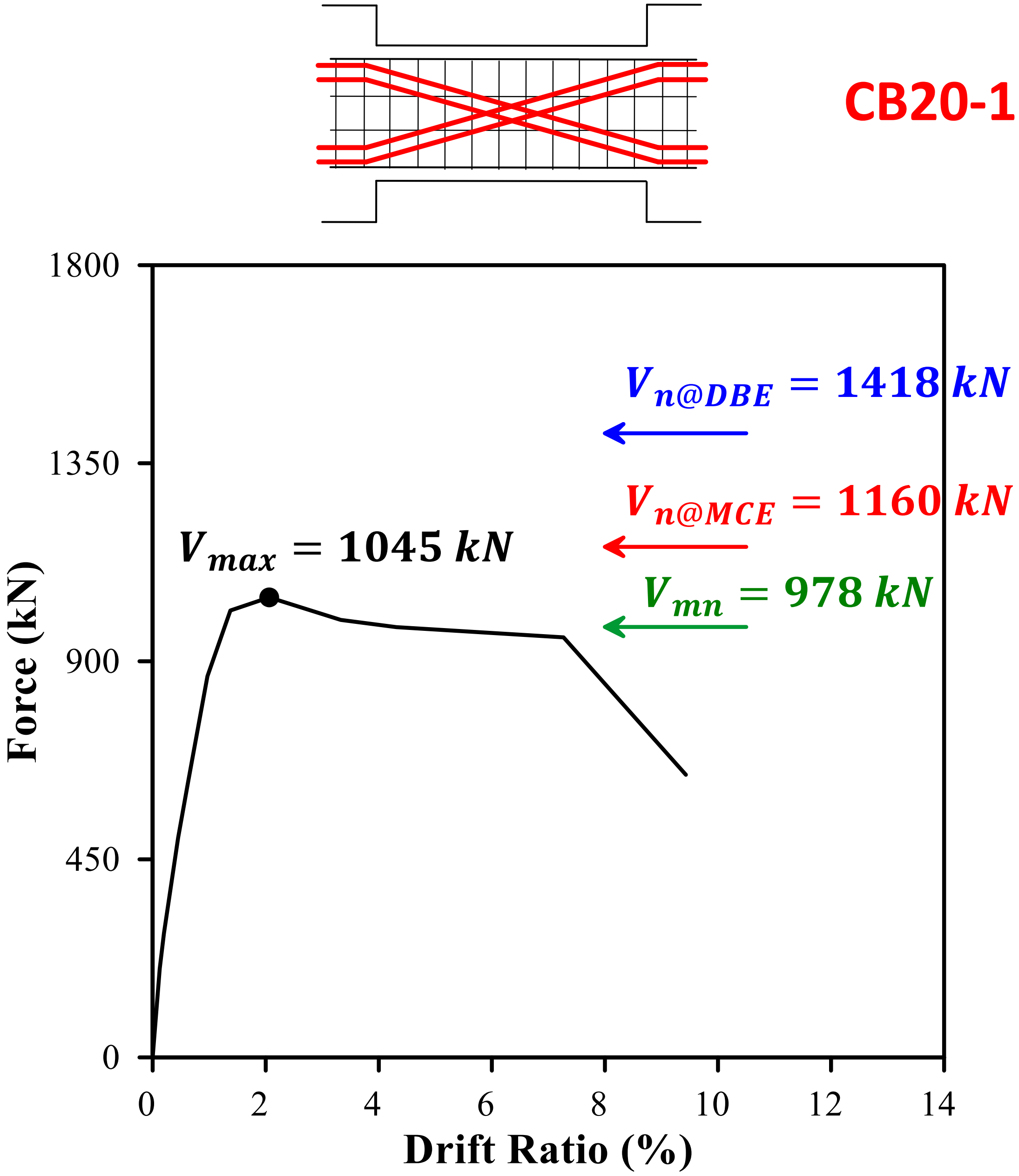
$\ell_n / h = 2.0$

測試行為評估

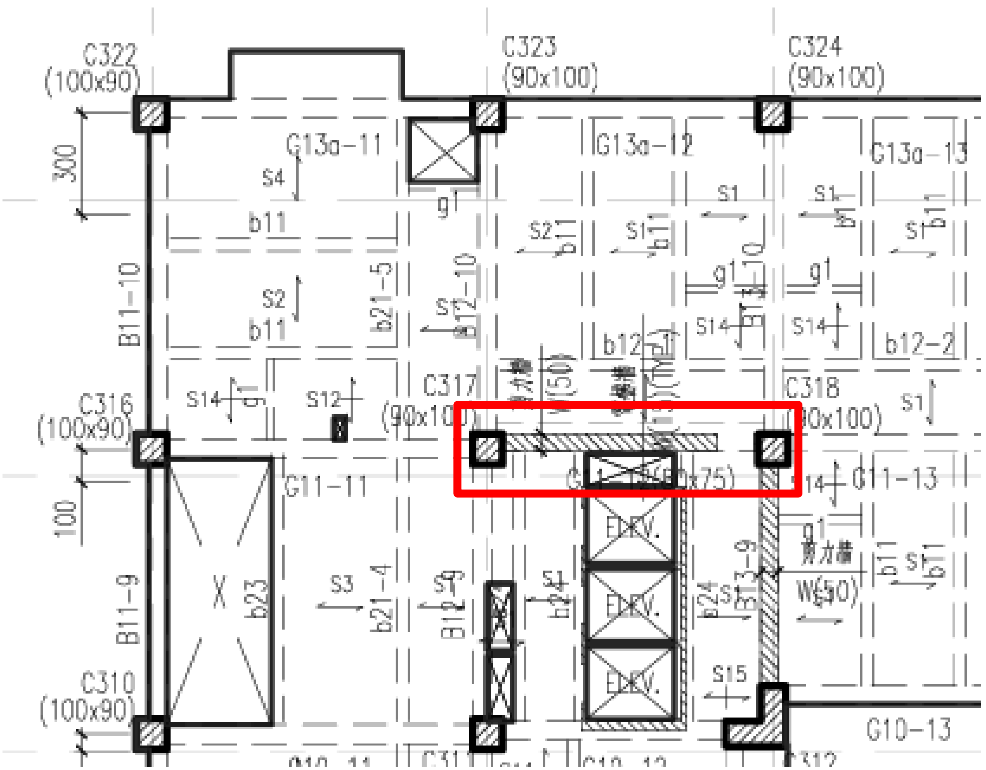
CB20-2



CB20-1



非結構RC牆所造成深梁之剪力強度設計



- 中小度地震剪力強度設計

彈性壓力區深度 $kd = \left(\sqrt{(\rho_f n)^2 + 2\rho_f n} - \rho_f n\right) d$

$$V_n = V_{n,c} + (1.0 + 1.0) A_{vd} f_{yd} \sin\alpha$$

$$\phi V_n \geq V_u$$

- 增加梁之剪力強度與轉動能力

壓力區深度 $c = \dfrac{f_{yd} \times A_{vd} \cos\alpha}{0.85 f'_c \times b_w}$　　塑鉸少量轉動

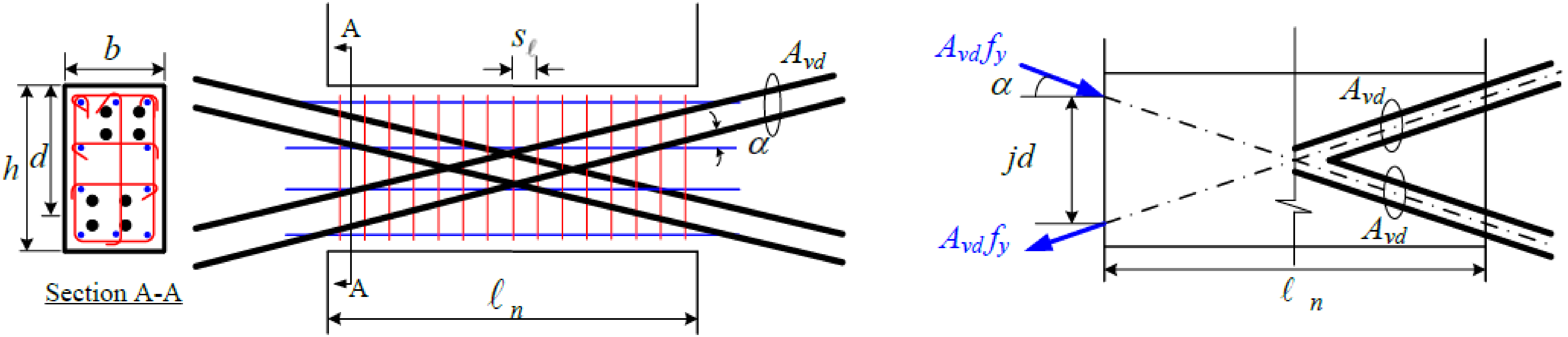
$$V_n = V_{n,c} + (1.0 + 1.0) A_{vd} f_{yd} \sin\alpha$$

$$\phi V_n \geq \frac{2 \times M_n}{\ell_n}$$

ACI 318-25 規範設計之檢討

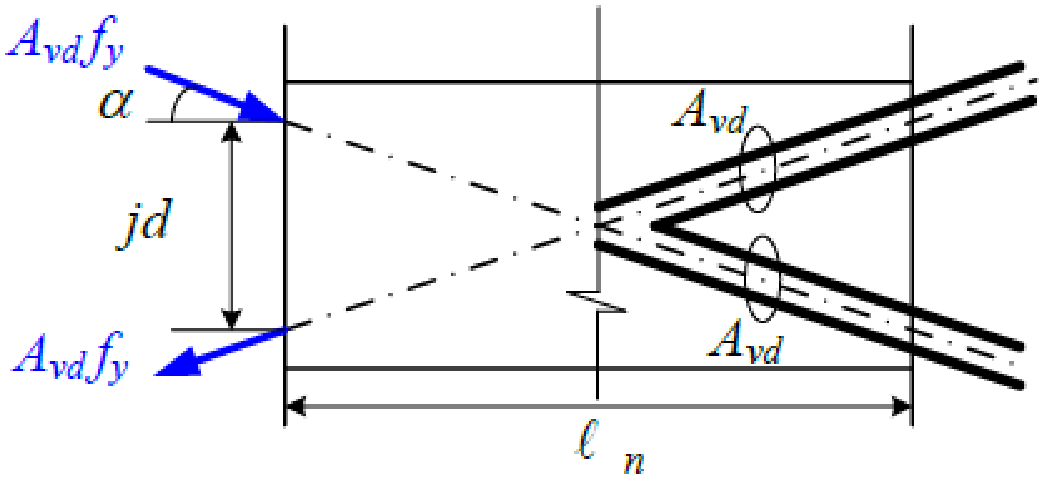
強度預測之缺陷

配置對角鋼筋連接梁之示意圖



a) Diagonally reinforced coupling beam b) Strength contributed by the diagonal reinforcement

ACI 318-25 規範之剪力強度設計



$$V_{n,ACI} = 2A_{vd} f_y \sin\alpha \geq \frac{V_u}{\phi}$$

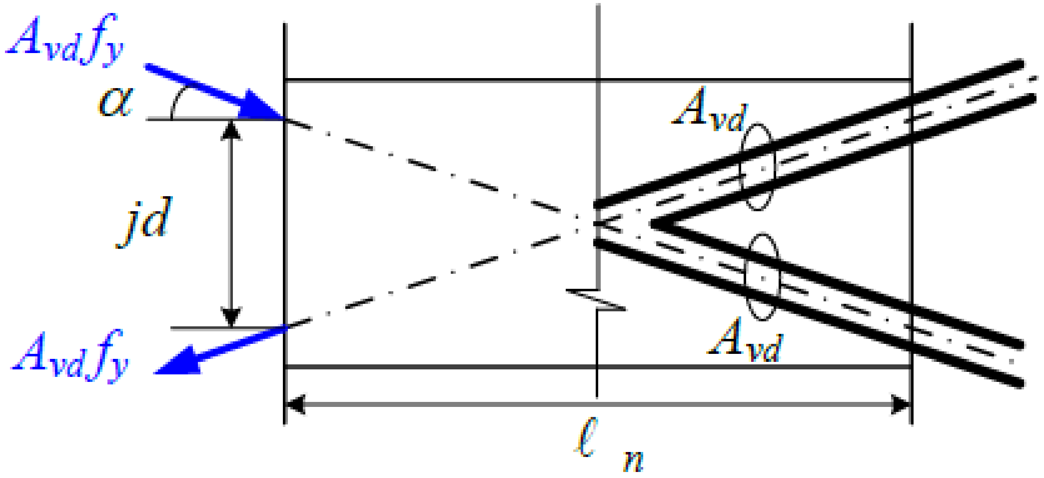
$$V_{n,ACI} \leq 0.83\sqrt{f_c'}bd \quad (MPa)$$

- 以DBE剪力設計需求配置對角鋼筋
- 缺陷1：忽略混凝土剪力強度之貢獻，引致超出預期之作用剪力
- 缺陷2：混凝土已無擠碎之虞，最大剪力限制應無必要

ACI 318-25 規範之撓曲強度設計

$$M_{n,ACI} = A_{vd} f_y \cos\alpha \, jd$$

$$\frac{jd}{\ell_n} = \tan\alpha = \frac{\sin\alpha}{\cos\alpha}$$

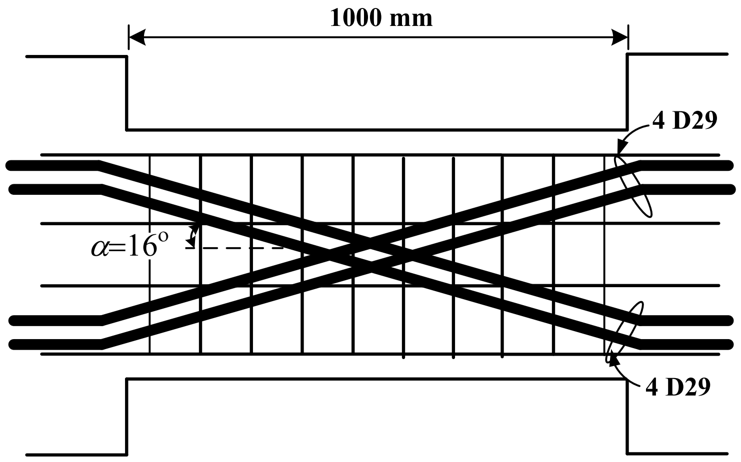


$$V_{n,ACI} = \frac{2 \times M_{n,ACI}}{\ell_n} = 2A_{vd} f_y \cos\alpha \times \frac{jd}{\ell_n} = 2A_{vd} f_y \sin\alpha$$

- 對角鋼筋足以發展所需之彎矩強度，無需查驗
- 缺陷：忽略混凝土彎矩強度之貢獻，引致超出預期之彎矩強度，因而產生較大之撓曲塑鉸作用剪力

未考慮混凝土貢獻之強度預測

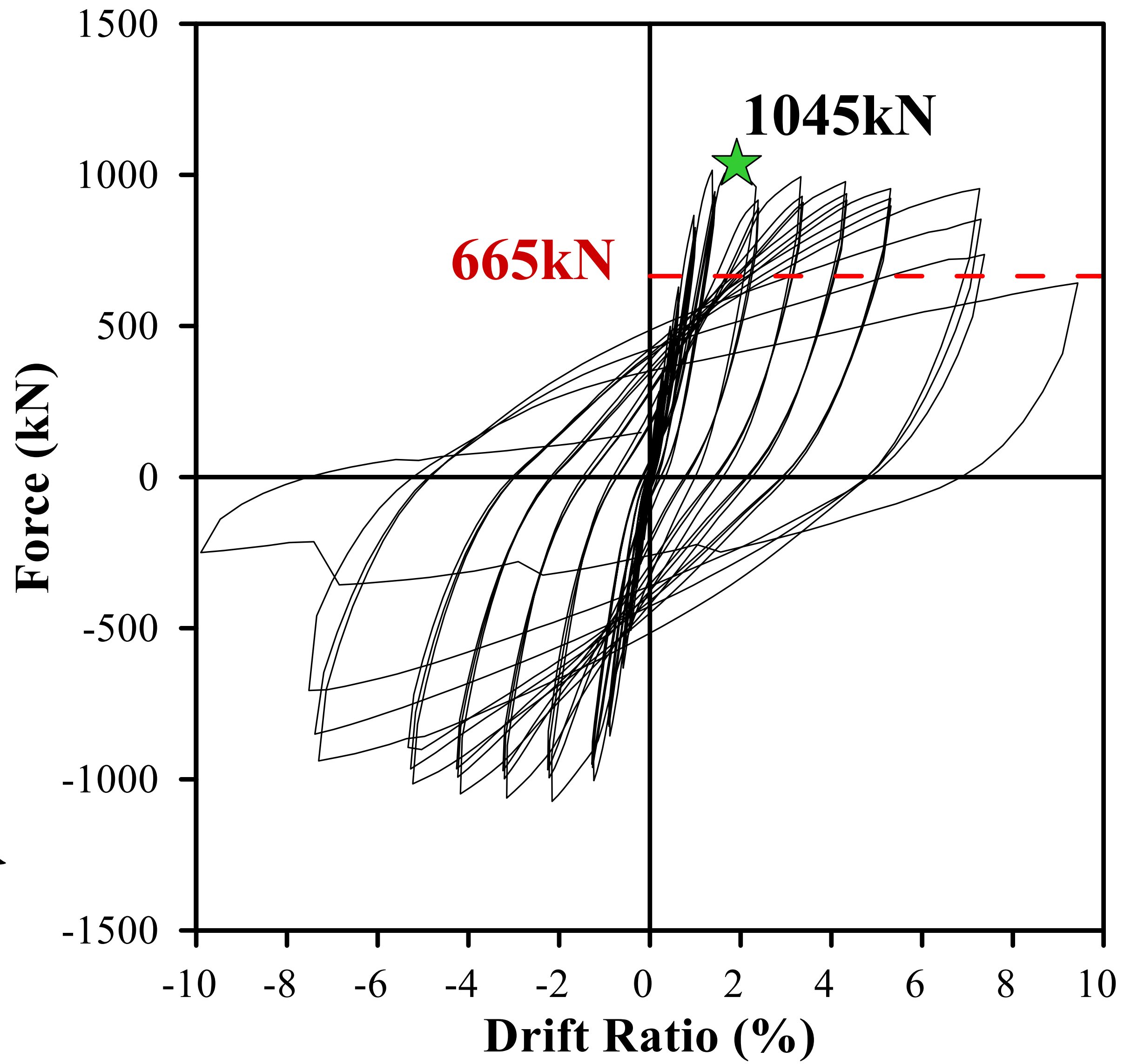
CB20-1



$$V_n = 2A_{vd} f_y \sin\alpha = 665kN$$

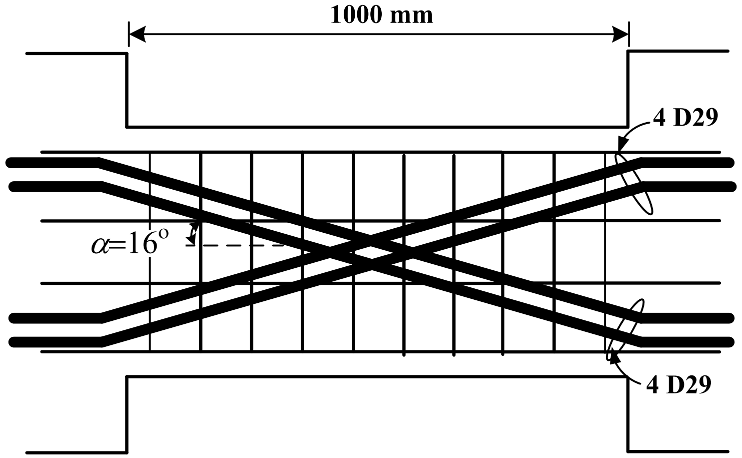
註：

1. ACI預測公式忽略混凝土強度貢獻
2. 引致超出預期之作用剪力

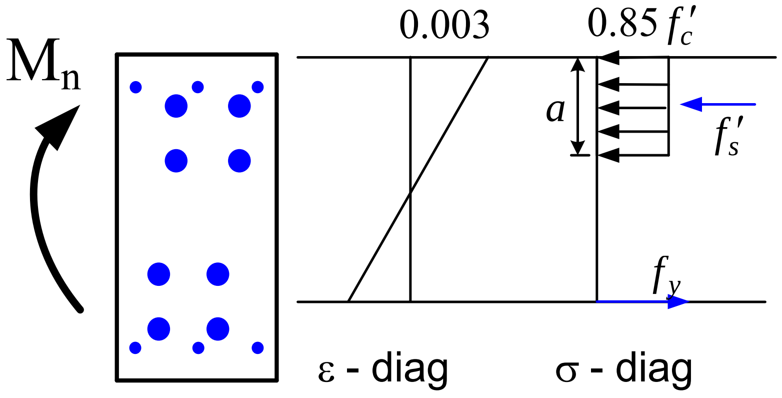


考慮混凝土貢獻之強度預測

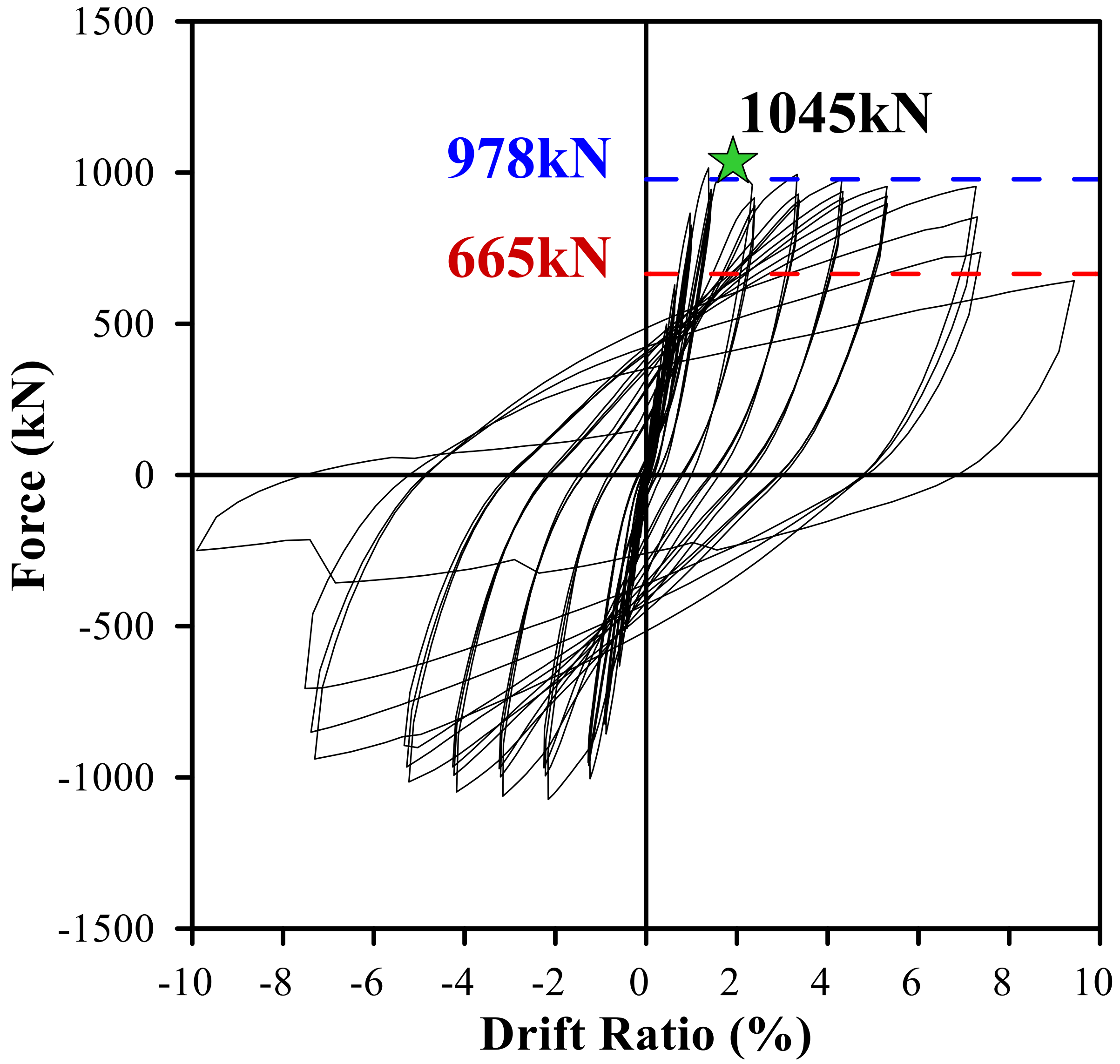
CB20-1



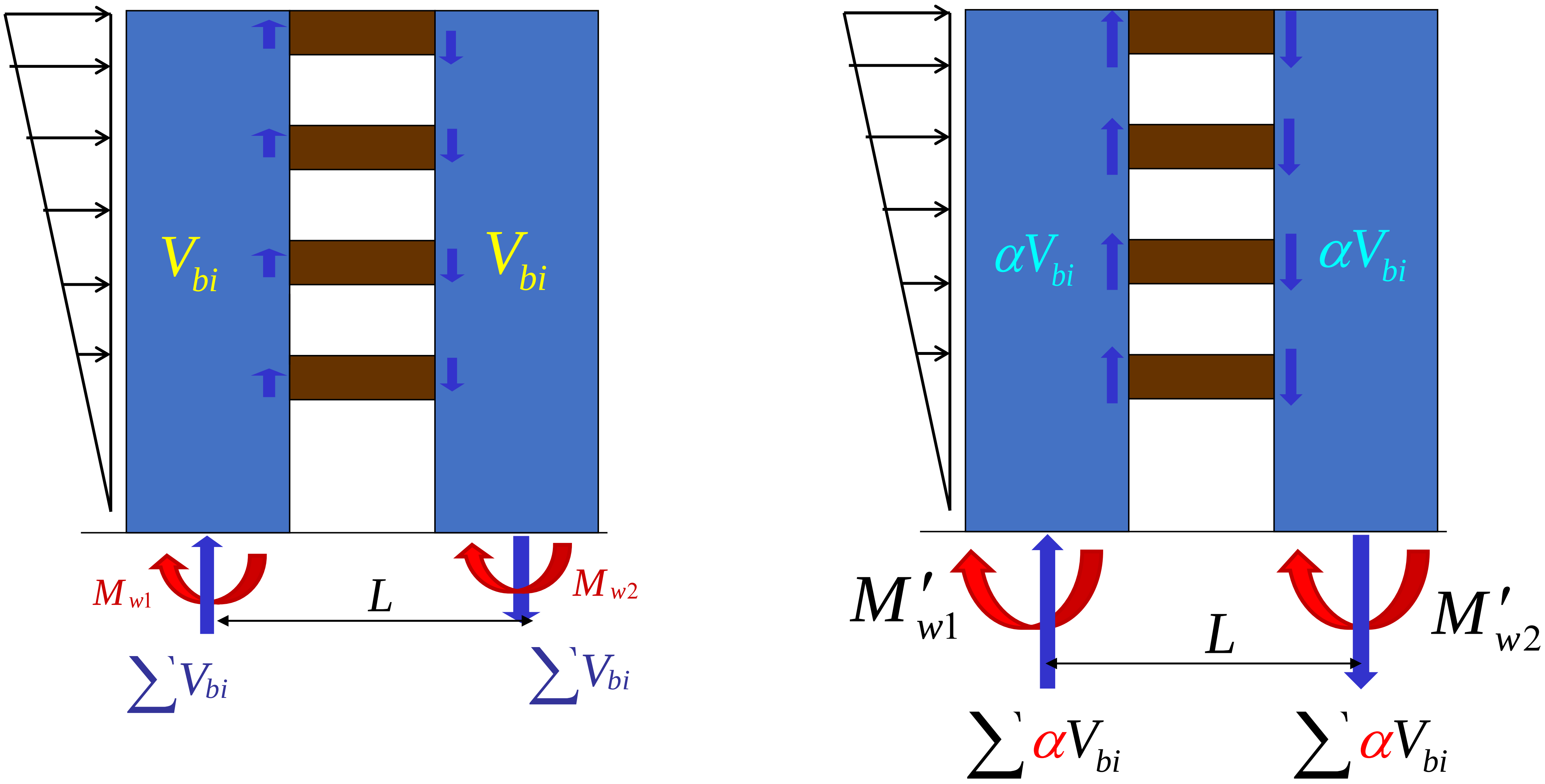
$$V_n = 2A_{vd}\, f_y \sin\alpha = 665kN$$



$$V_{mn} = \frac{2M_n}{\ell_n} = 978kN$$



超出預期之作用力易引致非預期之破壞



超量之對角鋼筋配置

Figure 1 Showing the intricate, labor-intensive steel reinforcing bar congestion in a typical high-rise coupling beam. Photo credit: Remy Lequesne, The University of Michigan

過多之對角鋼筋量 → 鋼筋壅塞 → 施工困難

ACI 318-25 強度設計之缺陷

1. 忽略混凝土強度貢獻，提供超量之對角鋼筋量

→ 引致較大之作用力，可能產生非預期之破壞

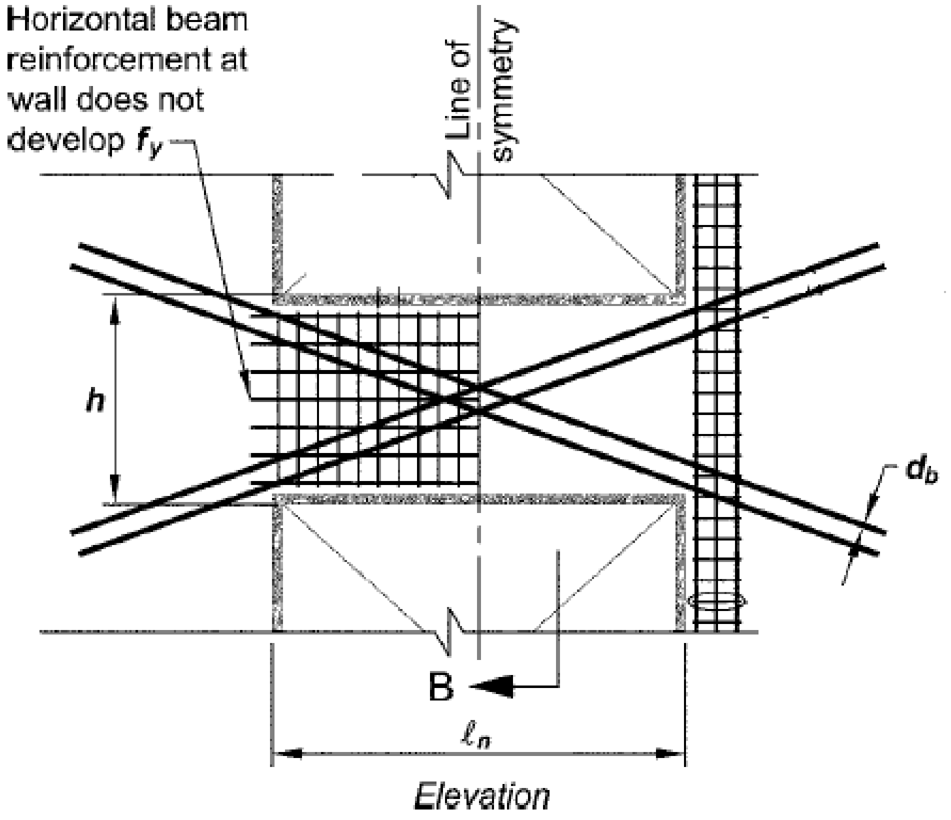
→ 造成鋼筋壅塞，施工困難

2. 足夠之對角鋼筋量，混凝土已無擠碎破壞之疑慮，

剪力強度上限應無必要，~~$V_n \le 0.83\sqrt{f_c'}bd$~~

鋼筋配置之改善

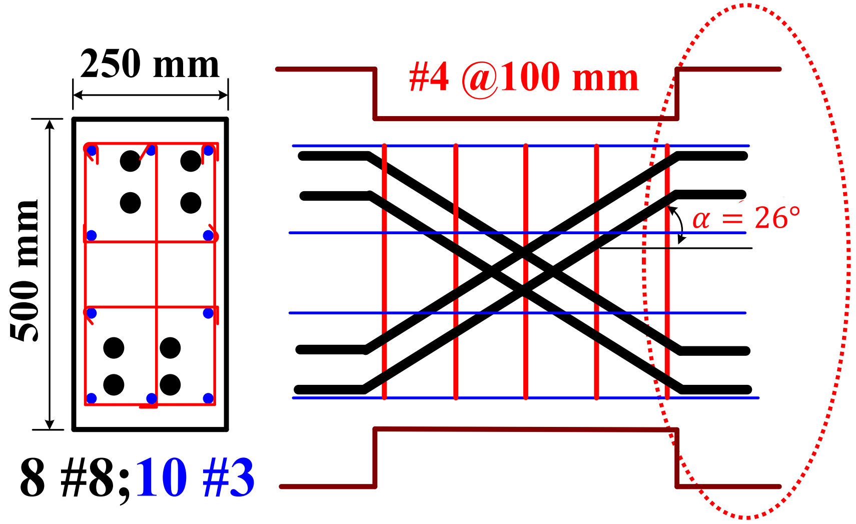
ACI 斜向對角鋼筋之配置



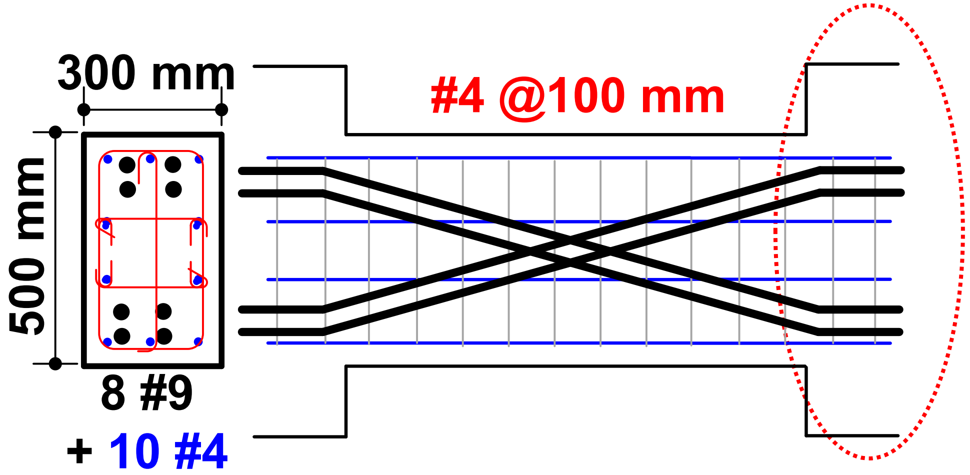
- 對角鋼筋採斜向配置於柱及剪力牆邊界構材內
- 柱及邊界構材內鋼筋壅塞，對角鋼筋綁紮困難

對角鋼筋配置之改善建議

CB10-1

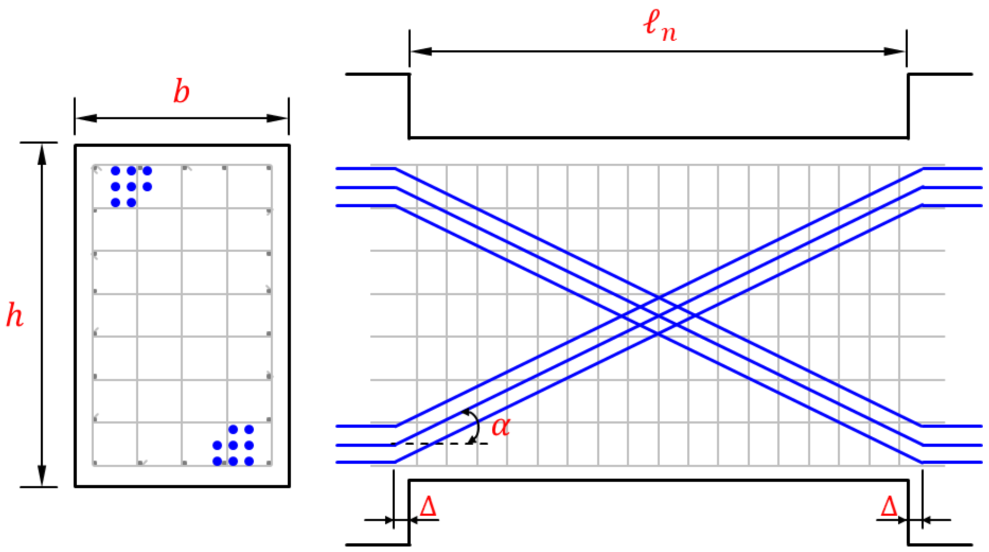


CB20-1

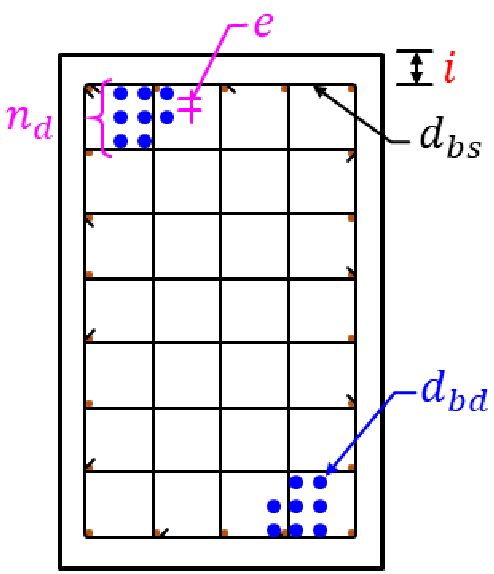


- 對角鋼筋於圍束核心內作偏折處理，採水平向配置於柱及剪力牆邊界構材內
- 對角鋼筋於柱及邊界構材內之鋼筋綁紮較為容易，適合預鑄組件施作

深短連接梁對角鋼筋夾角α計算公式



$$\alpha = \tan^{-1}\left(\frac{h-2\left(i+d_{bs}+d_{bd}\frac{n_d}{2}+e\frac{(n_d-1)}{2}\right)}{\ell_n+2\Delta}\right)$$



Δ：對角鋼筋延伸進牆體長度的水平分量，至少取5公分

i：混凝土保護層厚度

d_{bs}：箍筋直徑

d_{bd}：對角鋼筋直徑

n_d：對角鋼筋層數

$e = \max(2.5\,, d_{bd})$：對角鋼筋層與層之排置間距

測試資料庫之探討

深短連接梁之測試資料庫(節錄版)

No.	Author	Spec. ID	Beam Segment		Diagonal Reinforcement			Longitudinal Reinforcement		Transverse Reinforcement		V_{test} (kN)	UDR (%)
			$\frac{\ell_n}{h}$	f_c' (MPa)	α (°)	f_{yd} (MPa)	Detailing	f_{yl} (MPa)	ρ_ℓ (%)	f_{yt} (MPa)	$\rho_{t,y}$ (%)		
1	Weber-Kamin *et al.*	D80-1.5	1.5	52.4	23	573	12-#6	613	0.50	613	1.00	1133	6.4
2		D100-1.5	1.5	56.5	23	744	10-#6	613	0.50	613	1.00	1136	4.6
3	Ameen	CB1	1.9	41.0	18	434	12-#7	476	0.49	469	0.73	819	6.8
4		CB2	1.9	50.0	18	883	8-#6	476	0.49	469	0.73	925	5.1
5*		CB2D	1.9	43.0	18	883	8-#6	476	0.49	469	0.73	908	5.3
6*		CB3D	1.9	43.0	18	883	12-#6	476	0.49	469	0.73	1226	4.6
7*	Lim et *al.*	CB10-1	1.0	33.7	26	486	8-#8	440	0.57	468	1.52	1444	5.7
8*		CB20-1	2.0	52.1	16	466	8-#9	502	0.85	502	1.27	1045	7.2
9	Cheng *et al.*	L0	2.0	34.6	19	455	8-#8	539	0.53	539	0.95	882	6.8
10		H0	2.0	31.4	16	455	16-#8	539	0.53	539	0.95	1549	5.8

註：*縱向錨定鋼筋端部具有足夠錨定之試體

Tsai, R. J., Henry, R, S., Elwood, K. J., Hwang, S. J., and Cheng, M. Y. (2026). "Dataset of diagonally reinforced coupling beams: Axial restraint and strength design," Engineering Structures 349, 121759.

初始文獻

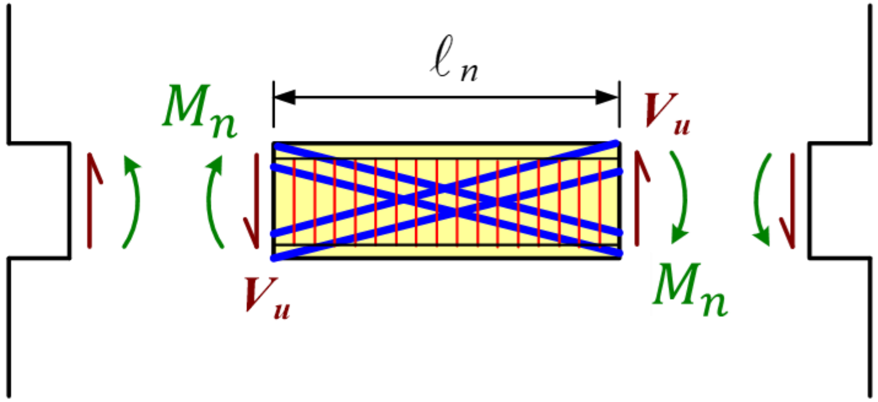
- Weber-Kamin A, Lepage A, Lequesne R. Reinforced concrete coupling beams with high-strength steel bars. PRJ-2876, DesignSafe-CI 2020. https://doi.org/ 10.17603/ds2-k0rq-s232.
- Ameen S, Lequesne RD, Lepage A. Diagonally reinforced concrete coupling beams with Grade 120(830) High-Strength steel bars. ACI Struct J 2020;117(6):199–210. https://doi.org/10.14359/51728067.
- Lim E, Hwang S-J, Wang T-W, Chang Y-H. An investigation on the seismic behavior of deep reinforced concrete coupling beams. ACI Struct J 2016;113(2):217–26. https://doi.org/10.14359/51687939.
- Cheng, M.-Y., R.-J. Tsai, J.-C. Hung, and R. S. Henry. Cyclic responses of axially restrained diagonally reinforced coupling beams. ACI Struct J 2025;122 (3):105–117.

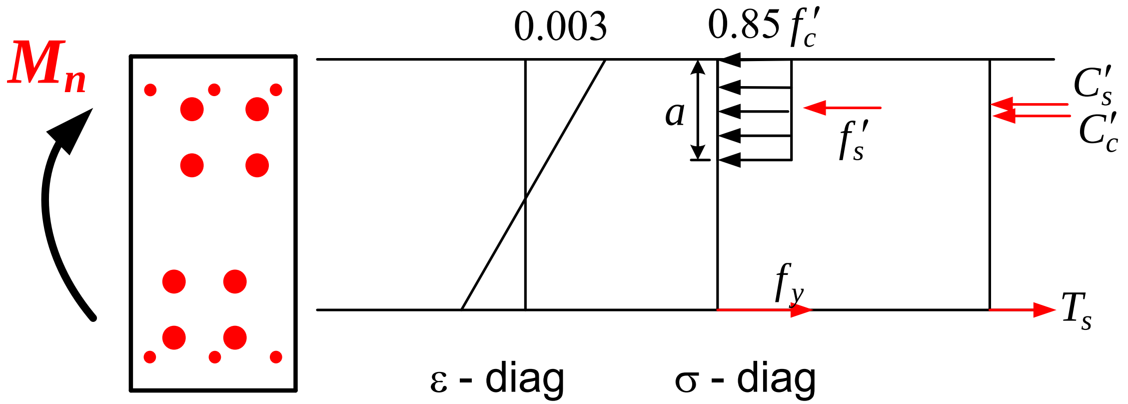
測試試體之耐震行為評估

撓曲強度之計算流程

撓曲強度:

由 XTRACT 取得斷面標稱彎矩 $M_n \Rightarrow V_{mn} = \dfrac{2M_n}{\ell_n}$

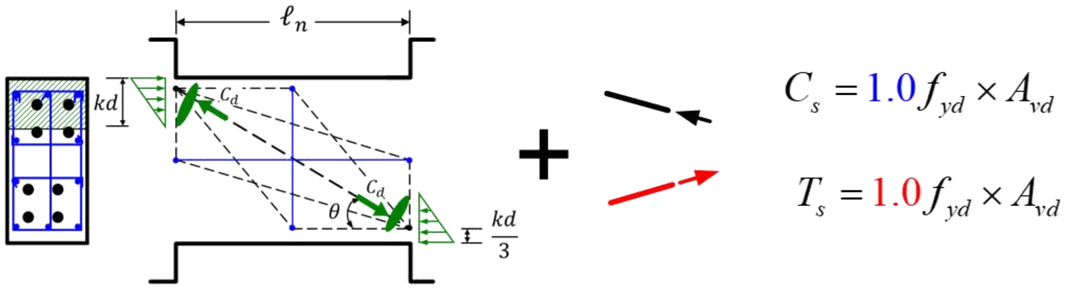




- 斷面分析 - XTRACT
- 平面維持平面

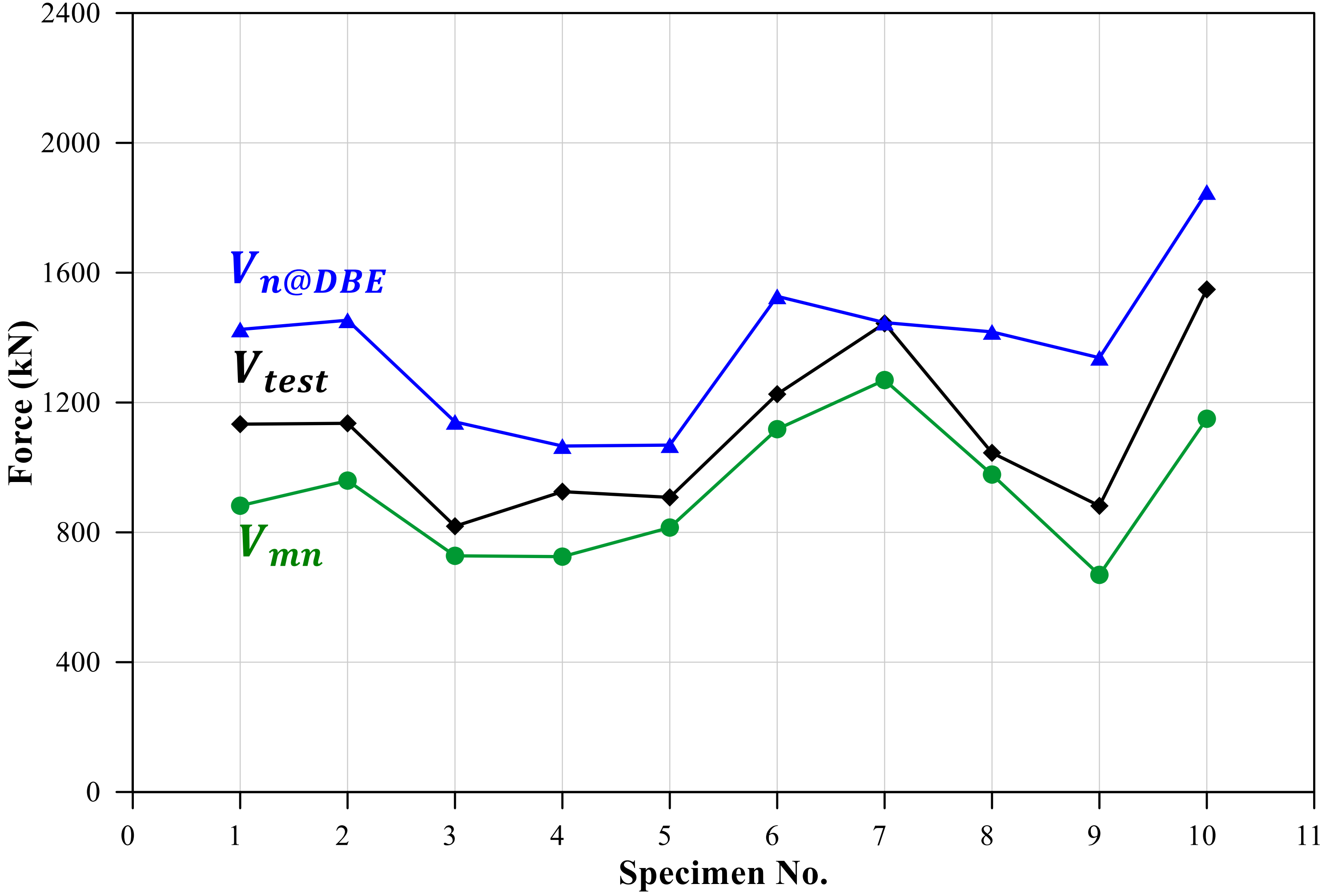
DBE剪力強度之計算流程

Strength (DBE):



1. 彈性壓力區深度 $kd = \left(\sqrt{(\rho_f n)^2 + 2\rho_f n} - \rho_f n\right) d$
2. 對角壓桿之有效面積 $A_{str} = b_w \times kd$
3. 對角壓桿之水平仰角 $\theta = \tan^{-1}\left(\frac{h - 2\times kd/3}{\ell_n}\right)$
4. 壓拉桿指標 $K = \tan^A \theta + \cot^A \theta - 1 + 0.14B \leq 1.64$
5. 開裂鋼筋混凝土軟化係數 $\zeta = \frac{3.35}{\sqrt{f_c'}} \leq 0.52$
6. 對角壓桿之垂直剪力強度 $C_d \sin\theta = K\zeta f_c' A_{str} \sin\theta$
7. 計算DBE需求下之剪力強度 $V_{n@DBE} = C_d \sin\theta + (1.0 + 1.0) A_{vd} f_{yd} \sin\alpha$

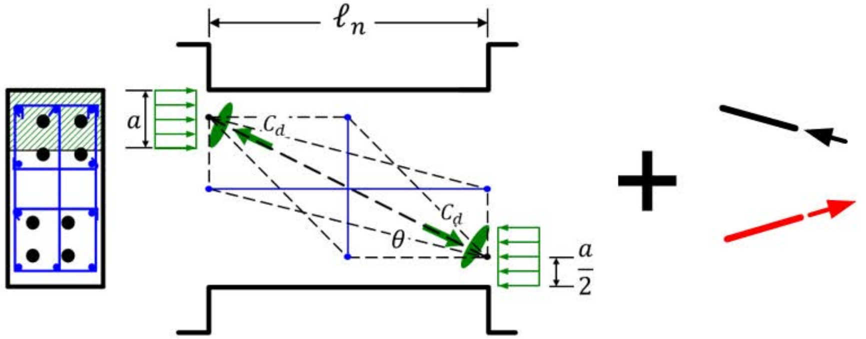
DBE強度行為評估



1. $V_{mn} < V_{n@DBE}$
2. 撓曲強度主控行為
3. $V_{test} > V_{mn}$
4. 撓曲塑鉸變形可以發展

MCE剪力強度之計算流程

Strength (MCE):

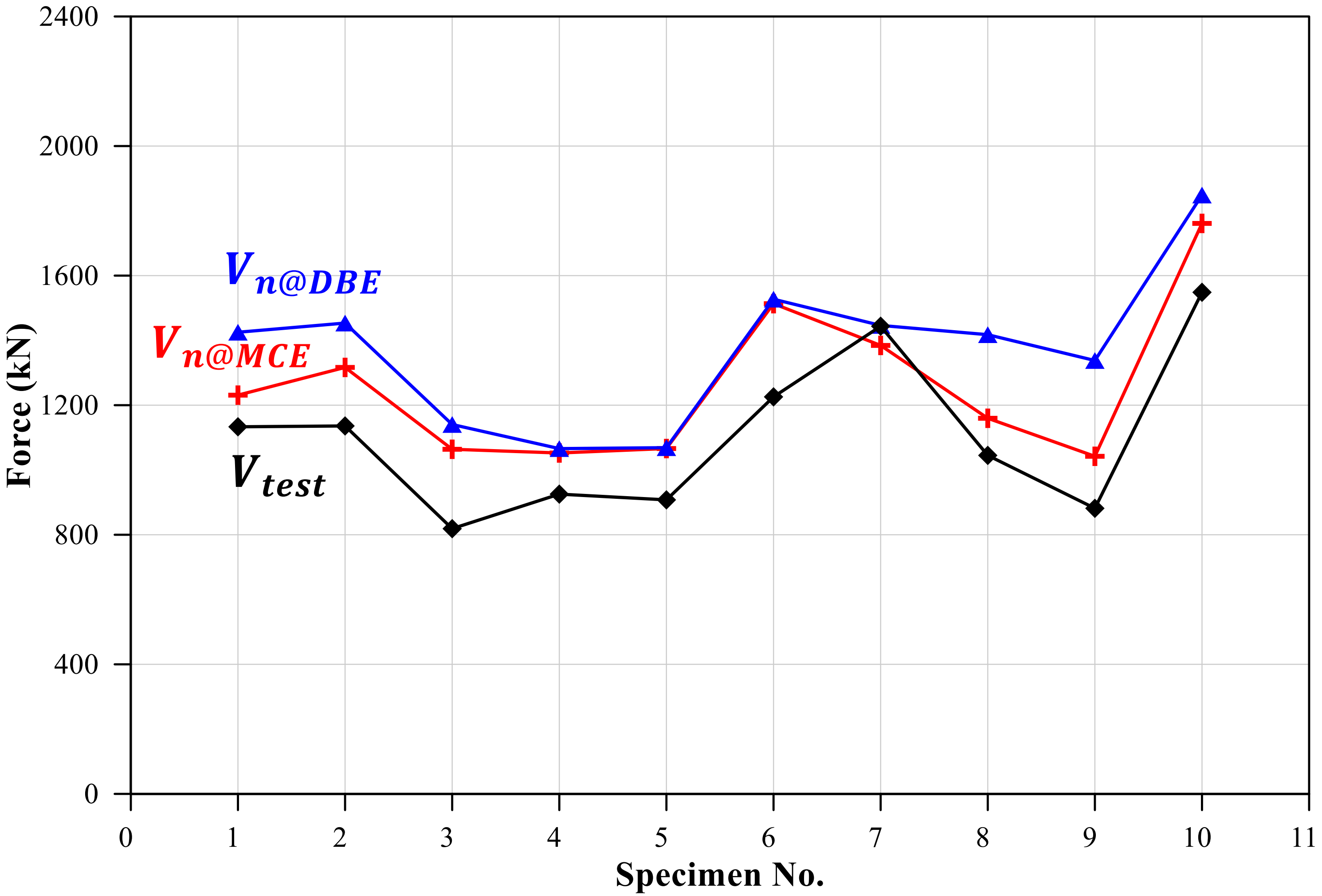


$C_s = 1.25 f_{yd} \times A_{vd}$

$T_s = 1.25 f_{yd} \times A_{vd}$

1. 壓力區深度 $a = \frac{1.25 f_{yd} \times A_{vd} \cos\alpha}{0.85 f_c' \times b_w}$
2. 對角壓桿之有效面積 $A_{str} = b_w \times a$
3. 對角壓桿之水平仰角 $\theta = \tan^{-1}\left(\frac{h - 2\times a/2}{\ell_n}\right)$
4. 壓拉桿指標 $K = \tan^A\theta + \cot^A\theta - 1 + 0.14B \leq 1.64$
5. 開裂鋼筋混凝土軟化係數 $\zeta = \frac{3.35}{\sqrt{f_c'}} \leq 0.52$
6. 對角壓桿之垂直剪力強度 $C_d \sin\theta = K\zeta f_c' A_{str} \sin\theta$
7. 計算MCE需求下之剪力強度 $V_{n@MCE} = C_d \sin\theta + (1.25 + 1.25) A_{vd} f_{yd} \sin\alpha$

MCE強度行為評估



1. $V_{n@MCE} > V_{test}$
2. 撓曲塑鉸變形充分發展
3. $V_{n@MCE} < V_{n@DBE}$
4. 剪力強度有劣化現象

資料庫試體之耐震行為數據

No.	Author	Spec. ID	V_{test} (kN)	UDR (%)	$\frac{V_{test}}{V_{mn}}$	$\frac{V_{test}}{V_{n@DBE}}$	$\frac{V_{test}}{V_{n@MCE}}$
1	Weber-Kamin *et al.*	D80-1.5	1133	6.4	1.28	0.80	0.92
2		D100-1.5	1136	4.6	1.18	0.78	0.86
3	Ameen	CB1	819	6.8	1.13	0.72	0.77
4		CB2	925	5.1	1.28	0.87	0.88
5*		CB2D	908	5.3	1.11	0.85	0.85
6*		CB3D	1226	4.6	1.10	0.80	0.81
7*	Lim et *al.*	CB10-1	1444	5.7	1.14	1.00	1.04
8*		CB20-1	1045	7.2	1.07	0.74	0.90
9	Cheng *et al.*	L0	882	6.8	1.32	0.66	0.85
10		H0	1549	5.8	1.35	0.84	0.88
				AVG	**1.21**	**0.80**	**0.89**
				COV	**0.09**	**0.11**	**0.09**

$$\frac{V_{test}}{V_{mn}} > 1$$

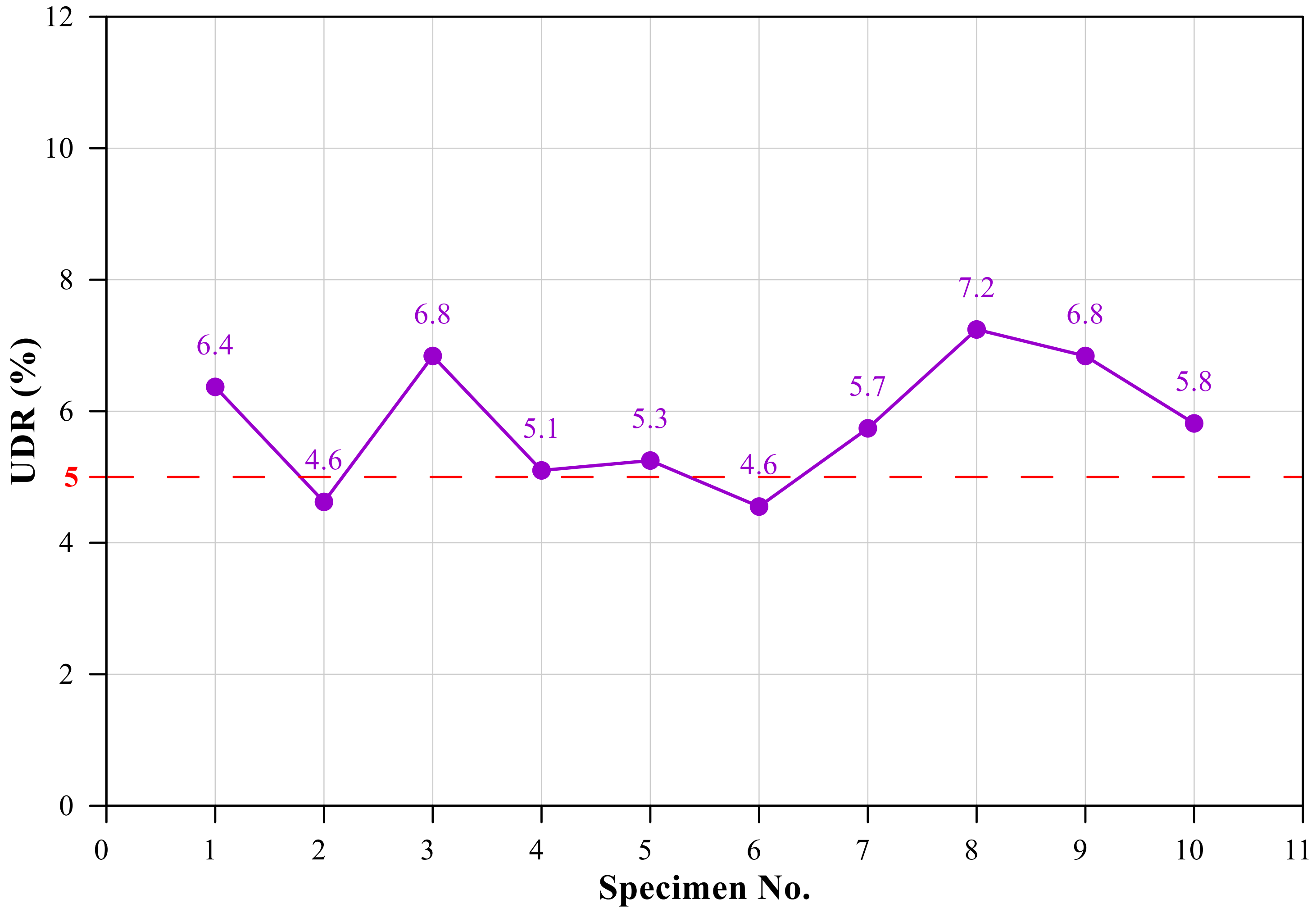
撓曲行為主控

$$\frac{V_{test}}{V_{n@MCE}} < 1$$

剪力強度充分支持撓曲塑鉸發展

註：*縱向錨定鋼筋端部具有足夠錨定之試體

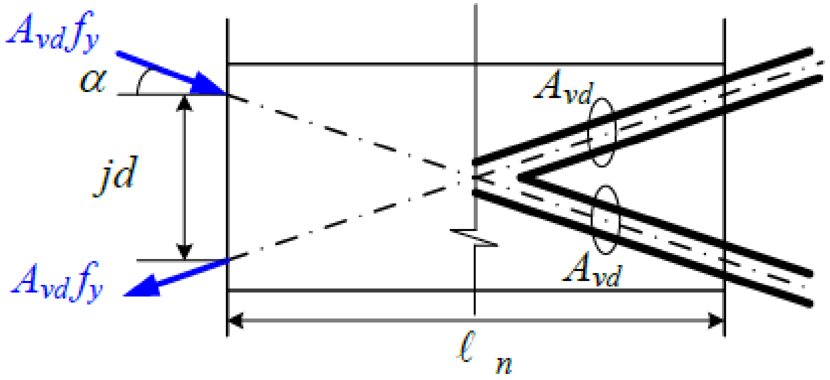
撓曲塑鉸變形能力評估



1. 極限變位角 (UDR)>≈5%
2. 撓曲塑鉸變形充分發展

塑鉸剪力強度驗證之必要性

軟化壓拉桿模型之參數化研究



假設：

1. **充足的剪力鋼筋配置** $\therefore K \rightarrow K_{max}$

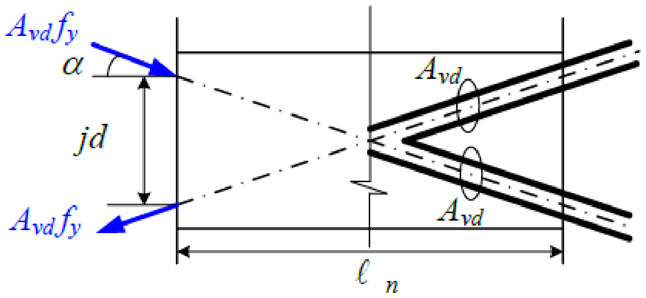
2. **對角壓桿之水平仰角 $\boldsymbol{\theta}$ 與對角鋼筋之水平仰角 $\boldsymbol{\alpha}$ 相等**

$$\theta = \tan^{-1}\left(\frac{h-2\times a/2}{\ell_n}\right) \quad \therefore assumed\ \theta = \alpha$$

3. **對於 $\frac{\ell_n}{h} \leq 2$ 連接梁構件，在MCE需求下具較大的塑性轉角(Plastic Rotation)**

$$\therefore f_s = f_s' = 1.25 f_y$$

參數化研究



1. Assumed $jd \approx (d - a/2)$ and $\alpha = \theta$
2. $tan\alpha = \frac{sin\alpha}{cos\alpha} = \frac{jd}{l_n}$
3. $C = T; 0.85f_c'b_w a = 1.25f_y A_{vd} cos\alpha$
4. $A_{str} = b_w \times a$

Supply: Assumed $f_s = f_s' = 1.25f_y$

$$V_{n@MCE} = K\zeta f_c' A_{str} \sin\alpha + (1.25 + 1.25)A_{vd} f_y \sin\alpha$$

$$\Rightarrow V_{n@MCE} = K\zeta f_c' b_w \frac{1.25 f_y A_{vd} \cos\alpha}{0.85 f_c' b_w} \sin\alpha + 2A_{vd}(1.25f_y)\sin\alpha$$

Demand:

$$V_p = \frac{2M_{pr}}{\ell_n} = \frac{2}{\ell_n} A_{vd} \cos\alpha\,(1.25f_y) \times jd = 2A_{vd}\cos\alpha\,(1.25f_y) \times \frac{\sin\alpha}{\cos\alpha}$$

$$\Rightarrow V_p = 2A_{vd}(1.25f_y)\sin\alpha$$

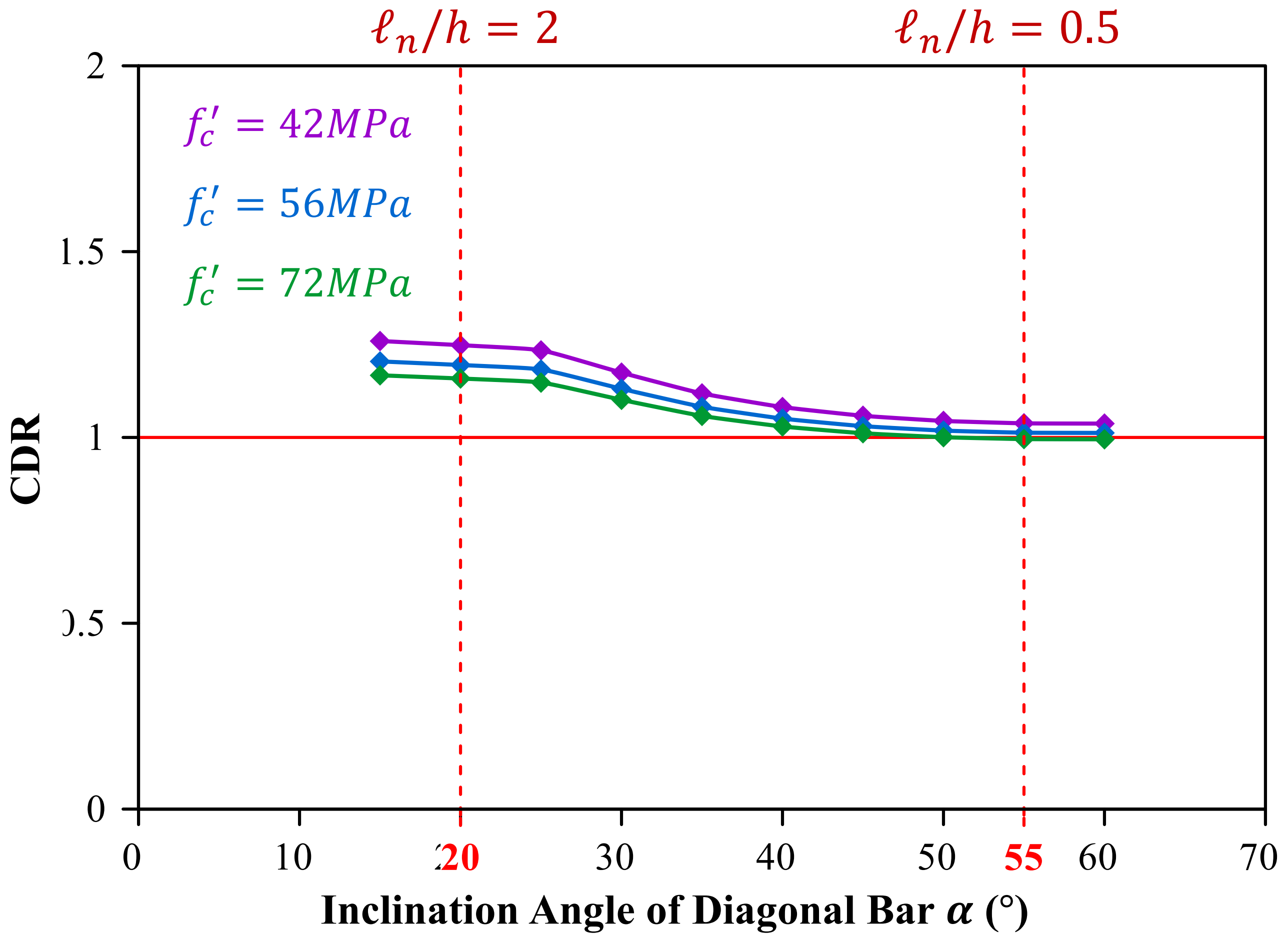
參數化研究

CDR= $\frac{\phi_s V_{n@MCE}}{V_p}$

$$\Rightarrow \frac{\phi_s\left(K\zeta f_c' b_w \frac{1.25 f_y A_{vd}\cos\alpha}{0.85 f_c' b_w}\sin\alpha + 2A_{vd}(1.25 f_y)\sin\alpha\right)}{2A_{vd}(1.25 f_y)\sin\alpha}$$

$$\Rightarrow \frac{\phi_s\left(K\zeta\frac{\cos\alpha}{0.85}+2\right)}{2} \geq 1\text{，}\ \phi_s = 0.85$$

$$\Rightarrow 0.5K\zeta\cos\alpha + 0.85$$



在MCE作用下深短連接梁：

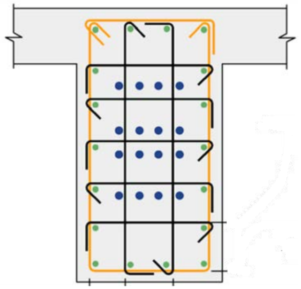
- CDR均大於1.0
- 塑鉸剪力強度之需求自動滿足，無需查驗

設計建議

1. Flexural design (DBE Level) - (撓曲鋼筋全數偏折)

$$M_n \geq \frac{M_u}{\phi} \quad \rightarrow \quad A_{vd} \times \cos\alpha \times f_y \times jd \geq \frac{M_u}{0.9}$$

2. Provide confinement and detailing



3. Shear capacity design (MCE level)

$$\phi V_{n@MCE} \geq V_p$$

剪力強度自動滿足塑鉸剪力之需求，不需查核

ACI 318-25設計之超量對角鋼筋

ACI 31-25 規範要求之對角鋼筋量

[設計剪力需求]

$$M_u = \phi M_n \; ; \; V_u = \frac{2M_u}{\ell_n}$$

[ACI 318-25]

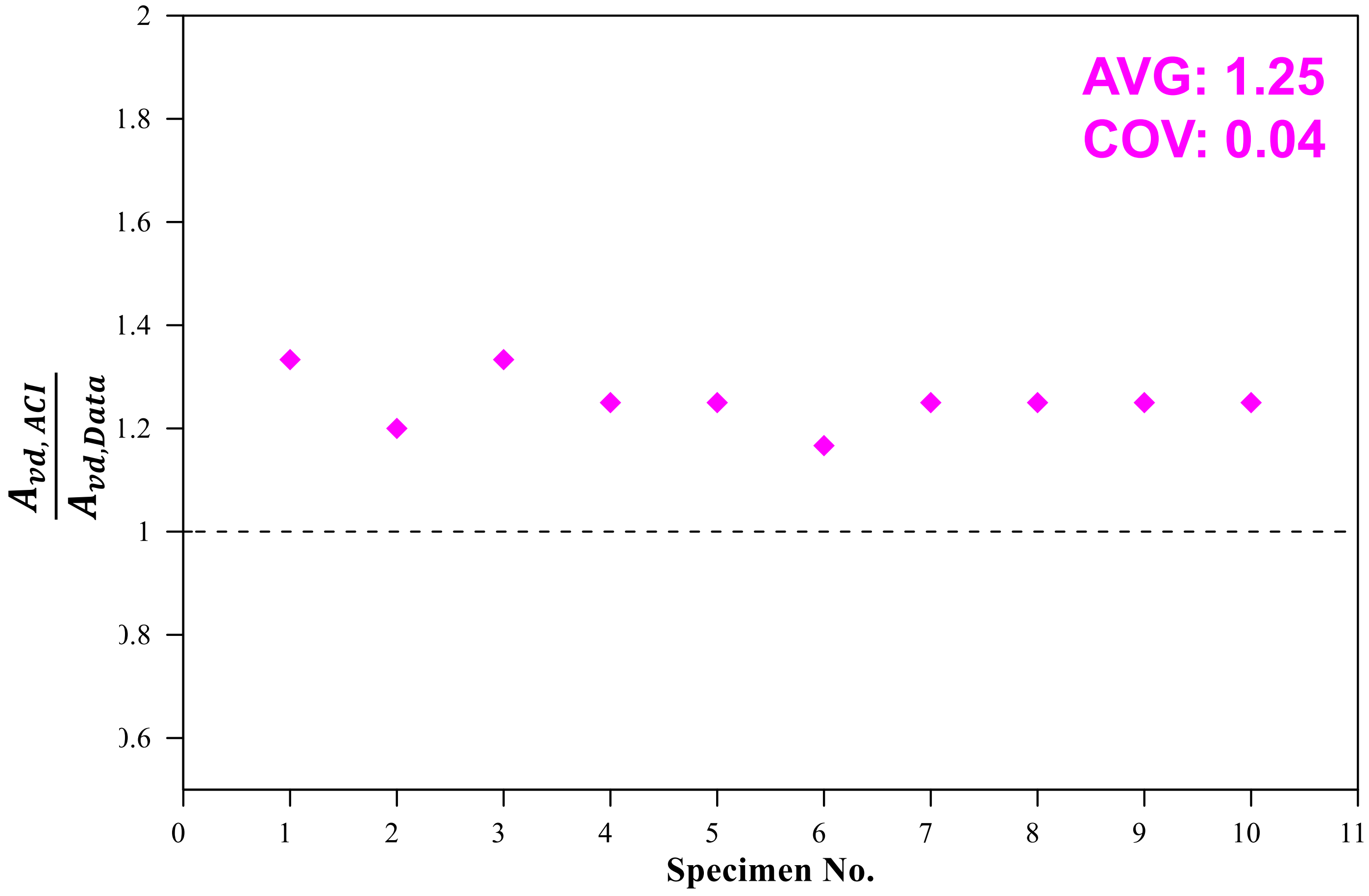
$$2A_{vd,req.}f_{yd}\sin\alpha \geq \frac{V_u}{\phi_s\ (\phi_s\!=\!0.85)}$$

$$A_{vd,ACI} = \frac{V_u}{\phi} \times \frac{1}{2\ f_{yd}\sin\alpha}$$

- 估計ACI 318 – 25 超量設計值

$$\Rightarrow \frac{A_{vd,ACI}}{A_{vd,data}}$$

ACI 318-25 超量設計25%

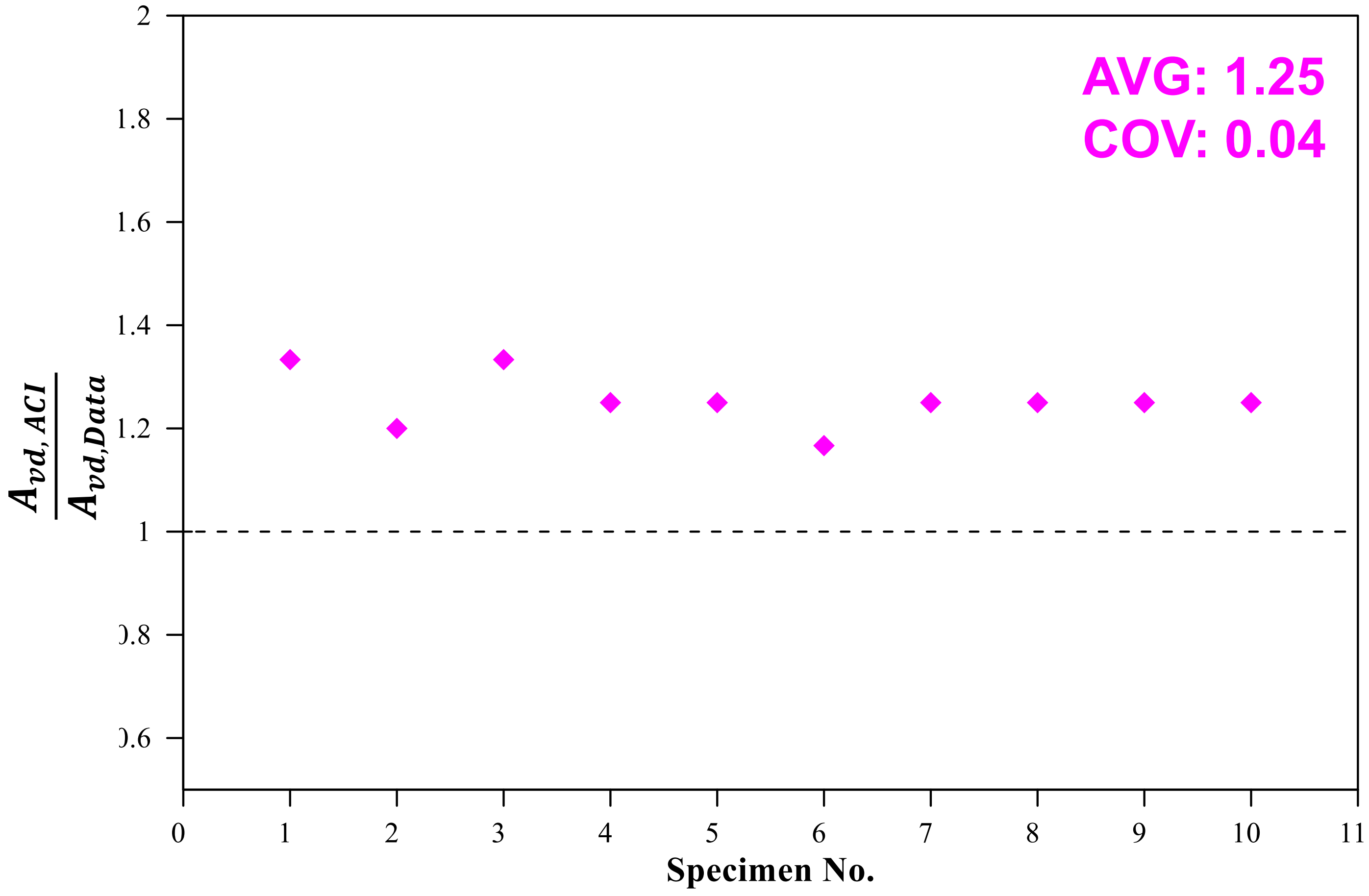


Tsai, R. J., Henry, R, S., Elwood, K. J., Hwang, S. J., and Cheng, M. Y. (2026). "Dataset of diagonally reinforced coupling beams: Axial restraint and strength design," Engineering Structures 349, 121759.

ACI 31-25 規範要求之對角鋼筋量

No.	Author	Spec. ID	V_{test} (kN)	$A_{vd,Data}$ (mm²)	$A_{vd,ACI}$ (mm²)	$\frac{A_{vd,ACI}}{A_{vd,Data}}$
1	Weber-Kamin *et al.*	D80-1.5	1133	1719	2292	1.33
2		D100-1.5	1136	1433	1719	1.20
3	Ameen	CB1	819	2323	3097	1.33
4		CB2	925	1146	1433	1.25
5		CB2D	908	1146	1433	1.25
6		CB3D	1226	1719	2006	1.17
7	Lim et *al.*	CB10-1	1444	2027	2534	1.25
8		CB20-1	1045	2588	3235	1.25
9	Cheng *et al.*	L0	882	2027	2534	1.25
10		H0	1549	4054	5067	1.25
					AVG	1.25
					COV	0.04

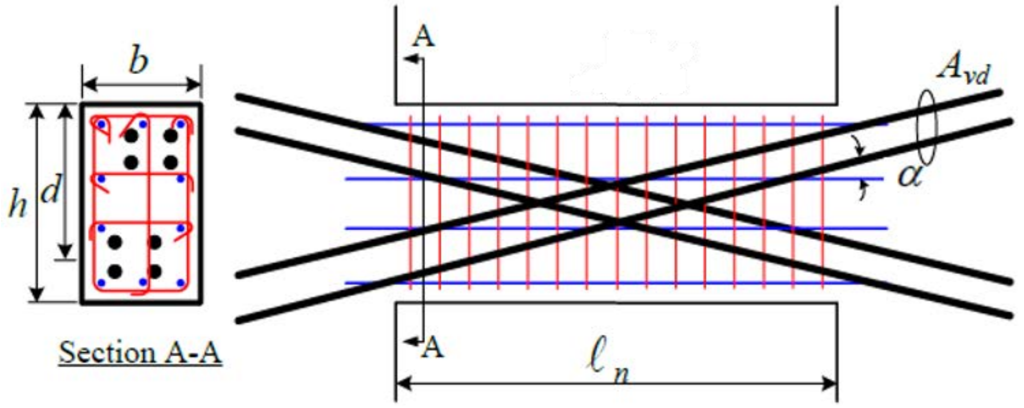
ACI 318-25 超量設計25%

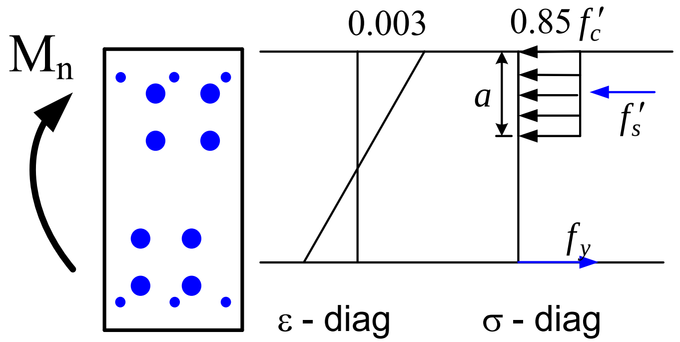


Tsai, R. J., Henry, R, S., Elwood, K. J., Hwang, S. J., and Cheng, M. Y. (2026). “Dataset of diagonally reinforced coupling beams: Axial restraint and strength design,” Engineering Structures 349, 121759.

深短連接梁之設計建議

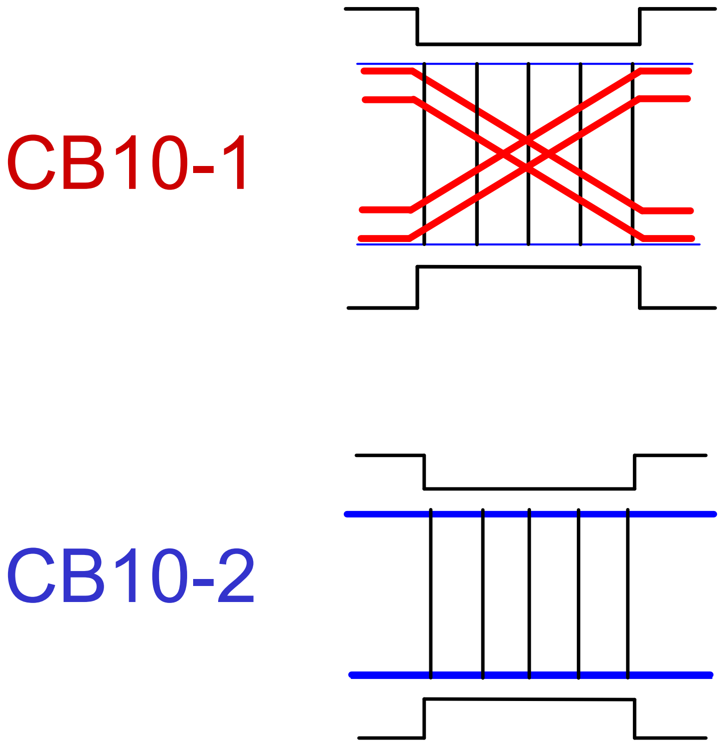
$\ell_n / h < 2$ 全對角鋼筋連接梁之設計

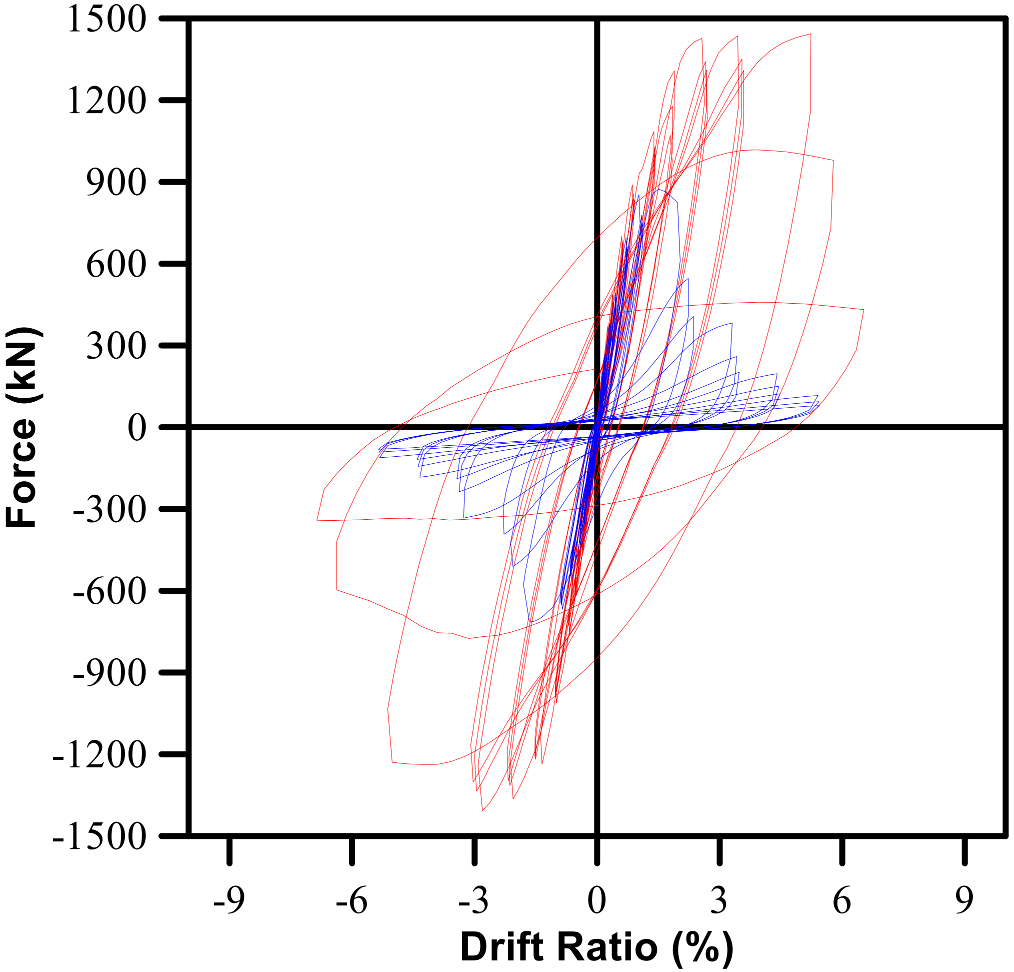




設計地震 Design Based Earthquake **DBE**	$M_n \geq \frac{M_u}{\phi}$ $A_{vd} \times \cos\alpha \times f_y \times jd \geq \frac{M_u}{0.9}$	選定破壞機制 並予以控制
最大可能地震 Max. Credible Earthquake **MCE**	-	$V_n \geq V_p / \phi$ 自動滿足

$\ell_n / h = 1$ 全對角鋼筋連接梁之設計驗證

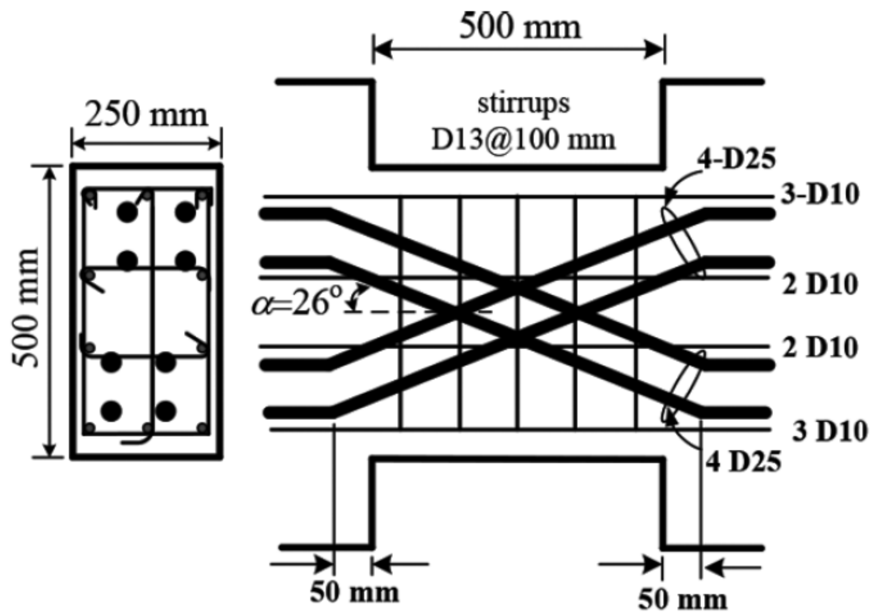




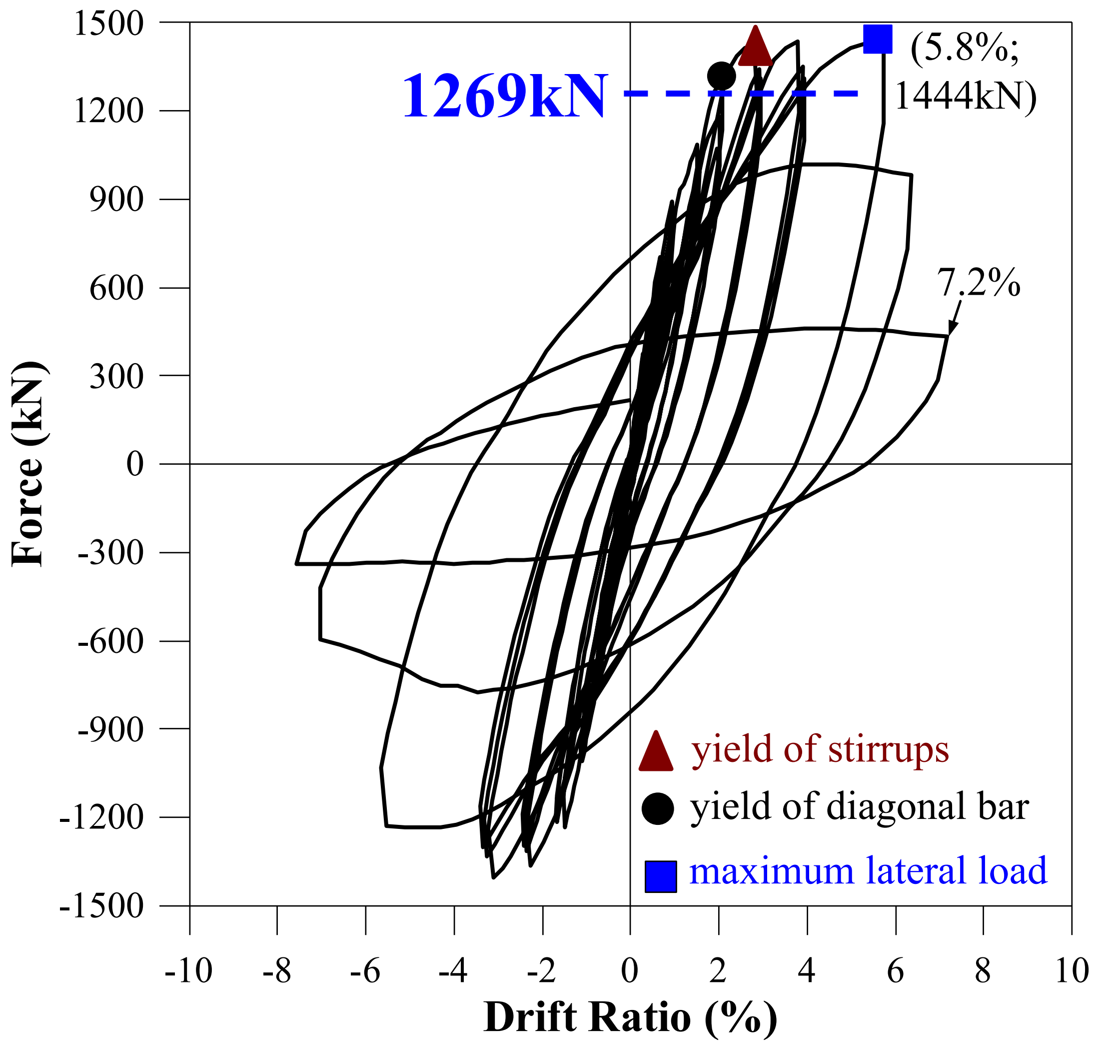
Lim, E., Hwang, S. J., Wang, T. W., and Chang, Y. H. (2016). "An Investigation on the Seismic Behavior of Deep Reinforced Concrete Coupling Beams," ACI Structural Journal, V. 113, No. 2, March-April, pp. 217-226.

$\ell_n / h = 1$ 全對角鋼筋連接梁之設計驗證

CB10-1

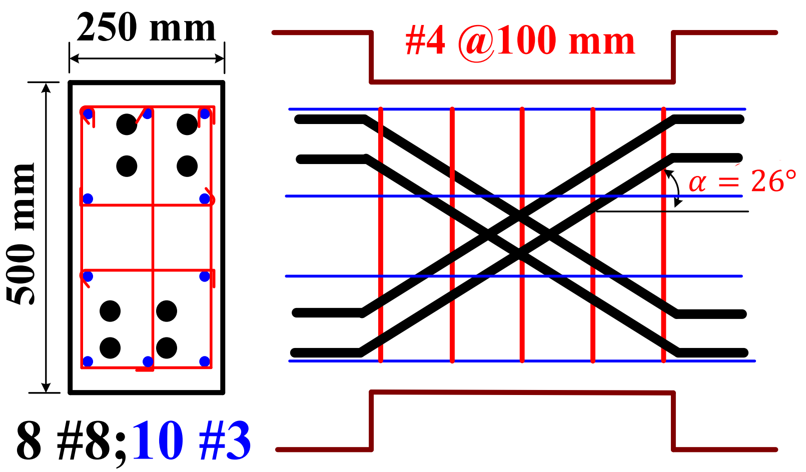


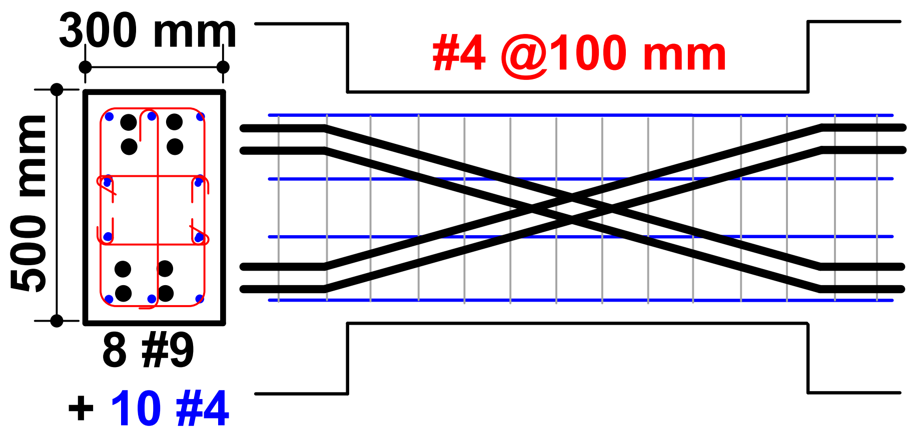
$$V_{mn} = 2M_n/\ell_n = 1269kN$$



Lim, E., Hwang, S. J., Wang, T. W., and Chang, Y. H. (2016). "An Investigation on the Seismic Behavior of Deep Reinforced Concrete Coupling Beams," ACI Structural Journal, V. 113, No. 2, March-April, pp. 217-226.

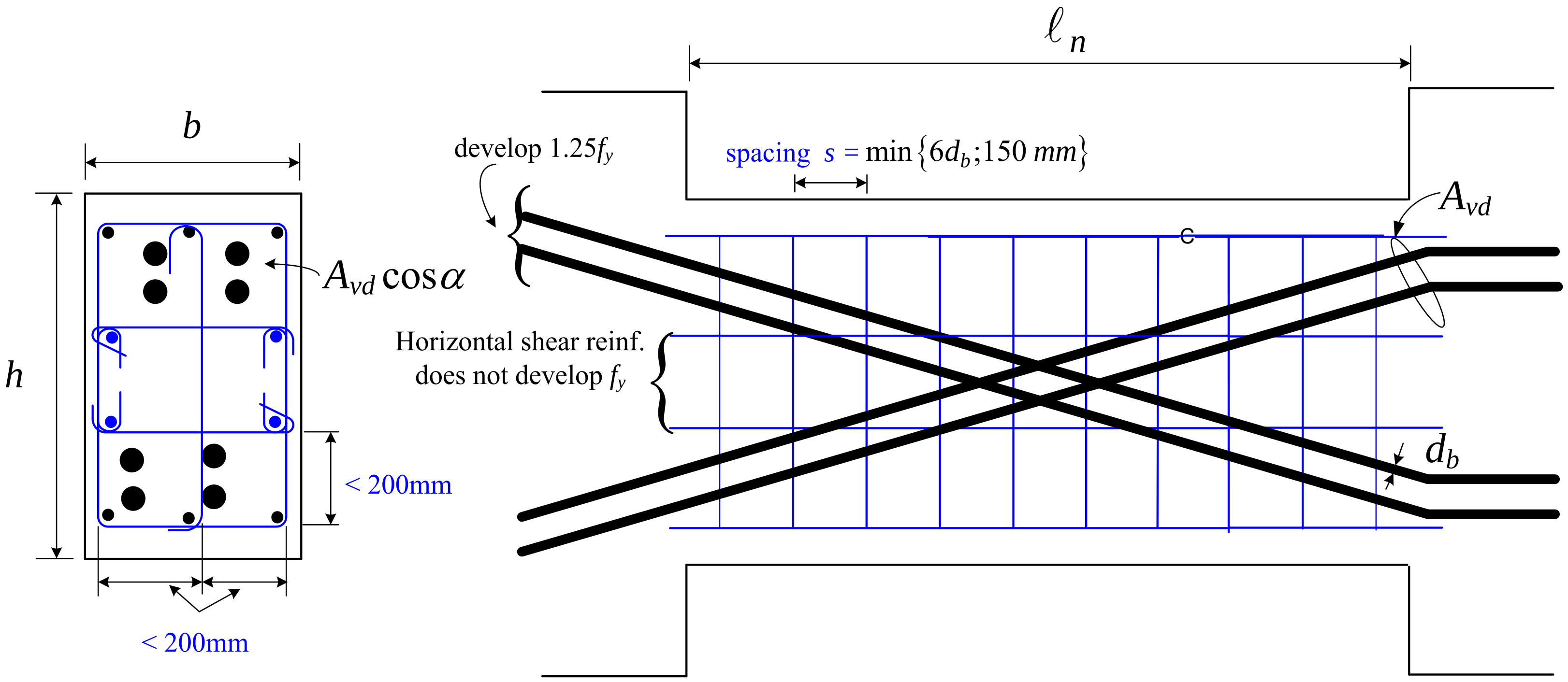
連接梁對角鋼筋配置





- 對角鋼筋應為對跨度中點對稱之兩組鋼筋

連接梁之圍束鋼筋配置



垂直梁寬之橫向鋼筋
暨
平行梁寬之橫向鋼筋

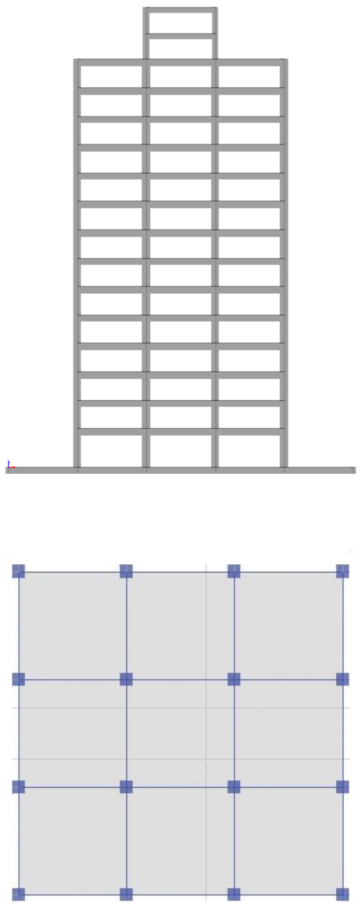
$$A_{sh} = \max\left\{0.3\frac{sb_c f_c'}{f_{yt}}\left(\frac{A_g}{A_{ch}}-1\right); 0.09\frac{sb_c f_c'}{f_{yt}}\right\}$$

確保連接梁混凝土之變形能力、剪力強度

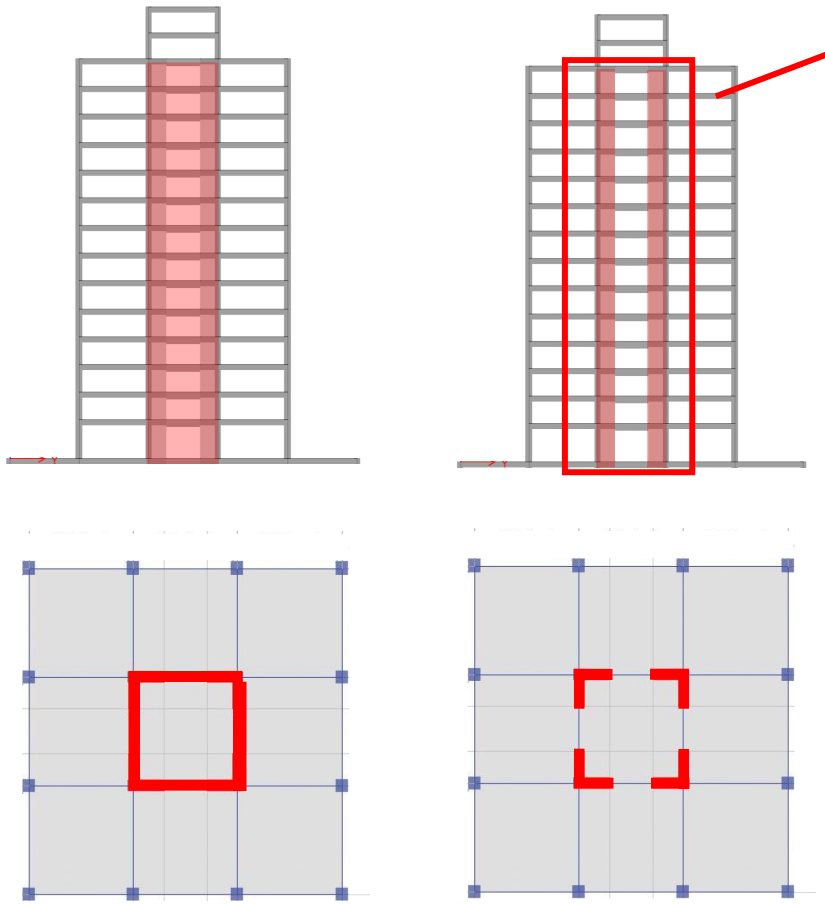
設計案例

結構系統探討

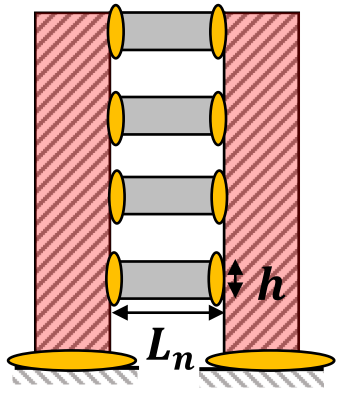
構架系統



二元系統



Span to depth ratio = L_n/h



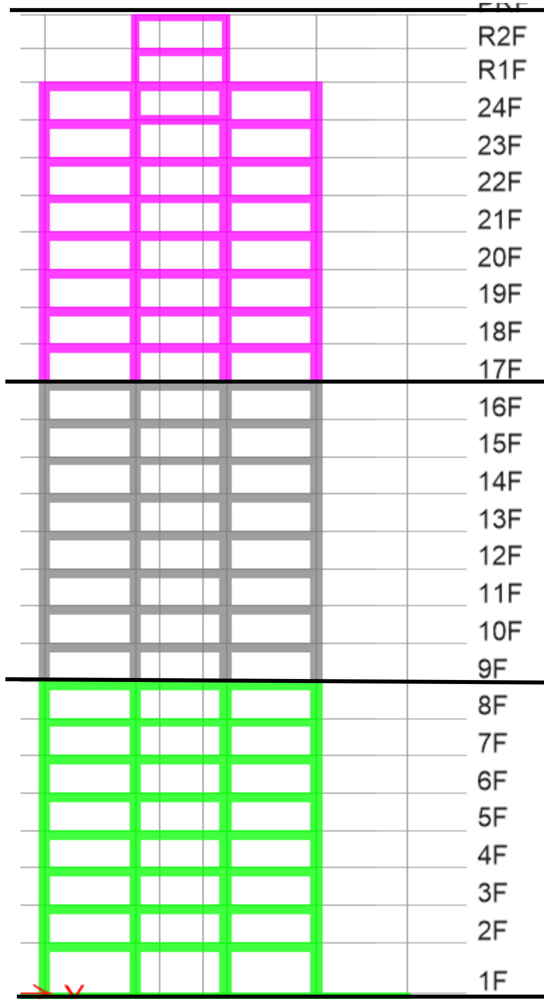
Study Cases:

1. Without opening
2. $L_n/h = 1.5$
3. $L_n/h = 2.4$
4. $L_n/h = 3.3$
5. $L_n/h = 4.2$

沈家瑋，(2024)「鋼筋混凝土耦合牆系統之設計與應用」，碩士論文，國立台灣大學土木工程學系，臺北，臺灣。

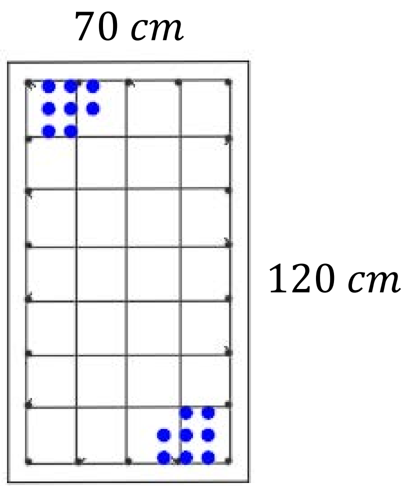
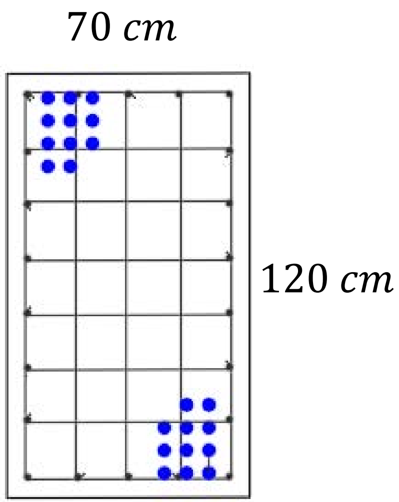
材料強度

24F structure



Story	f_c'	f_y
17F~PRFL	350 kgf/cm^2	4200 kgf/cm^2
9F~17FL	420 kgf/cm^2	5000 kgf/cm^2
1F~9FL	490 kgf/cm^2	5000 kgf/cm^2

24F連接梁設計($L_n/h=1.5$)

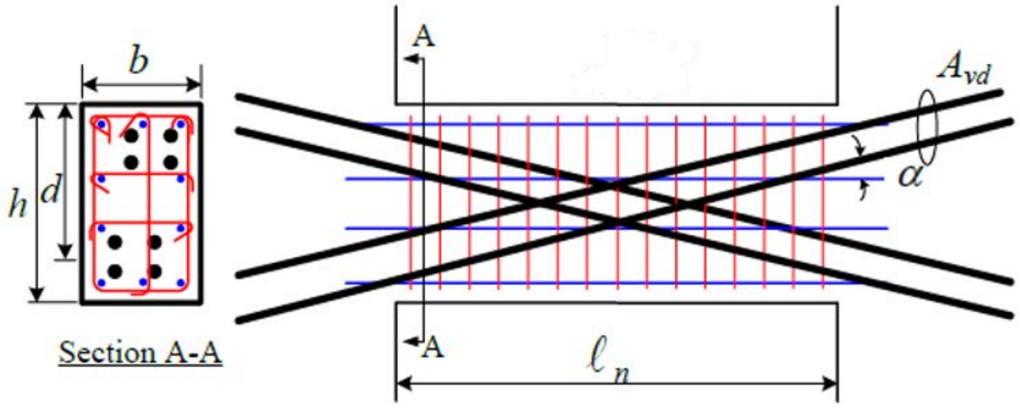
		SST	ACI
Section			
Steel	Diagonal	8-#10	11-#10
	Longitudinal	–	–
備註		考慮混凝土壓桿貢獻	忽略混凝土壓桿貢獻

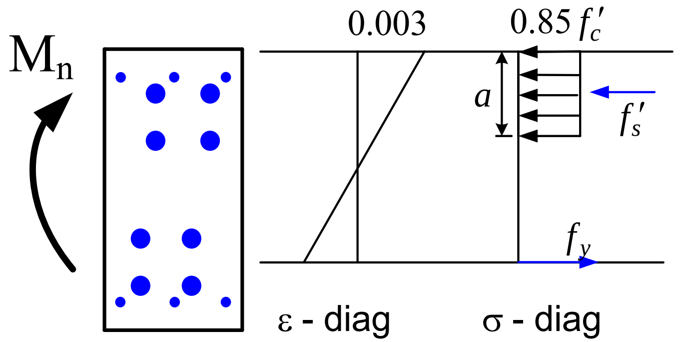
ACI
超量設計
+ 37.5%

詳SST設計案例

結論

$\ell_n / h < 2$ 全對角鋼筋連接梁之設計



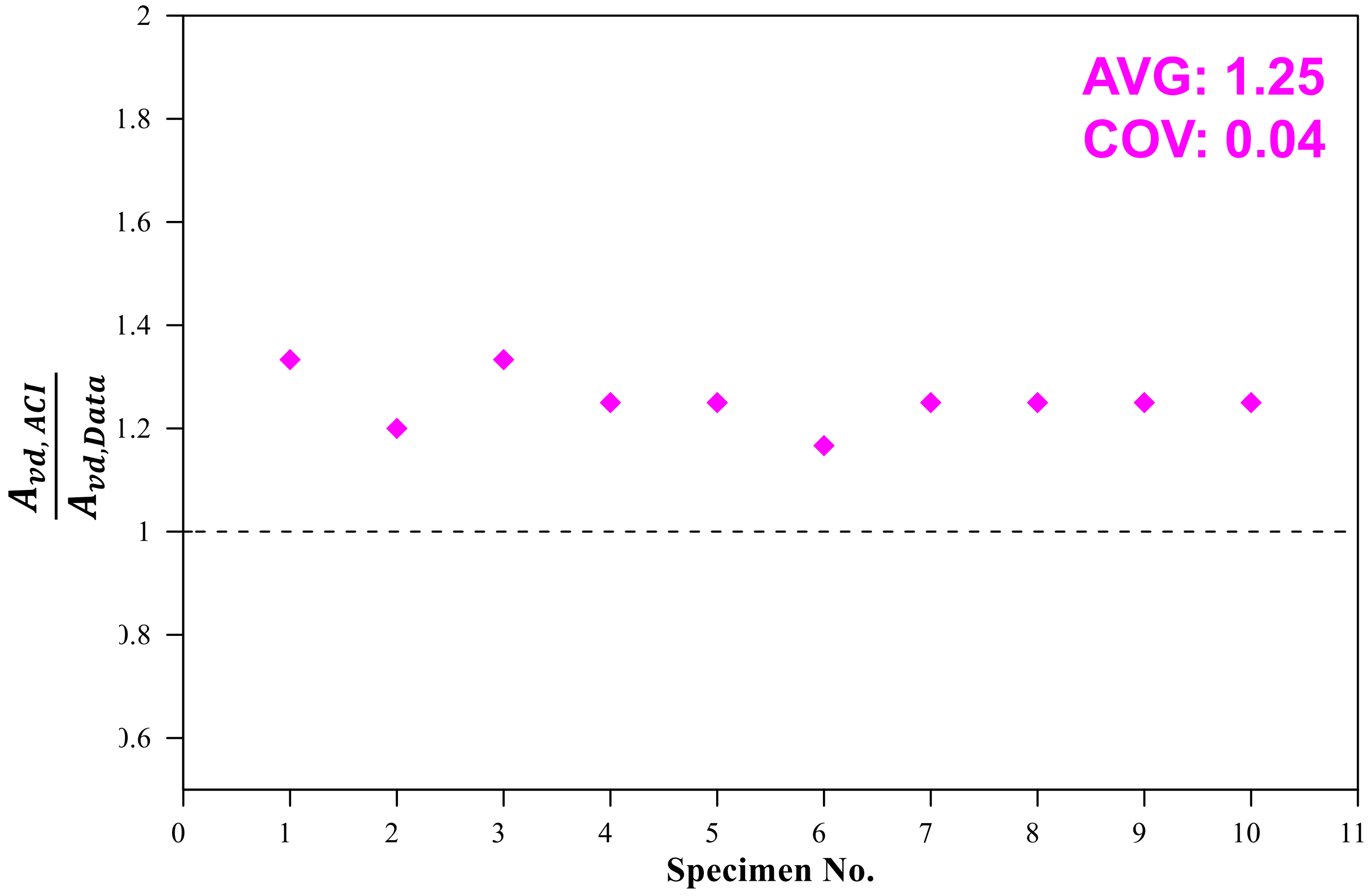


設計地震 Design Based Earthquake **DBE**	$M_n \geq \frac{M_u}{\phi}$ $A_{vd} \times \cos\alpha \times f_y \times jd \geq \frac{M_u}{0.9}$	選定破壞機制 並予以控制
最大可能地震 Max. Credible Earthquake **MCE**	-	$V_n \geq V_p / \phi$ 自動滿足

ACI 31-25 規範要求超量之對角鋼筋

No.	Author	Spec. ID	V_{test} (kN)	$A_{vd,Data}$ (mm²)	$A_{vd,ACI}$ (mm²)	$\frac{A_{vd,ACI}}{A_{vd,Data}}$
1	Weber-Kamin *et al.*	D80-1.5	1133	1719	2292	1.33
2		D100-1.5	1136	1433	1719	1.20
3	Ameen	CB1	819	2323	3097	1.33
4		CB2	925	1146	1433	1.25
5		CB2D	908	1146	1433	1.25
6		CB3D	1226	1719	2006	1.17
7	Lim et *al.*	CB10-1	1444	2027	2534	1.25
8		CB20-1	1045	2588	3235	1.25
9	Cheng *et al.*	L0	882	2027	2534	1.25
10		H0	1549	4054	5067	1.25
					AVG	1.25
					COV	0.04

ACI 318-25 超量設計25%



Tsai, R. J., Henry, R, S., Elwood, K. J., Hwang, S. J., and Cheng, M. Y. (2026). "Dataset of diagonally reinforced coupling beams: Axial restraint and strength design," Engineering Structures 349, 121759.

敬請指教！

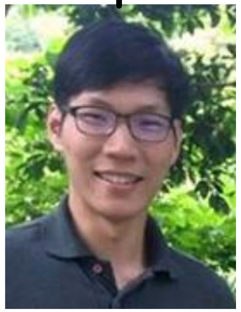
耦合牆系統連接梁研究

2009~2016

耦合牆連接梁

實驗研究

鄭志宏
(2009-2010)



王亭惟
(2010-2011)

張于軒
(2011-2012)

蔡尚錡
(2012-2013)

林秉誼
(2013-2014)

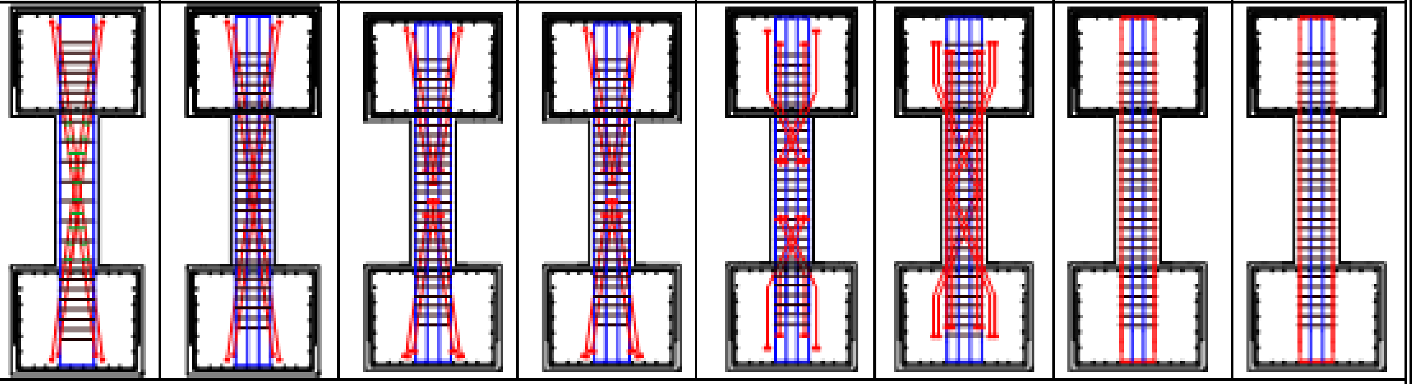
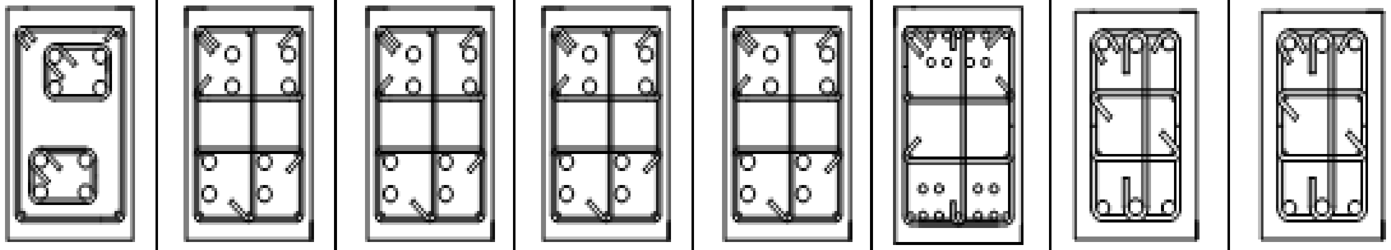
分析研究

Erwin Lim
(2009-2015)

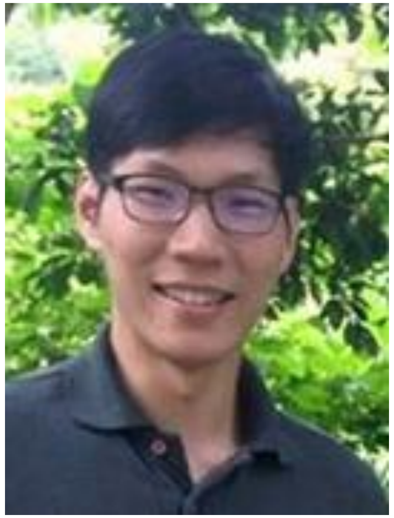
設計研究

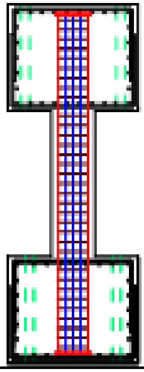
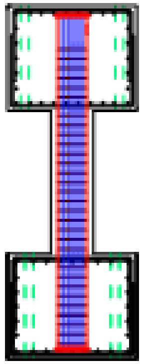
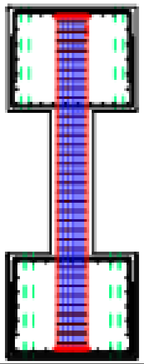
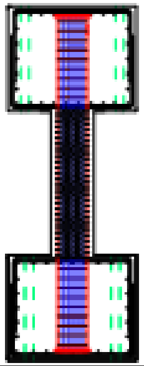
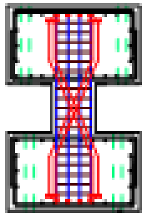
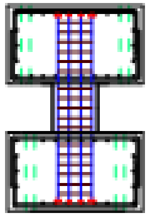
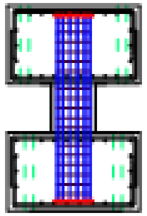
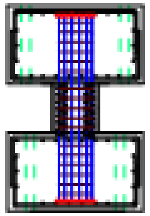
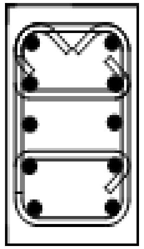
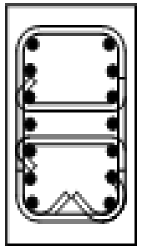
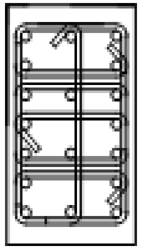
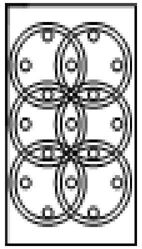
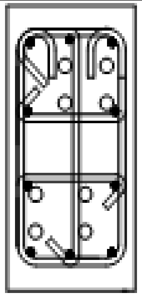
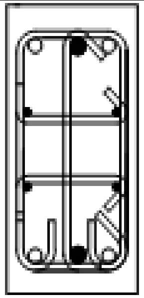
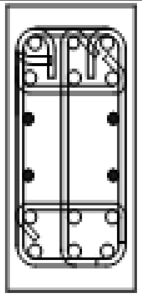
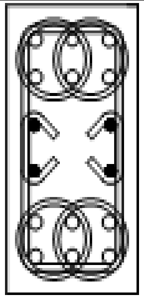
Luan Tat Nguyen
(2015-2016)

First Year Study (鄭志宏 2010)

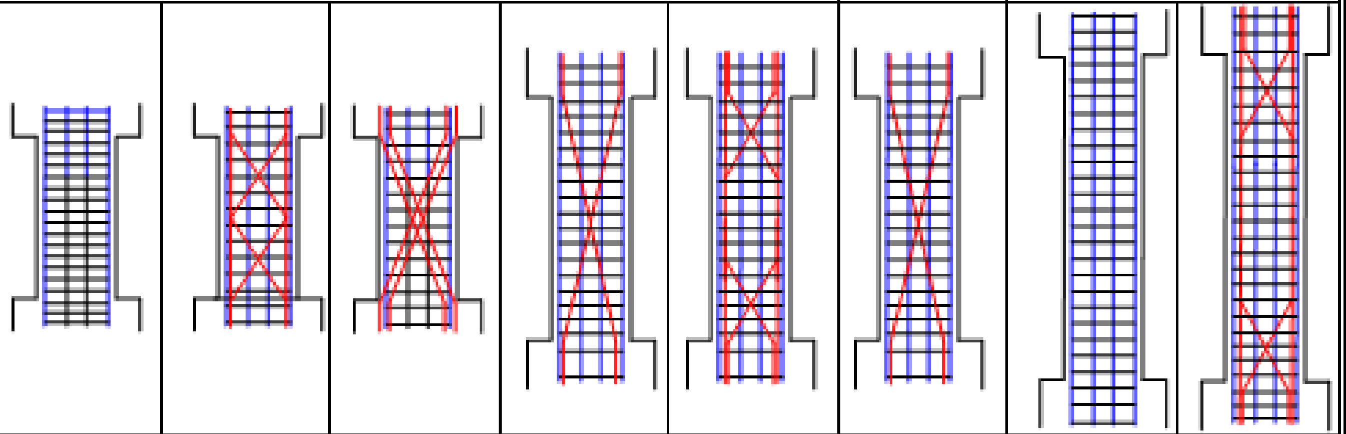
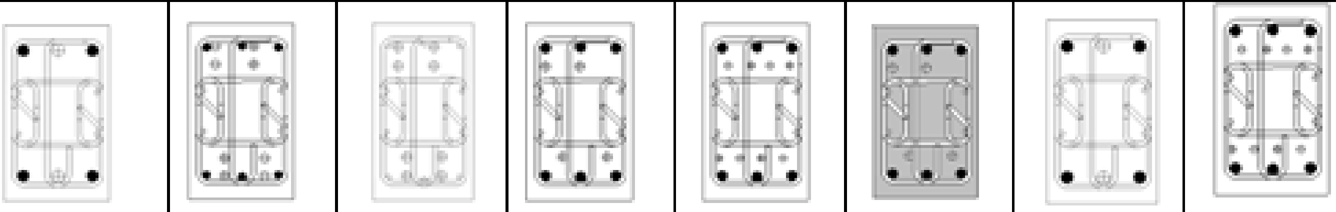
試體名稱	CB1	CB2	CB3	CB4	CB5	CB6	CB7	CB8
試體圖示								
梁斷面尺寸	$50 \times 30\,cm^2$							
跨深比(l_n/h)	3							
材料性質 鋼筋	$f_y = 4200\,kgf/cm^2$							
材料性質 混凝土	$f'_c = 280\,kgf/cm^2$		350		420		420+ FRC	
斷面圖示								
對角鋼筋	#10	#10	#10	#10	#10	#6	-	-
縱向鋼筋	#4	#3	#3	#3	#3	#3+#5	#3+#10 +#11	#3+#10 +#11

Second Year Study (王亭惟 2011)

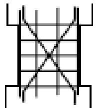
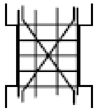
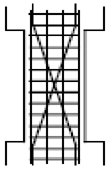
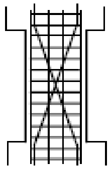
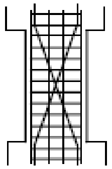
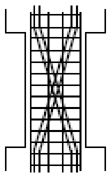
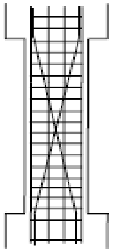
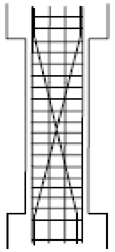
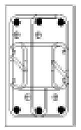
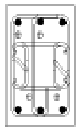
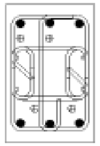
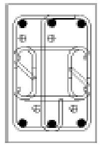
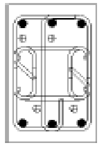
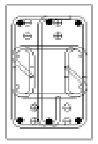
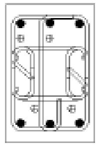
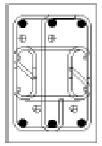


試體名稱		CB9	CB10	CB11	CB12	CBI	CBII	CBIII	CBIV
試體圖示									
梁斷面尺寸		$50\times30\,cm^2$				$50\times25\,cm^2$			
跨深比(l_n/h)		3				1			
材料性質	鋼筋	$f_y=4200\,kgf/cm^2$							
	混凝土	$f_c'=420\,kgf/cm^2$				$f_c'=280\,kgf/cm^2$			
斷面圖示									
對角鋼筋		-	-	-	-	#8	-	-	-
縱向鋼筋		#9	#8	#7	#7	#4	#4+#8+#9	#6+#8	#6+#8

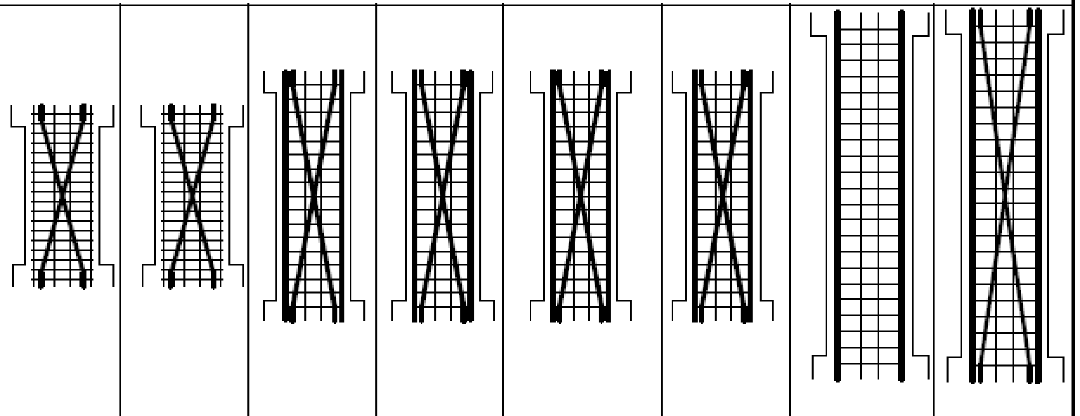
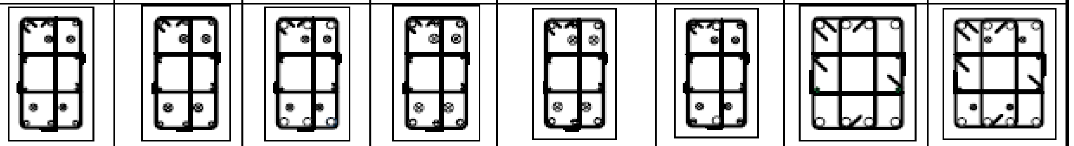
Third Year Study (張于軒 2012)

編號	B20-1	B20-2	B20-3	B30-13	B30-14	B30-15	B40-1	B40-2
梁斷面尺寸	$50\times30cm^2$							
跨深比(l_n/h)	2			3			4	
材料性質 鋼筋	4200 kgf/cm²							
材料性質 混凝土	420 kgf/cm²					210 kgf/cm²	420 kgf/cm²	
								
斷面圖示								
對角鋼筋	-	4-#8(⊕)	8-#9(⊕)	4-#8(⊕)	4-#6(⊕)	4-#8(⊕)	-	4-#6(⊕)
縱向鋼筋	4-#4(∘) 4-#10(•) 2-#11(⊕)	4-#4(∘) 6-#7(•) 4-#8(⊖)	10-#4(∘)	4-#4(∘) 6-#9(•)	4-#4(∘) 4-#6(⊖) 6-#9(•)	4-#4(∘) 6-#9(•)	4-#4(∘) 4-#10(•) 2-#11(⊕)	4-#4(∘) 4-#6(⊖) 6-#9(•)

Fourth Year Study (蔡尚錡 2013)

編號		CB10		CB20				CB30	
		P-N-N	P -H-H	P-N-N	P-H-N	P-H-H	D-H-H	P-H-N	P-H-H
梁斷面尺寸		25 *cm* ×40 *cm*		30 *cm* ×50 *cm*					
跨深比(l_n/h)		1		2				3	
材料性質	縱向鋼筋	420MPa	420 MPa	420 MPa		420 MPa		420 MPa	420 MPa
	對角鋼筋		685 MPa			685 MPa			685 MPa
	混凝土	28MPa	70 MPa	42 MPa	70 MPa				
									
斷面圖示									
箍筋&繫筋		#4	#4	#4	#4	#4	#4	#4	#4
對角鋼筋		4-#8(⊕)	4-#8(⊕)	4-#8(⊕)	4-#8(⊕)	4-#8(⊕)	8-#8(⊕)	4-#8(⊕)	4-#8(⊕)
縱向鋼筋		4-#4(◦) 6-#6(•)	4-#4(◦) 6-#6(•)	4-#4(◦) 6-#9(•)	4-#4(◦) 6-#9(•)	4-#4(◦) 6-#9(•)	4-#4(◦) 6-#4(•)	4-#4(◦) 6-#9(•)	4-#4(◦) 6-#9(•)

Fifth Year Study (林秉誼2014)

編號	CB20		CB30				CB40	
	1.5-53-H	1.5-68-N	2.5-34-H	2.5-54-N	2.5-54-N(P)	2.0-41-N	2.0-0-N	2.0-23-N
梁斷面尺寸	30 cm ×50 cm						40 cm ×50 cm	
跨深比(L_n/h)	2		3				4	
材料性質 縱向鋼筋	420MPa							
材料性質 對角鋼筋	420MPa							
材料性質 混凝土	42MPa							
								
斷面圖示								
箍筋&繫筋	#4@7cm 785MPa	#4@7cm 420MPa	#4@10cm 785MPa	#4@10cm 420MPa				
對角鋼筋	4-#8(⊗)	4-#9(⊗)	4-#8(⊗)	4-#10(⊗)	4-#10(⊗)	4-#8(⊗)	-	4-#7(⊗)
縱向鋼筋	4-#4(•) 6-#6(○)	4-#4(•) 6-#5(○)	4-#4(•) 6-#9(○)	4-#4(•) 4-#8(○) 2-#7(⊖)	4-#4(•) 4-#8(○) 2-#7(⊖)	4-#4(•) 2-#9(○) 4-#7(⊖)	4-#4(•) 8-#10(○)	4-#4(•) 8-#9(○)

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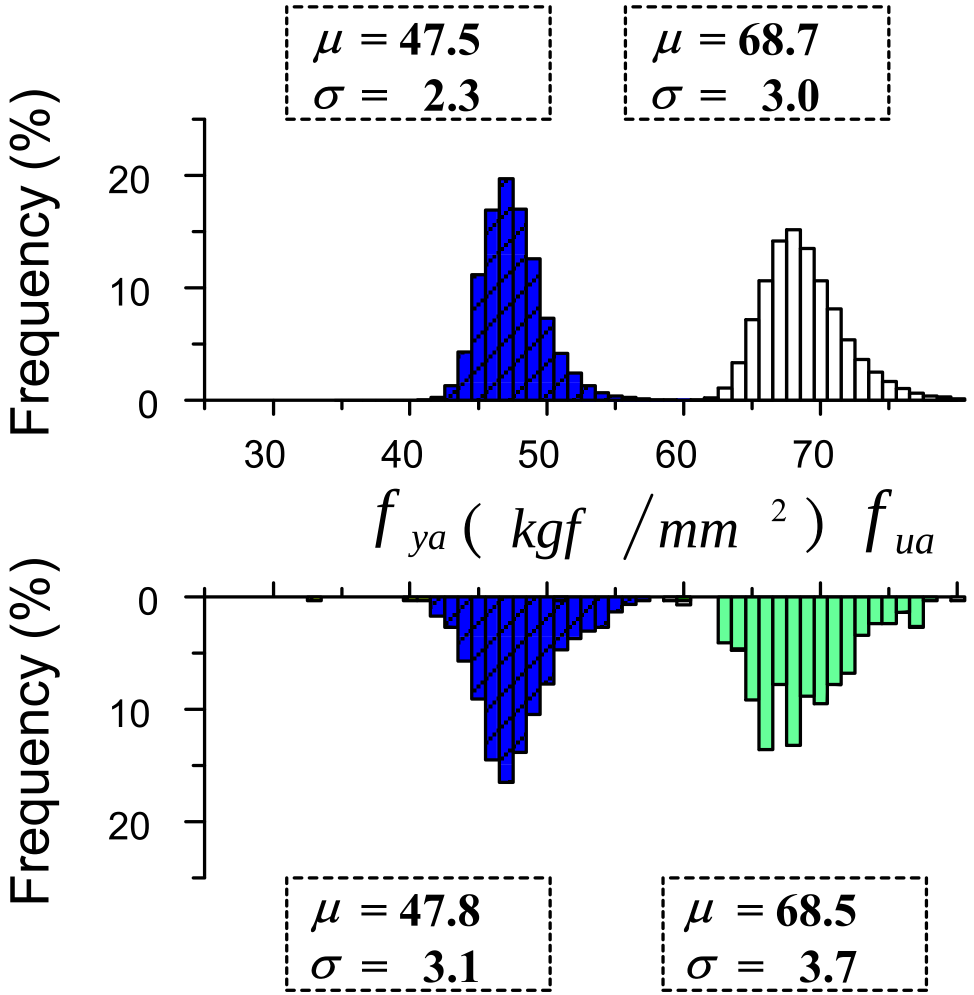
- 沈家瑋，(2024)「鋼筋混凝土耦合牆系統之設計與應用」，碩士論文，國立台灣大學土木工程學系，臺北，臺灣。

附錄：連接梁強度評估計算

台灣SD420鋼筋之力學性質統計



- #6 or larger

Sample = 48,921

- #5 or smaller

Sample = 297

黃世建、陳正誠、李宏仁，(2000)「台灣熱軋竹節鋼筋之力學性質與耐震韌性設計可行性探討」，結構工程，第十五卷，第一期，第23-38頁。

撓曲主筋超額強度之估計 － αf_y

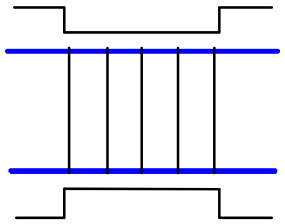
Place	Reference	f_y kgf/cm²	$f_{y,\ actual}$ kgf/cm²		$\frac{(f_{y,actual})_\mu}{f_y}$
			μ	σ	
Taiwan	本研究1999	4200	4750	230	1.13

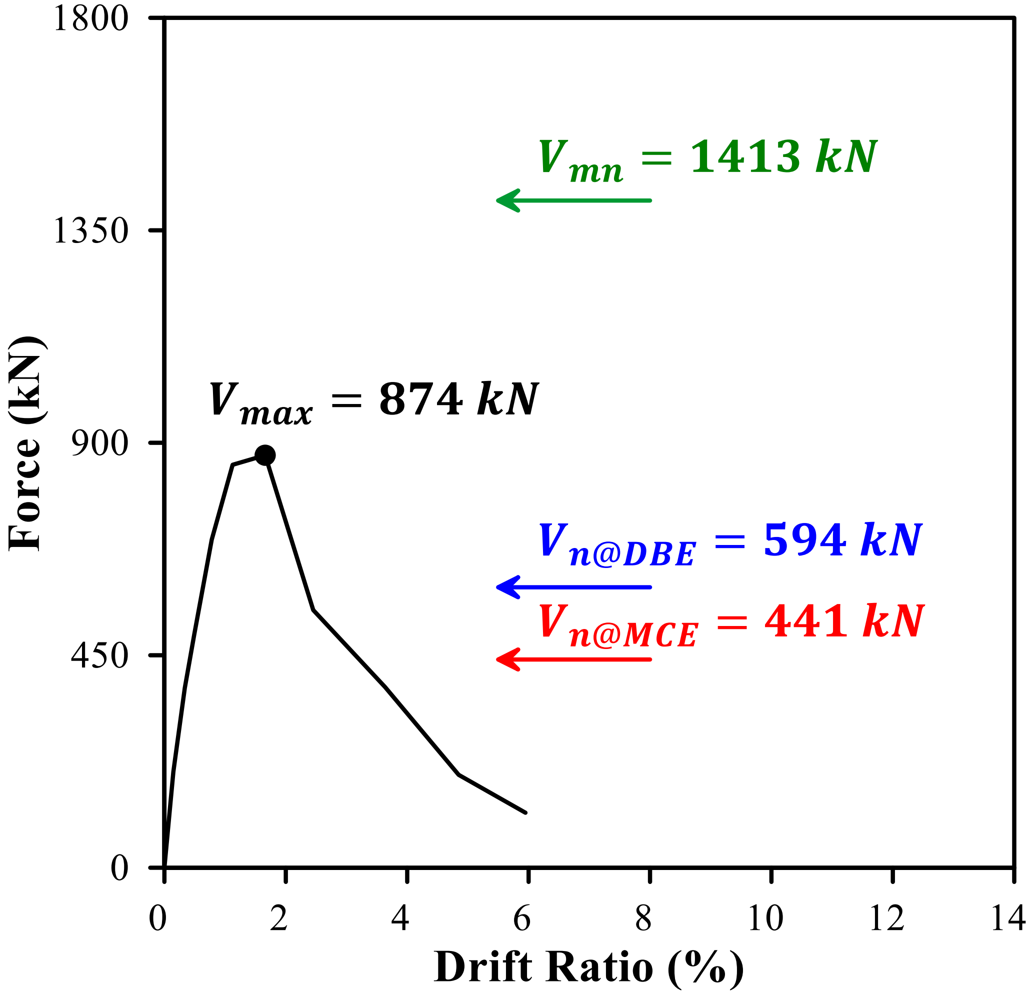
- $f_{y,actual} = 1.13 \times f_y$
- 塑鉸充分發展時，撓曲主筋之應變硬化約略達$1.10 \times f_{y,actual}$
- $1.10 \times f_{y,actual} = 1.10 \times 1.13 f_y \approx 1.25 \times f_y$

$\ell_n / h = 1.0$

撓曲與剪力強度評估

CB10-2





[Flexure Strength]

$M_n = 353\ kN\text{-}m$

$V_{mn} = 2M_n/\ell_n = 1413\ kN$

[Strength - DBE]

$kd = \left(\sqrt{(\rho_f n)^2 + 2\rho_f n} - \rho_f n\right) d = 175\ mm$

$A_{str} = b_w \times kd = 43796\ mm^2$

$\theta = \tan^{-1}\left(\frac{h - 2\times kd/3}{\ell_n}\right) = 37.5°$

$K = \tan^A \theta + \cot^A \theta - 1 + 0.14B \le 1.64 \xrightarrow{take} 1.21$

$\zeta = \frac{3.35}{\sqrt{f_c'}} \le 0.52 \xrightarrow{take} 0.52$

$C_d \sin\theta = K A_{str} \zeta f_c' \sin\theta = 594\ kN$

$V_{n@DBE} = C_d \sin\theta + 0 = 594\ kN$

[Strength - MCE]

$a = \frac{1.1 f_{yl,actual} \times A_{st}}{0.85 f_c' \times b_w} = 131\ mm$

$A_{str} = b_w \times a = 32746\ mm^2$

$\theta = \tan^{-1}\left(\frac{h - 2\times a/2}{\ell_n}\right) = 36.4°$

$K = \tan^A \theta + \cot^A \theta - 1 + 0.14B \le 1.64 \xrightarrow{take} 1.23$

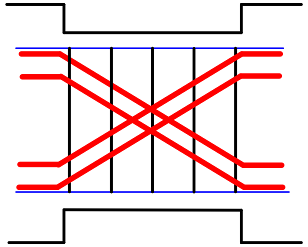
$C_d \sin\theta = K A_{str} \zeta f_c' \sin\theta = 441\ kN$

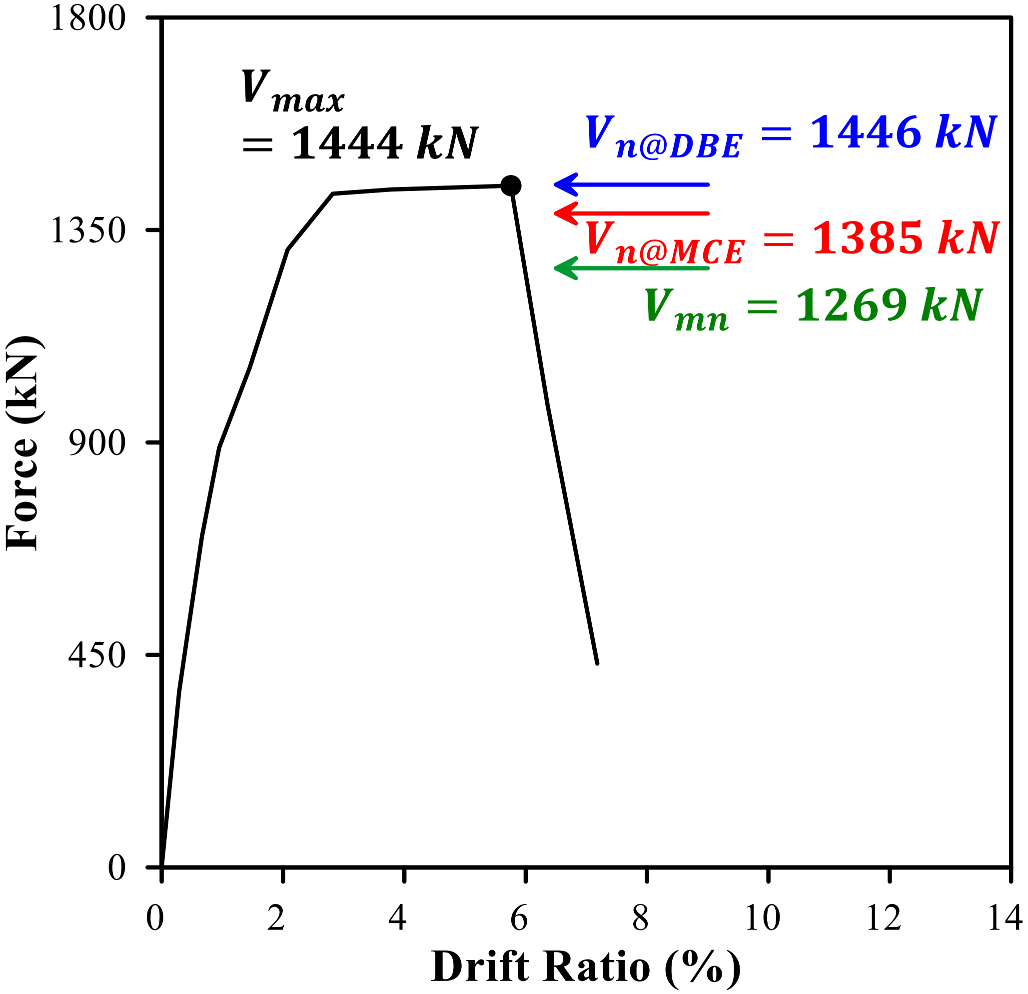
$V_{n@MCE} = C_d \sin\theta + 0 = 441\ kN$

$\ell_n / h = 1.0$

撓曲與剪力強度評估

CB10-1





[Flexure Strength]

$M_n = 317\ kN\text{-}m$

$V_{mn} = 2M_n/\ell_n = 1269\ kN$

[Strength - DBE]

$kd = \left(\sqrt{(\rho_f n)^2 + 2\rho_f n} - \rho_f n\right) d = 181\ mm$

$A_{str} = b_w \times kd = 45252\ mm^2$

$\theta = \tan^{-1}\left(\frac{h - 2 \times kd/3}{\ell_n}\right) = 37.2°$

$K = \tan^A \theta + \cot^A \theta - 1 + 0.14B \leq 1.64 \xrightarrow{take} 1.22$

$\zeta = \frac{3.35}{\sqrt{f_c'}} \leq 0.52 \xrightarrow{take} 0.52$

$C_d \sin\theta = K A_{str} \zeta f_c' \sin\theta = 583\ kN$

$(1.0 + 1.0) A_{vd} f_{yd} \sin\alpha = 863\ kN$

$V_{n@DBE} = C_d \sin\theta + (1.0 + 1.0) A_{vd} f_{yd} \sin\alpha = 1446\ kN$

[Strength - MCE]

$a = \frac{1.1 f_{yd,actual} \times A_{vd} \cos\alpha}{0.85 f_c' \times b_w} = 136\ mm$

$A_{str} = b_w \times a = 33977\ mm^2$

$\theta = \tan^{-1}\left(\frac{h - 2 \times a/2}{\ell_n}\right) = 36.1°$

$K = \tan^A \theta + \cot^A \theta - 1 + 0.14B \leq 1.64 \xrightarrow{take} 1.24$

$C_d \sin\theta = K A_{str} \zeta f_c' \sin\theta = 435\ kN$

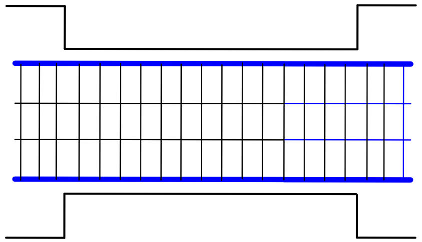
$(1.1 + 1.1) A_{vd} f_{yd,actual} \sin\alpha = 949\ kN$

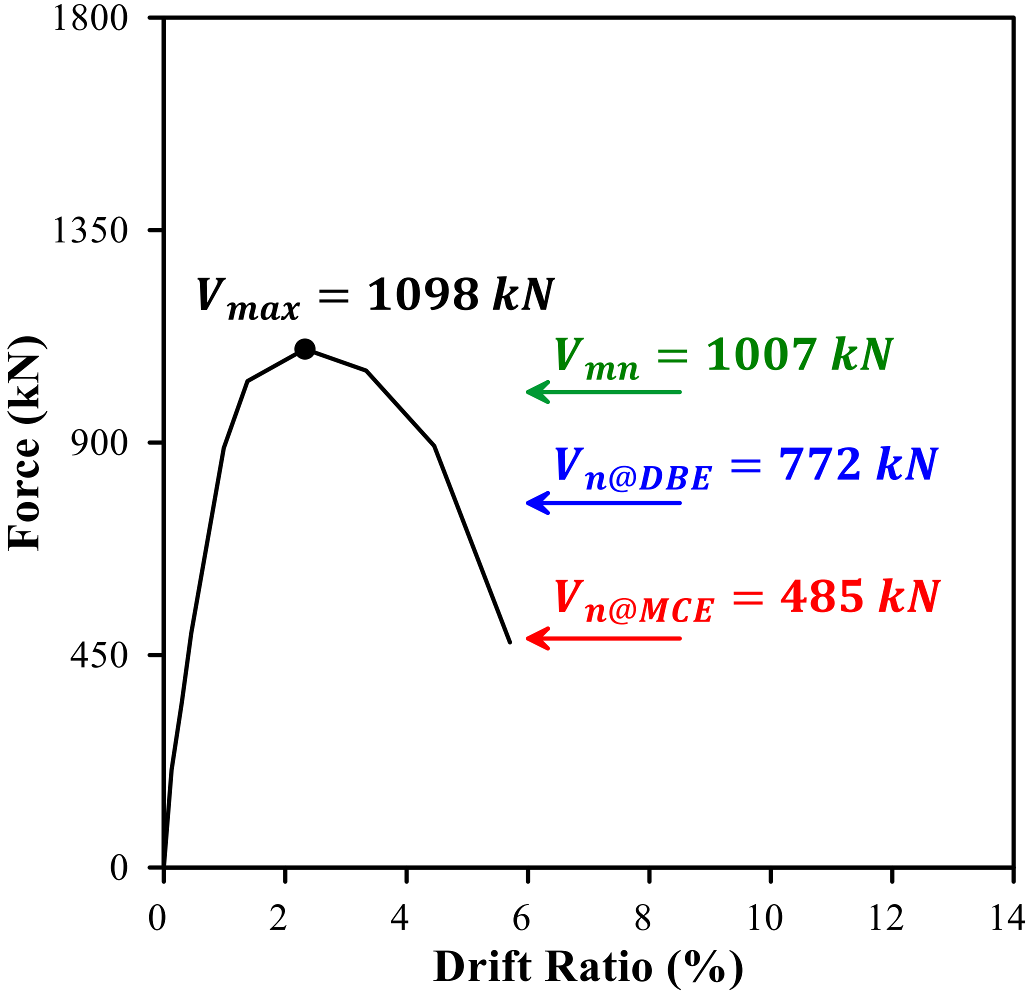
$V_{n@MCE} = C_d \sin\theta + (1.1 + 1.1) A_{vd} f_{yd,actual} \sin\alpha = 1385\ kN$

撓曲與剪力強度評估

$\ell_n / h = 2.0$

CB20-2





[Flexure Strength]

$M_n = 503\ kN\text{-}m$

$V_{mn} = 2M_n/\ell_n = 1007\ kN$

[Strength – DBE]

$$kd = \left(\sqrt{(\rho_f n)^2 + 2\rho_f n} - \rho_f n\right) d = 179\ mm$$

$$A_{str} = b_w \times kd = 53596\ mm^2$$

$$\theta = \tan^{-1}\left(\frac{h - 2\times kd/3}{\ell_n}\right) = 20.9°$$

$$K = \tan^A \theta + \cot^A \theta - 1 + 0.14B \le 1.64 \xrightarrow{take} 1.64$$

$$\zeta = \frac{3.35}{\sqrt{f_c'}} \le 0.52 \xrightarrow{take} 0.46$$

$$C_d \sin\theta = K A_{str} \zeta f_c' \sin\theta = 772\ kN$$

$$V_{n@DBE} = C_d \sin\theta + 0 = 772\ kN$$

[Strength - MCE]

$$a = \frac{1.1 f_{yl,actual} \times A_{st}}{0.85 f_c' \times b_w} = 110\ mm$$

$$A_{str} = b_w \times a = 33034\ mm^2$$

$$\theta = \tan^{-1}\left(\frac{h - 2\times a/2}{\ell_n}\right) = 21.3°$$

$$K = \tan^A \theta + \cot^A \theta - 1 + 0.14B \le 1.64 \xrightarrow{take} 1.64$$

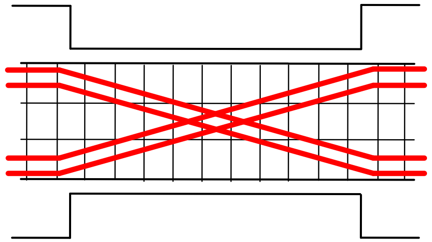
$$C_d \sin\theta = K A_{str} \zeta f_c' \sin\theta = 485\ kN$$

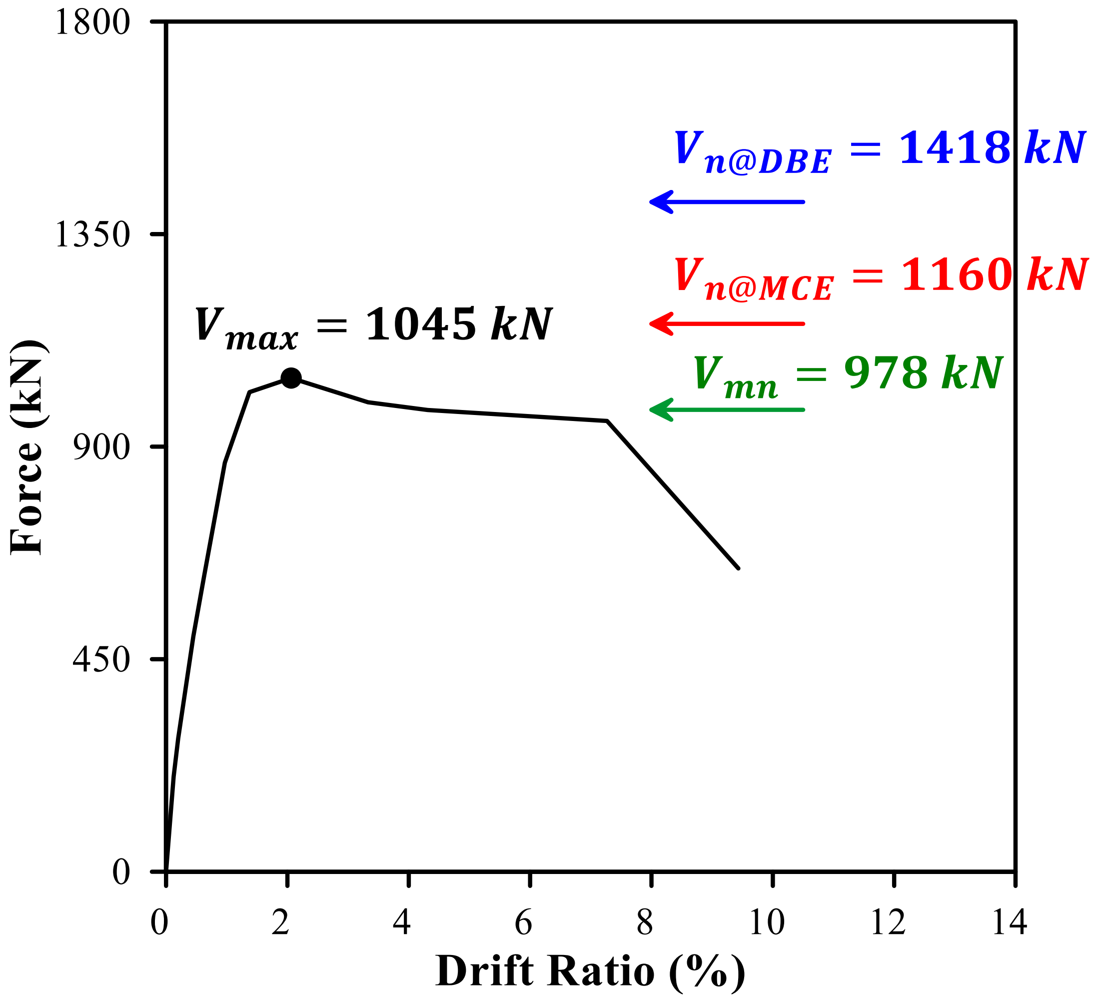
$$V_{n@MCE} = C_d \sin\theta + 0 = 485\ kN$$

撓曲與剪力強度評估

$\ell_n / h = 2.0$

CB20-1





[Flexure Strength]

$M_n = 489\ kN\text{-}m$

$V_{mn} = 2M_n/\ell_n = 978\ kN$

[Strength - DBE]

$$kd = \left(\sqrt{(\rho_f n)^2 + 2\rho_f n} - \rho_f n\right) d = 177\ mm$$

$$A_{str} = b_w \times kd = 53203\ mm^2$$

$$\theta = \tan^{-1}\left(\frac{h - 2 \times kd/3}{\ell_n}\right) = 20.9°$$

$$K = \tan^A \theta + \cot^A \theta - 1 + 0.14B \leq 1.64 \xrightarrow{take} 1.64$$

$$\zeta = \frac{3.35}{\sqrt{f_c'}} \leq 0.52 \xrightarrow{take} 0.46$$

$$C_d \sin\theta = K A_{str} \zeta f_c' \sin\theta = 753\ kN$$

$$(1.0 + 1.0) A_{vd} f_{yd} \sin\alpha = 665\ kN$$

$$V_{n@DBE} = C_d \sin\theta + (1.0 + 1.0) A_{vd} f_{yd} \sin\alpha = 1418\ kN$$

[Strength - MCE]

$$a = \frac{1.1 f_{yd,actual} \times A_{vd} \cos\alpha}{0.85 f_c' \times b_w} = 96\ mm$$

$$A_{str} = b_w \times a = 28810\ mm^2$$

$$\theta = \tan^{-1}\left(\frac{h - 2 \times a/2}{\ell_n}\right) = 22.0°$$

$$K = \tan^A \theta + \cot^A \theta - 1 + 0.14B \leq 1.64 \xrightarrow{take} 1.64$$

$$C_d \sin\theta = K A_{str} \zeta f_c' \sin\theta = 428\ kN$$

$$(1.1 + 1.1) A_{vd} f_{yd,actual} \sin\alpha = 732\ kN$$

$$V_{n@MCE} = C_d \sin\theta + (1.1 + 1.1) A_{vd} f_{yd,actual} \sin\alpha = 1160\ kN$$

鋼筋混凝土中短連接梁之設計

黃世建 教授
宋羽 專任助理
蔡仁傑 博士後研究員
土木工程學系

國立臺灣大學
National Taiwan University

大綱

- 前言
- 傳力機制與強度預測
- ACI 318-25規範討論
- 鋼筋混合配置之建議
- 中短連接梁之設計建議
- 設計案例
- 結論

前言

Coupled Wall System – Kabuki Hotel @ Japanese Town, SF, California

國家實驗研究院
國家地震工程研究中心

NTU CEER

鋼筋混凝土框架剪力牆與地梁之剪力強度設計講習會

時間		講題	主講人	主持人
114年7月25日(五)	13:30～13:50	報到		
	13:50～14:00	致詞	歐昱辰	黃尹男
	14:00～14:50	框架結構牆之剪力強度設計	黃世建	
	14:50～15:40	框架非結構牆之損傷控制		
	15:40～16:00	休息與茶點		
	16:00～16:50	基礎地梁之剪力強度設計	黃世建	林瑞良
	16:50～17:10	綜合座談		

- 講義資料下載

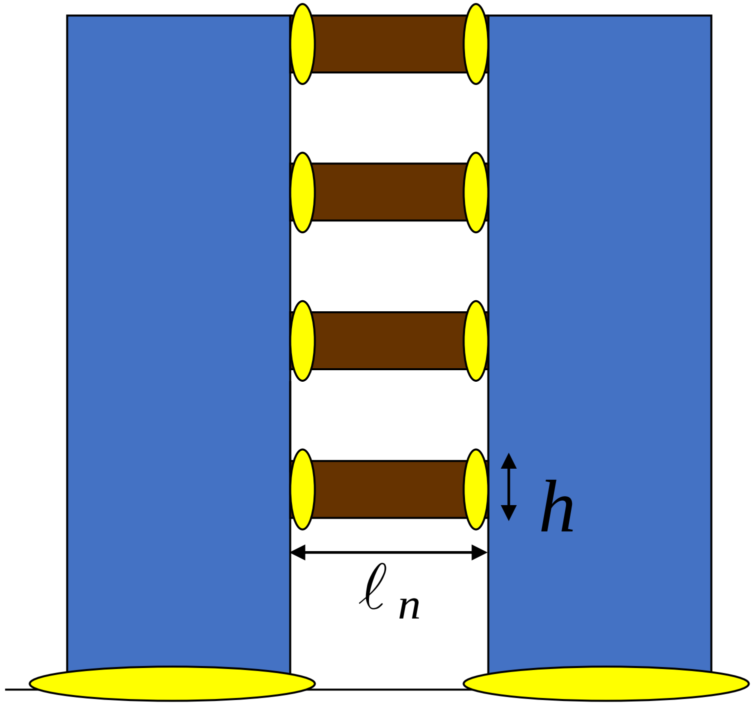
https://conf.ncree.org.tw/download/0-A1140725-會議資料R1.zip

- 錄影檔觀看

https://youtu.be/0bc1ixYgVsg

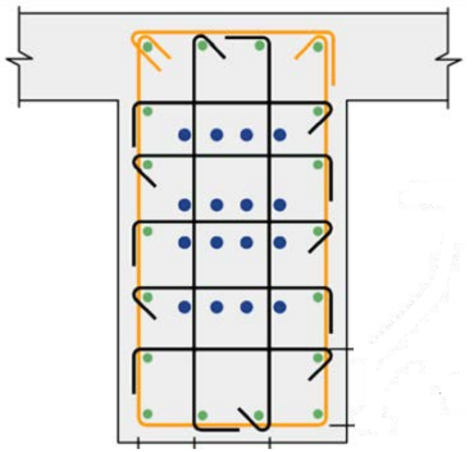
連接梁之耐震設計理念

講習宗旨：
中短連接梁剪力設計之探討



- 設計梁兩端產生撓曲塑鉸

$$\phi V_{n@DBE} \geq \frac{2M_n}{\ell_n}$$



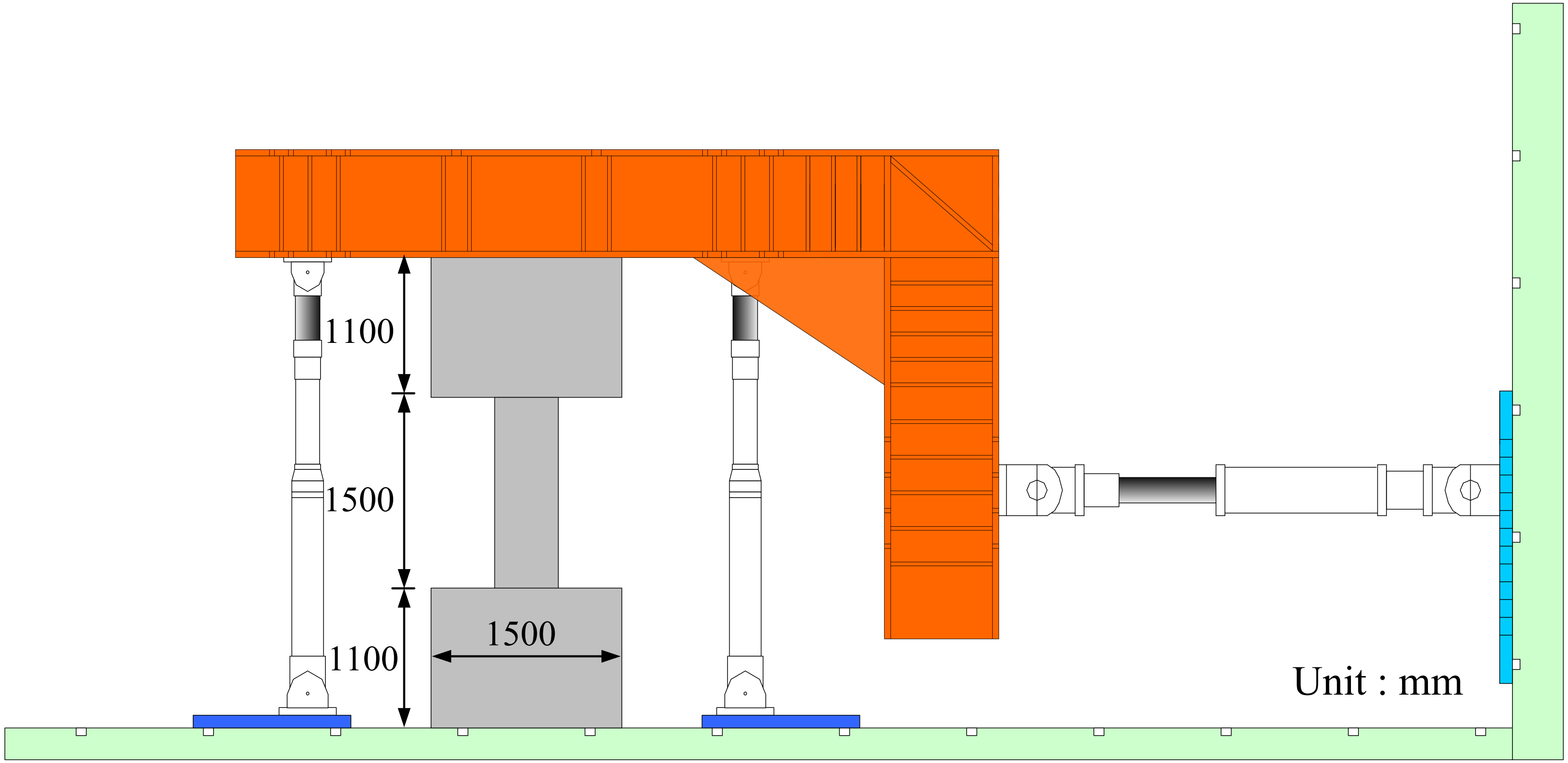
- 緊密箍筋配置維持塑鉸消能

- 全面排除剪力破壞

$$\phi V_{n@MCE} \geq \frac{2M_{pr}}{\ell_n}$$

測試布置

$$\frac{\ell_n}{h} = 3.0$$

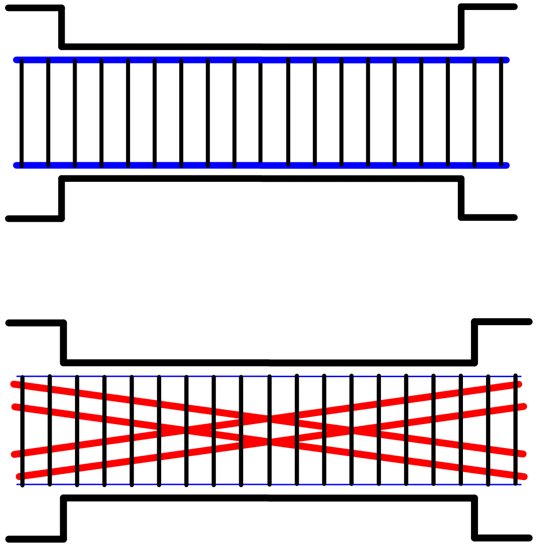


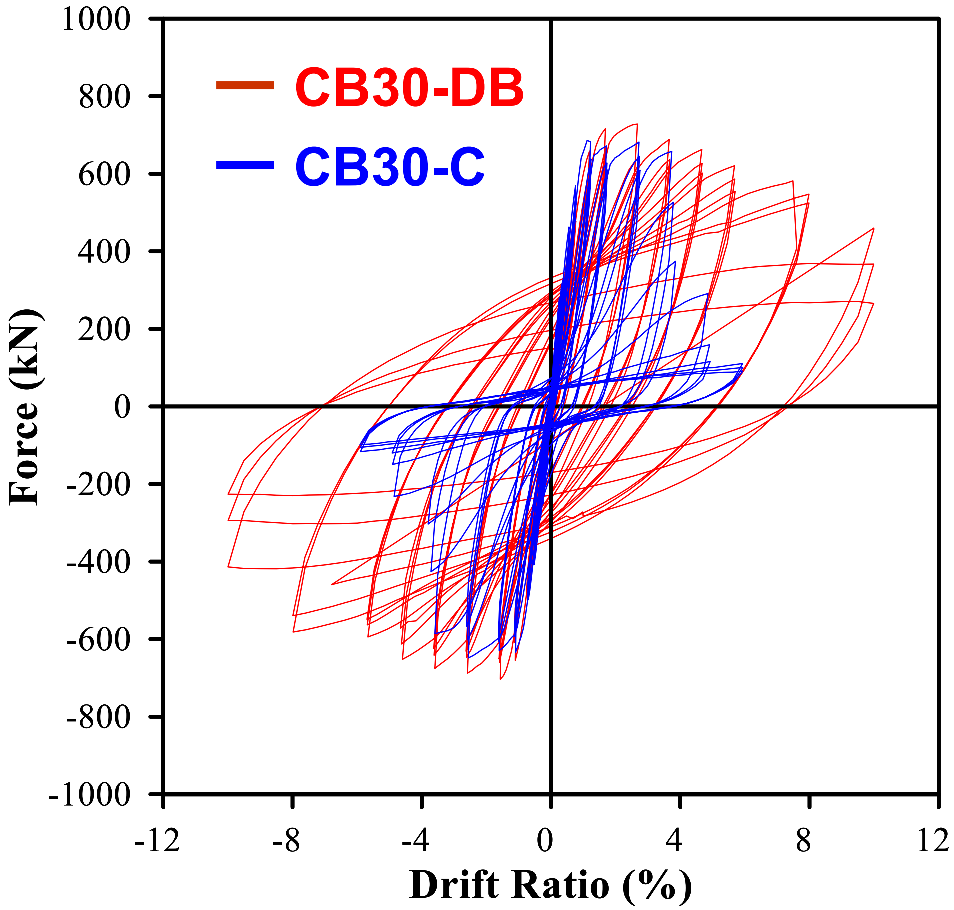
Lim, E., Hwang, S. J., Cheng, C. H., and Lin, P. Y. (2016). "Cyclic Tests of Reinforced Concrete Coupling Beam with Intermediate Span-to-Depth Ratio," ACI Structural Journal, V. 113, No. 3, May-June, pp. 515-524.

行為比較

CB30-C

CB30-DB



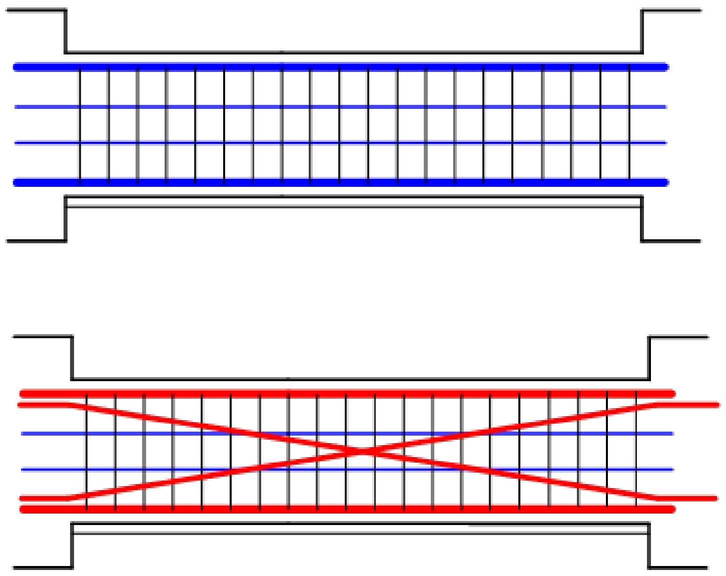


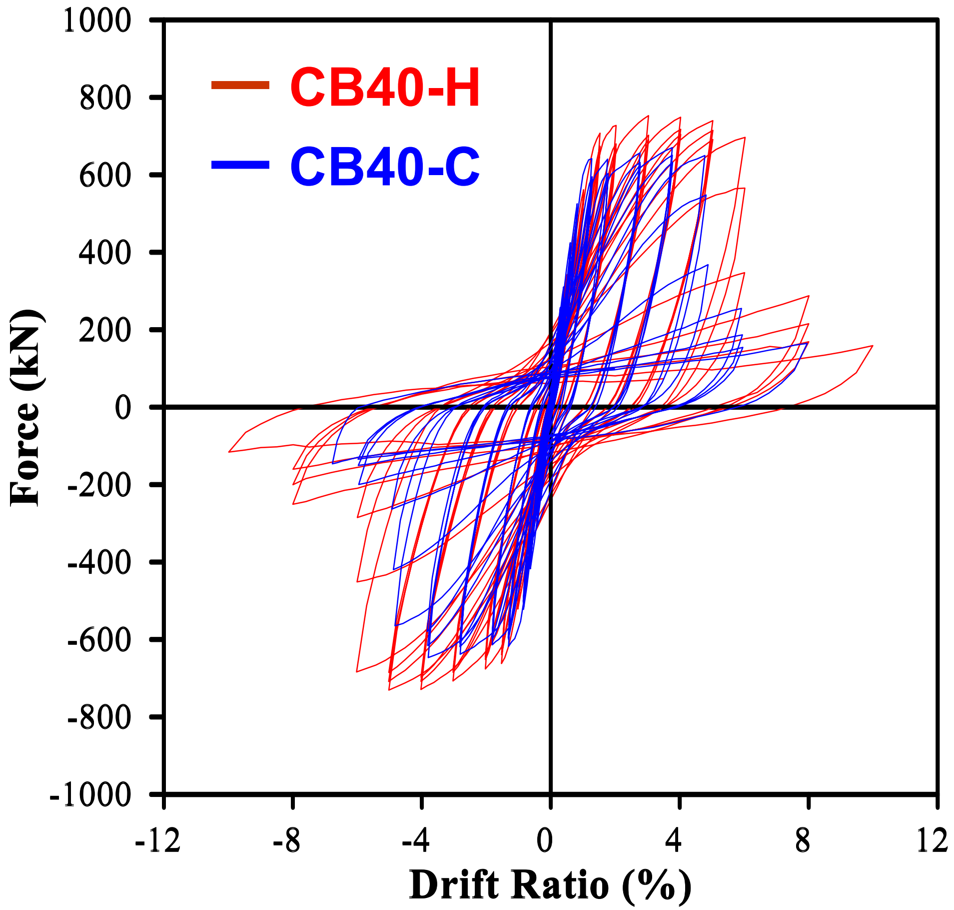
	Failure Mode	Ult. DR	Qual. DR	$\upsilon_{\max}[MPa]$
CB30-DB	Flexure	7.4%	7.9%	$1.04\sqrt{f_c'}$
CB30-C	Flexural Shear	4.1%	2.8%	$0.75\sqrt{f_c'}$

行為比較

CB40-C

CB40-H



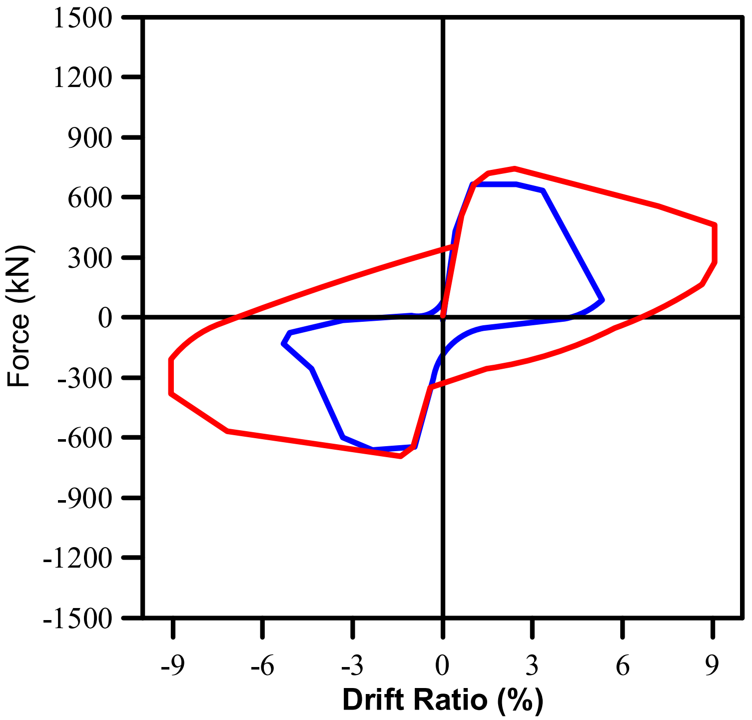


	Failure Mode	Ult. DR	Qual. DR	$\upsilon_{max}[MPa]$
CB40-H	Flexure	6.7%	5.9%	$0.71\sqrt{f_c'}$
CB40-C	Flexure	5.3%	3.9%	$0.62\sqrt{f_c'}$

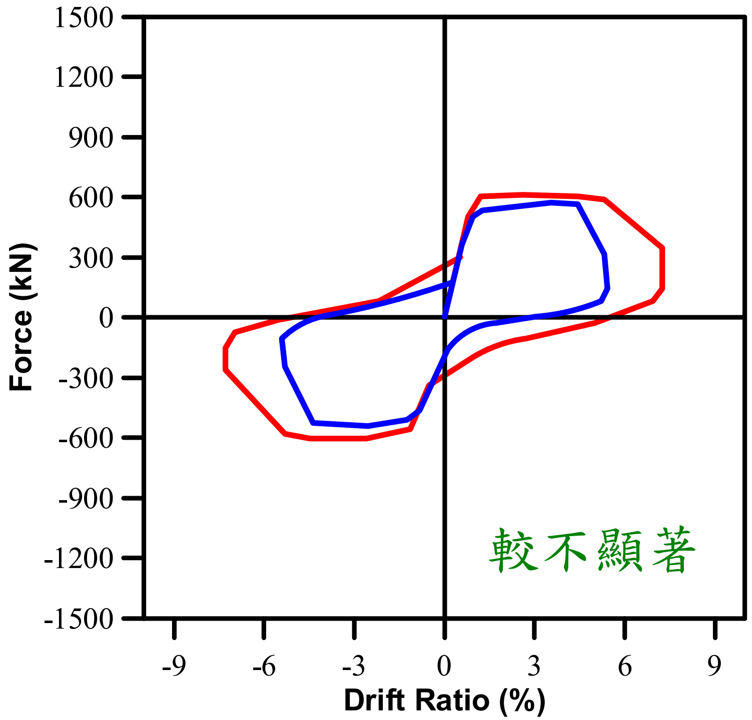
對角鋼筋之功能

— 對角鋼筋
— 傳統配筋

$$\frac{\ell_n}{h} = 3.0$$

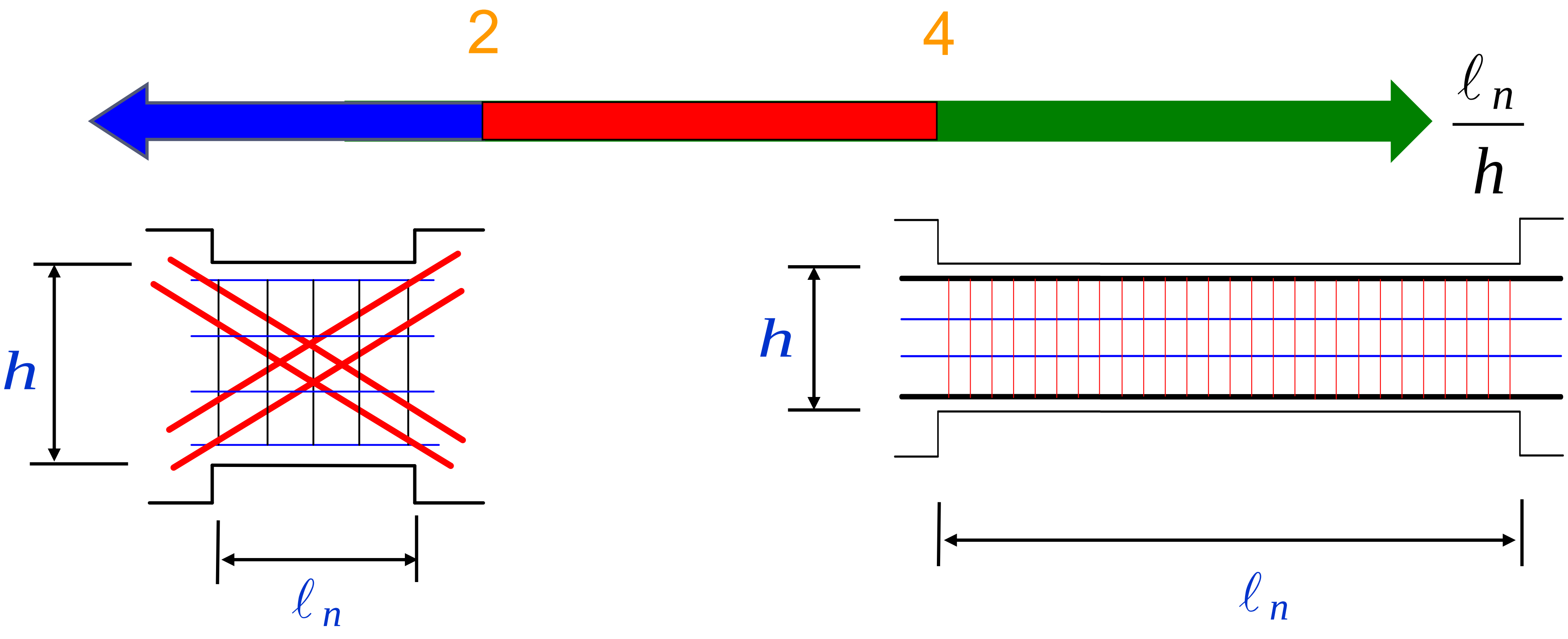


$$\frac{\ell_n}{h} = 4.0$$



- 避免剪力束縮效應
- 提高剪力強度
- 增加變形能力

土木401-112規範



應配置對跨度中點對稱之兩組對角向鋼筋

得配置對角鋼筋

不可配置對角鋼筋

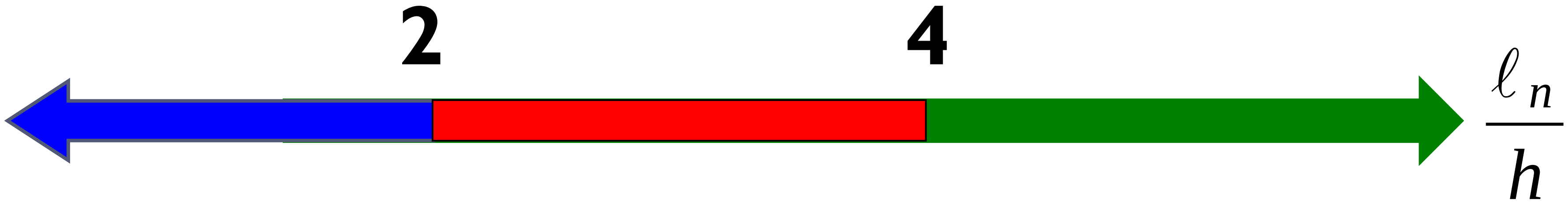
因對角鋼筋傾斜角度較小，故抗剪能力差

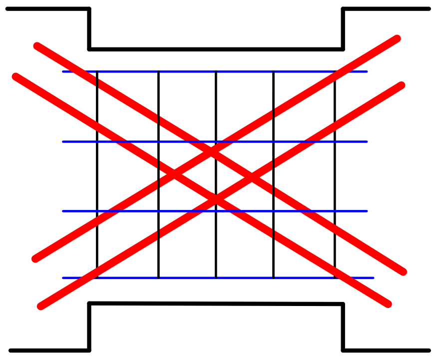
中短連接梁之設計方法尚待開發

一般連接梁統計： $2.4 \le \dfrac{\ell_n}{h} \le 3.3$

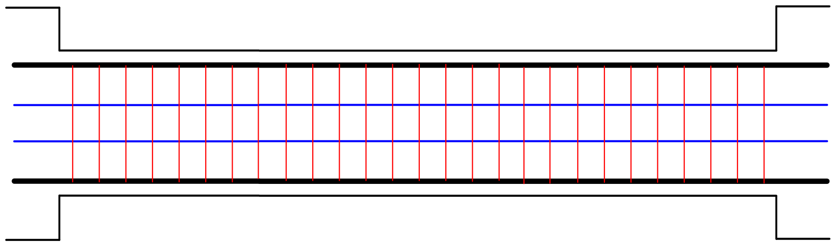
(Naish et al. 2013)

- 住宅大樓 $\dfrac{\ell_n}{h} = 2.4$
- 辦公廳舍 $\dfrac{\ell_n}{h} = 3.3$

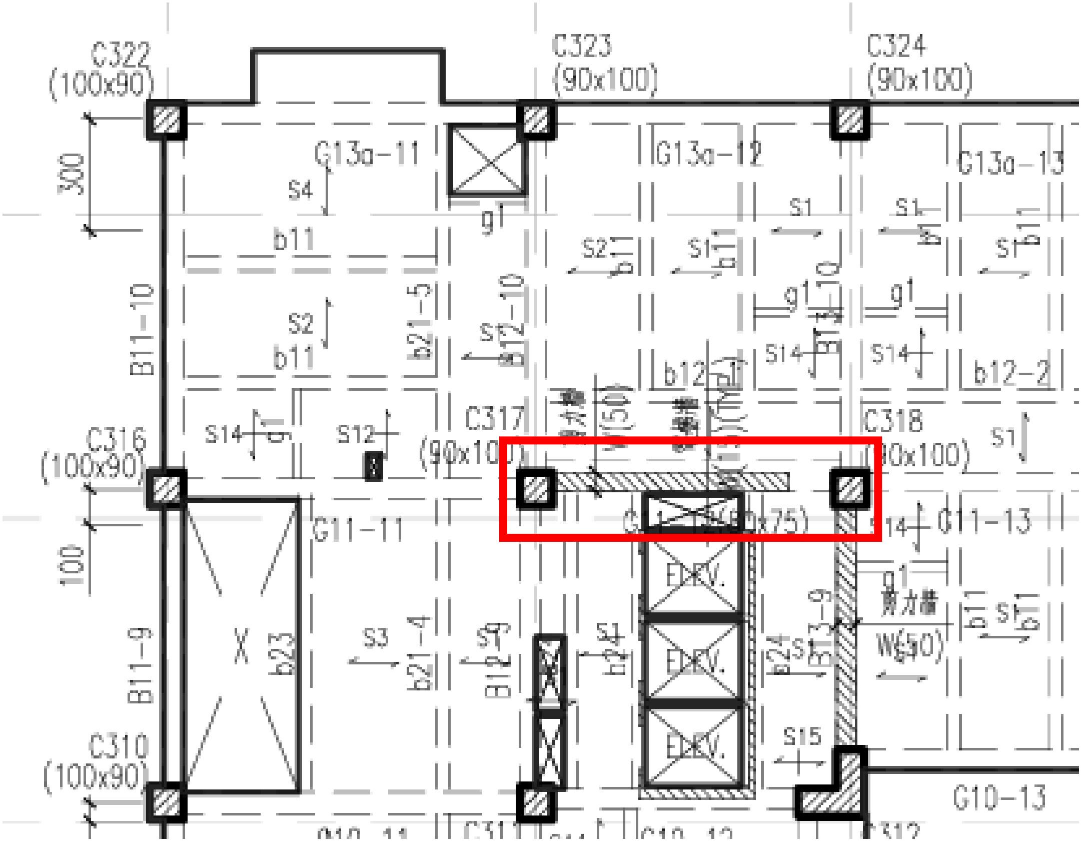


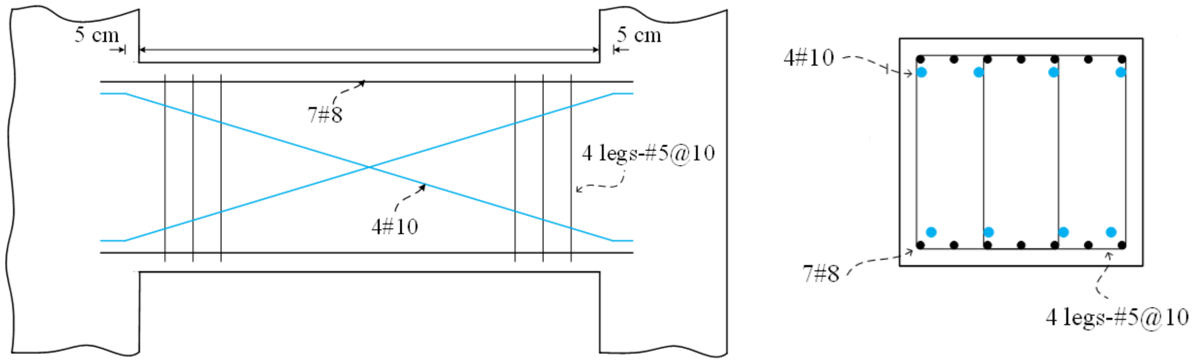


為數最多，
但設計方法
不明確 ??



非結構RC牆所造成之中短梁亦適用





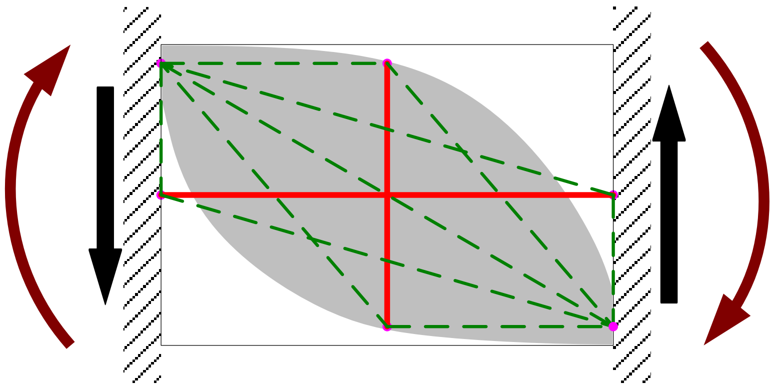
- 突破ACI最大剪力強度之限制
- 增加梁之剪力強度與轉動能力

傳力機制與強度預測

傳統配筋梁

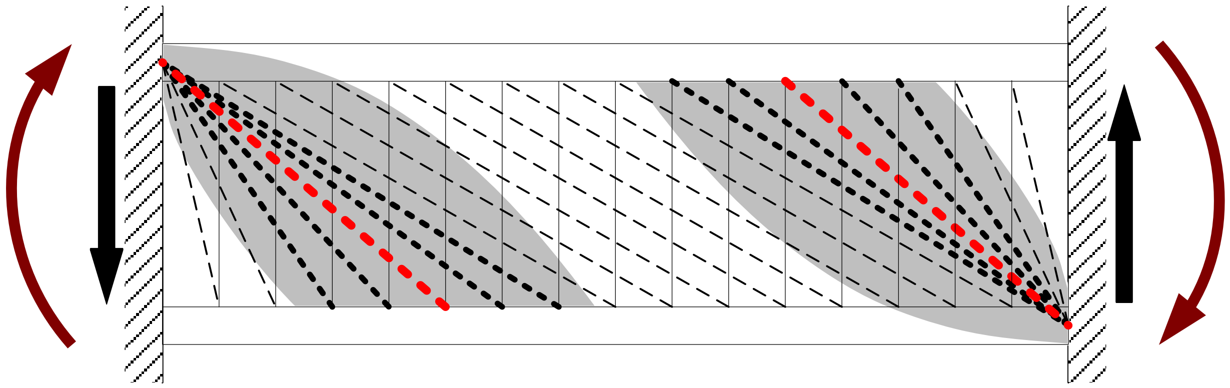
梁剪力傳遞機制

單一D-區域構件



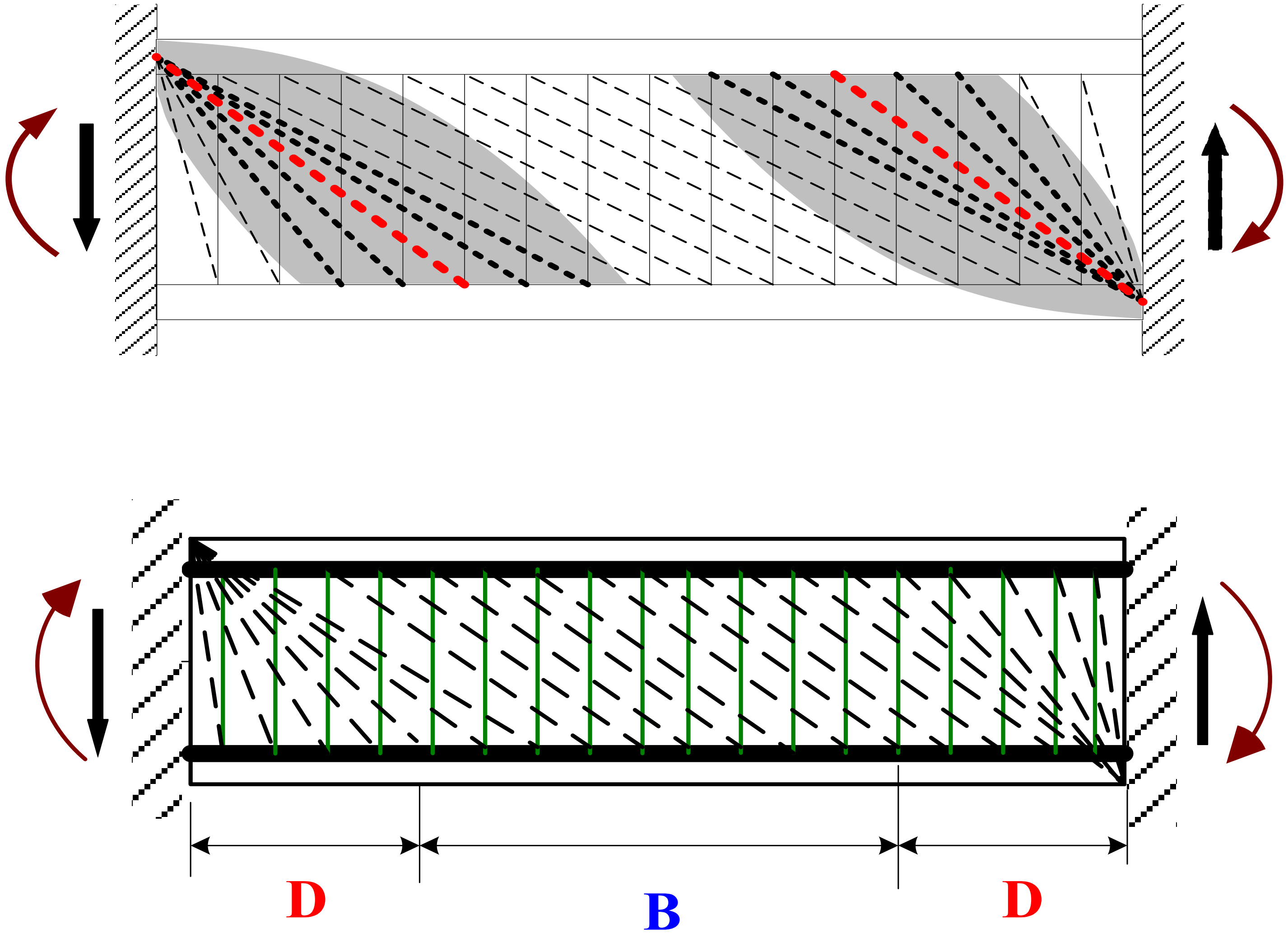
瓶狀壓桿

DBD-區域構件



扇形壓桿

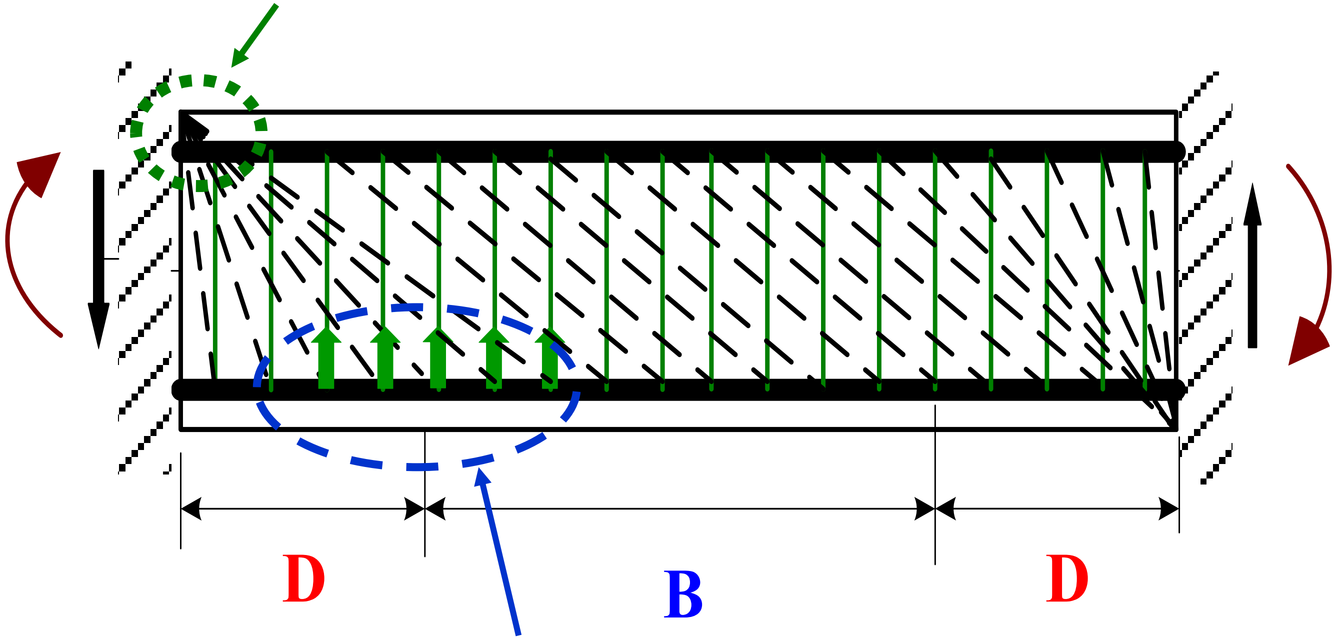
剪力傳遞機制 - DBD 梁



- D-區域
- 應力擾動區
- 對角應力流
- 扇形壓桿
- 混凝土壓桿位於裂縫之間

DBD 梁之破壞模式

壓桿端部應力集中處 → 混凝土擠碎 → 剪壓破壞 → $V_{n,c}$

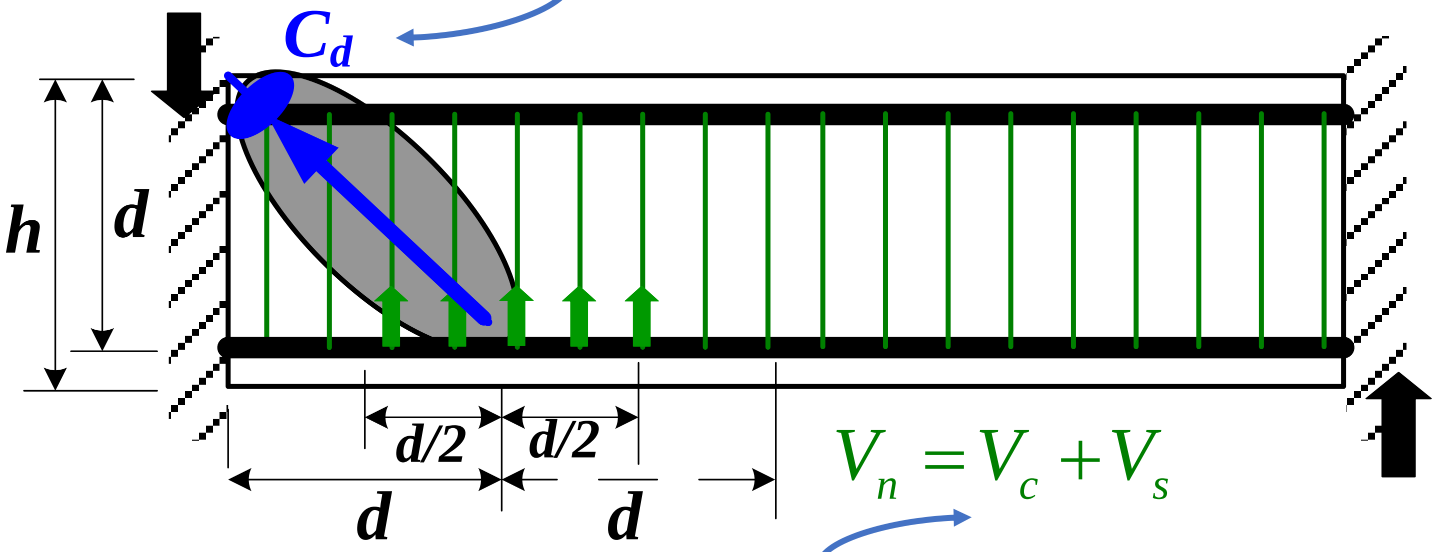


扇形壓桿力量分散處 → 內部拉力支承 → 剪拉破壞 → $V_{n,t}$

$$V_n = smaller(V_{n,c}\ ,\ V_{n,t})$$

扇形壓桿之剪力強度

- 剪壓破壞→ 混凝土擠碎→ $V_{n,c}$



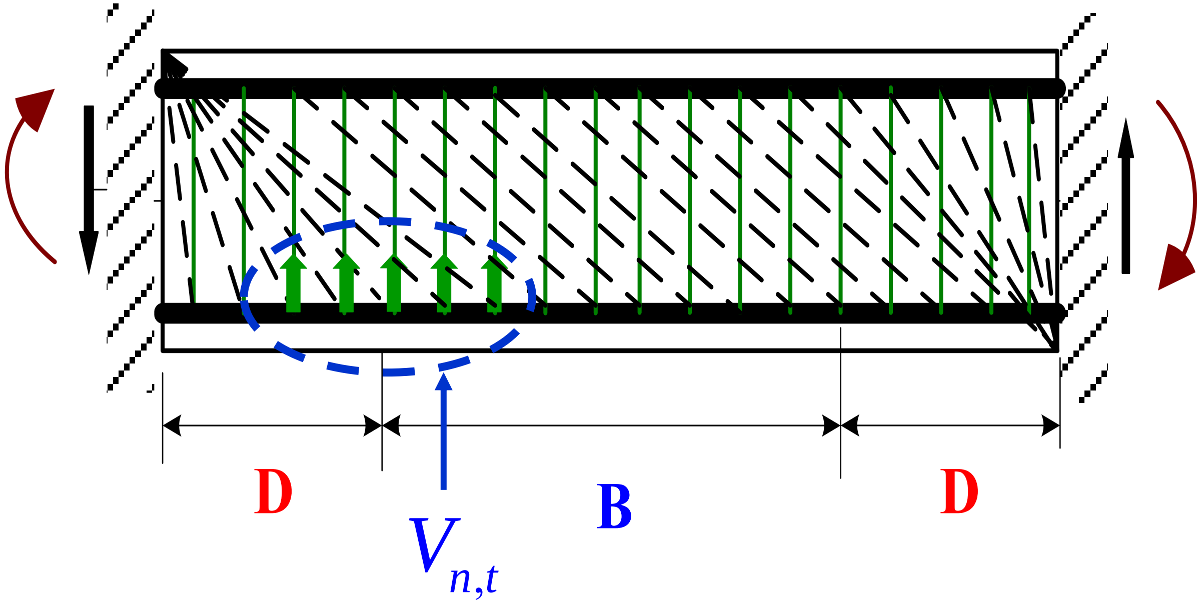
$V_n = V_c + V_s$

$V_n =$ smaller of $(V_{n,c}; V_{n,t})$

- 剪拉破壞→ 橫向鋼筋降伏→ $V_{n,t}$

Hwang, S. J., Yang, Y. H., and Li, Y. A. (2022). "Maximum Shear Strength of Reinforced Concrete Beams," ACI Structural Journal V. 119, No. 2, March, pp. 19-20.

梁剪拉破壞之強度 – ACI 318 Code

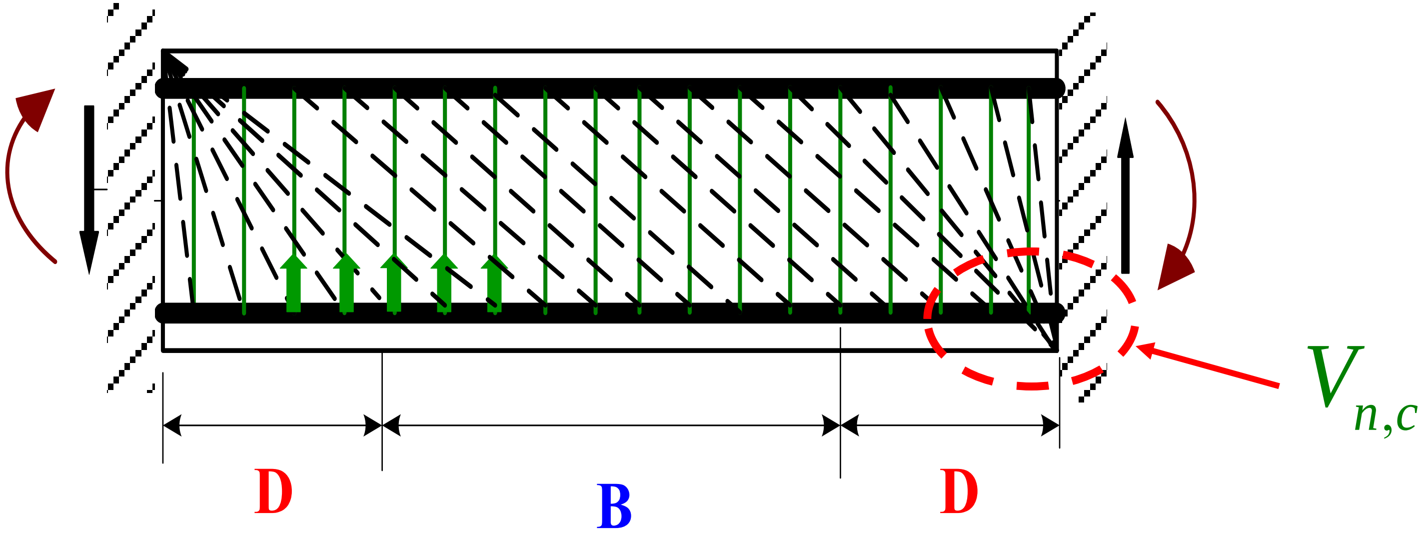


$$V_{n,t} = V_c + V_s = 0.66\lambda_s \rho_w^{1/3}\sqrt{f_c'} \times b_w d + \frac{A_v f_{yt} d}{s} \quad \text{(MPa)}$$

$$\lambda_s = \sqrt{\frac{2}{1+\frac{d}{254}}} \le 1 \quad \text{(mm)}$$

若梁配置最小剪力鋼筋量，λ_s 可取1.0。

梁剪壓破壞之強度 – 軟化壓拉桿模型(SST Model)

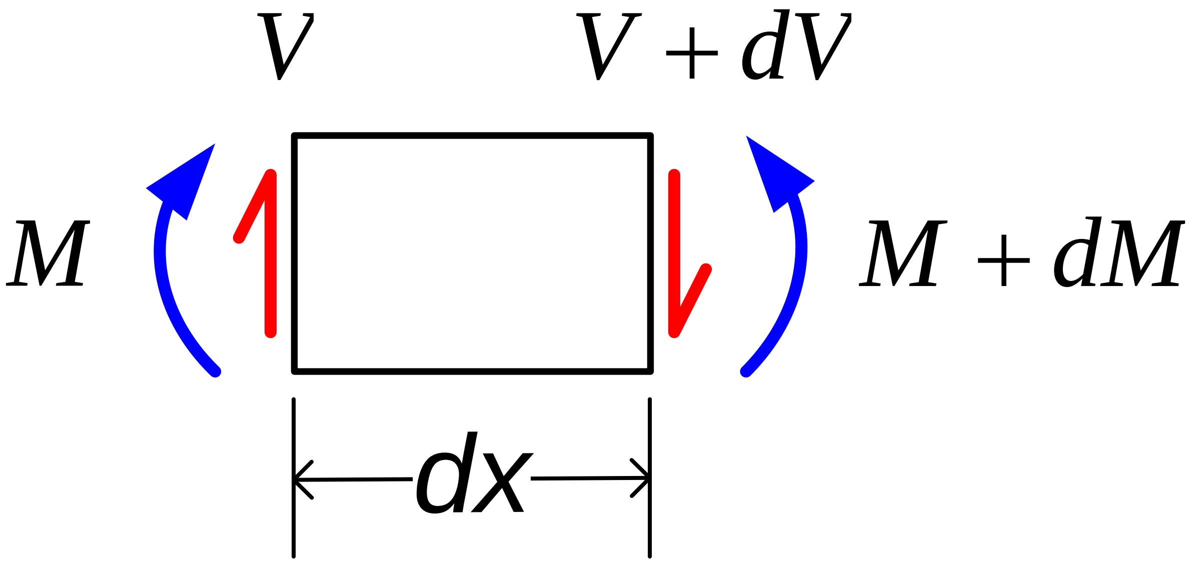


$$V_{n,c} = K \times \zeta \times f_c' \times A_{str} \times \sin\theta$$

$$K = \tan^A\theta + \cot^A\theta - 1 + 0.14B \le 1.64$$

$$A = 12\frac{f_y}{f_c'}\rho \le 1,\ B = 30\frac{f_y}{f_c'}\rho \le 1$$

彎矩與剪力交互影響



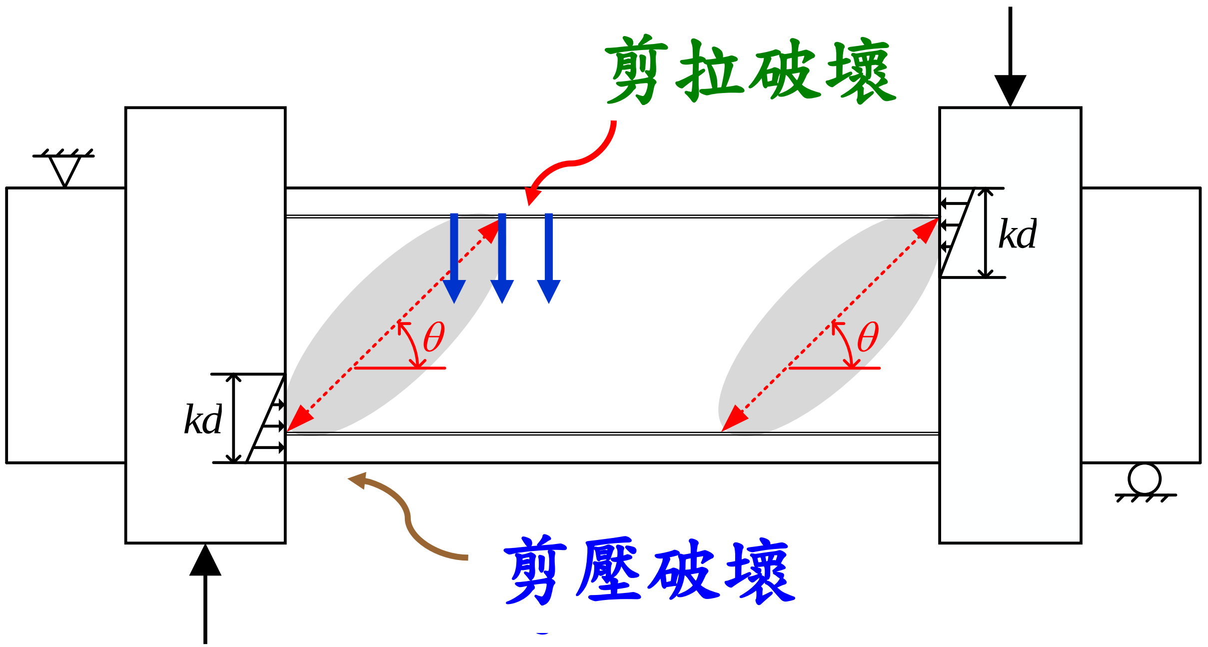
$$V = \frac{dM}{dx}$$

剪力設計必須考慮對應之彎矩行為

剪力強度評估 – Design Based Earthquake (DBE)

$$V_{n,t} = V_c + V_s;\ V_c \neq 0$$

微量開裂，忽略混凝土劣化



彈性壓力區深度kd

$$A_{str} = b_w \times kd$$

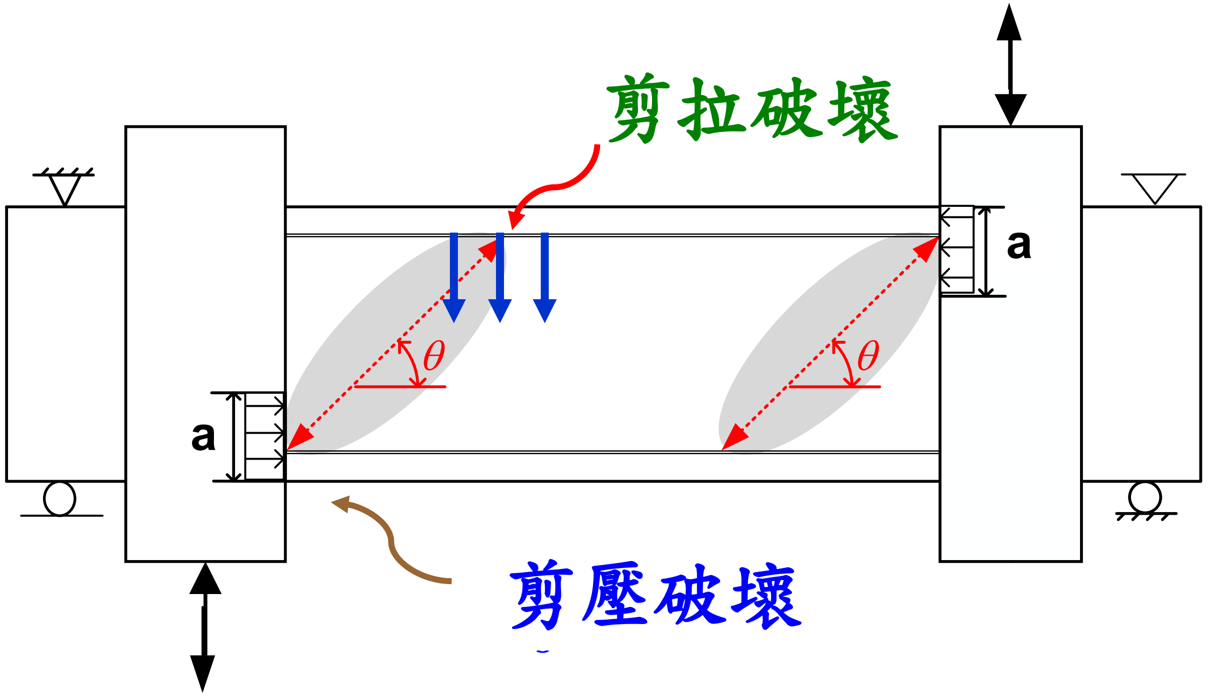
$$kd = \left(\sqrt{(\rho_f n)^2 + 2\rho_f n} - \rho_f n\right) d$$

$$V_{n,c} = K \times \zeta f_c' \times b_w \times kd \times \sin\theta$$

剪力強度評估 – Maximum Credible Earthquake (MCE)

$V_{n,t} = V_s$; $V_c = 0$

大量開裂，考慮混凝土劣化



塑性壓力區深度a；

$f_s = 1.25 f_y$

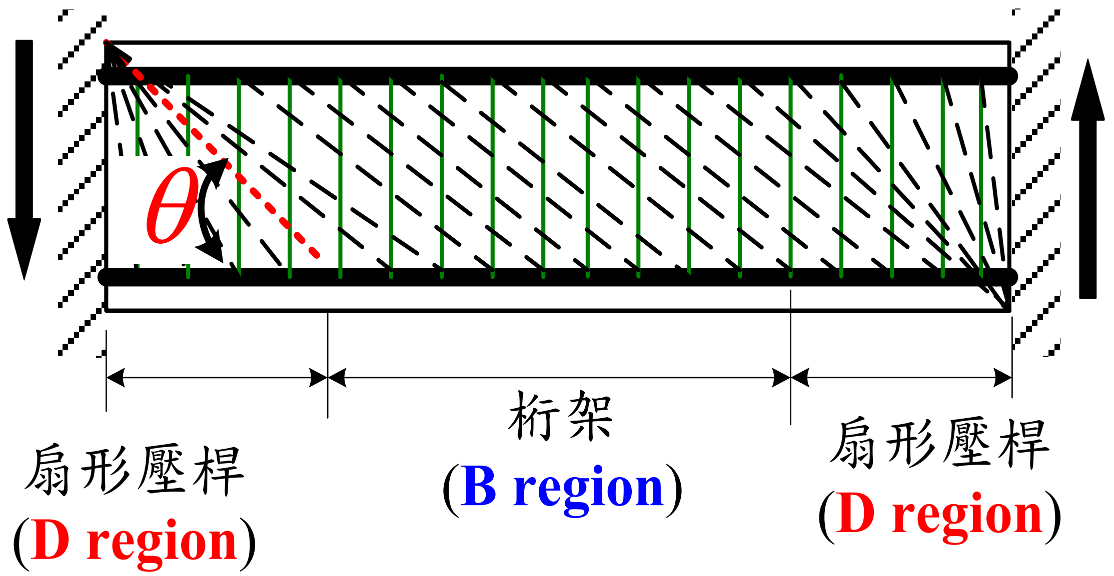
$A_{str} = b_w \times a$

$V_{n,c} = K \times \zeta f_c' \times b_w \times a \times \sin\theta$

$$a = \frac{1.25 f_y \times A_s}{0.85 f_c' \times b_w}$$

剪力鋼筋維持彈性之束制效應

梁無軸力作用，初始開裂角度 $\theta = 45^o$ → 強度點之裂縫角度，由於彈性鋼筋束制，故約略維持初始開裂角度 $\theta = 45^o$

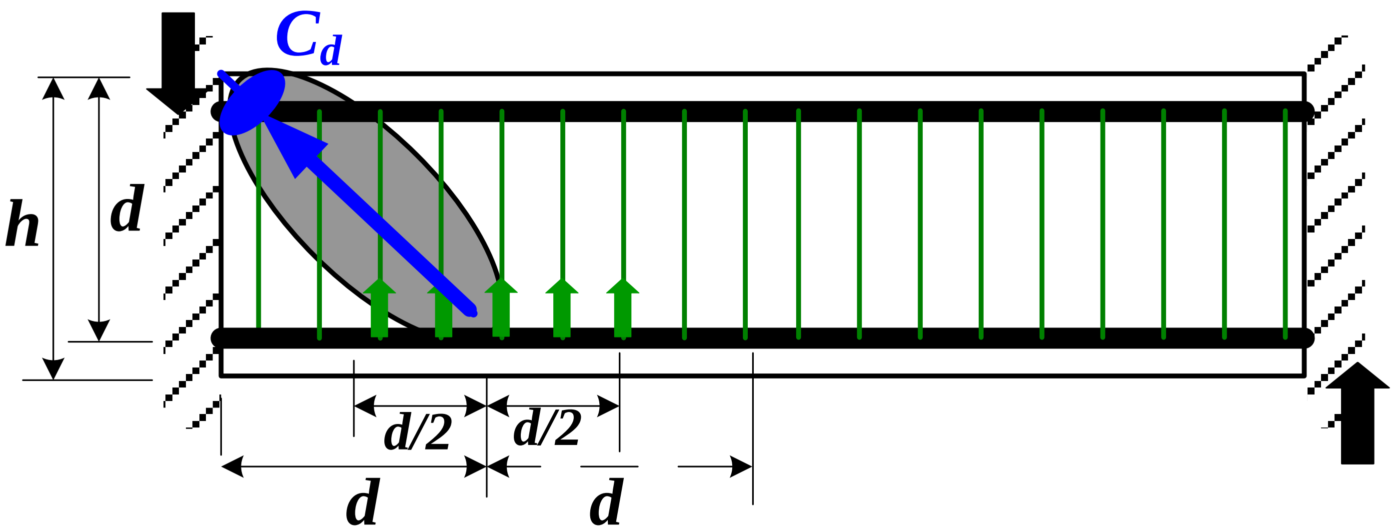


- 梁壓桿傾斜角θ介於25°與45°之間
- $\theta = 45°$可得最大之壓桿強度，V_{max}

Hwang, S. J., Yang, Y. H., and Li, Y. A. (2022). "Maximum Shear Strength of Reinforced Concrete Beams," ACI Structural Journal V. 119, No. 2, March, pp. 19-20.

確保壓桿在MCE時發展最大抗壓強度之條件

提供充足的剪力鋼筋，剪力鋼筋維持彈性，確保$\theta = 45°$

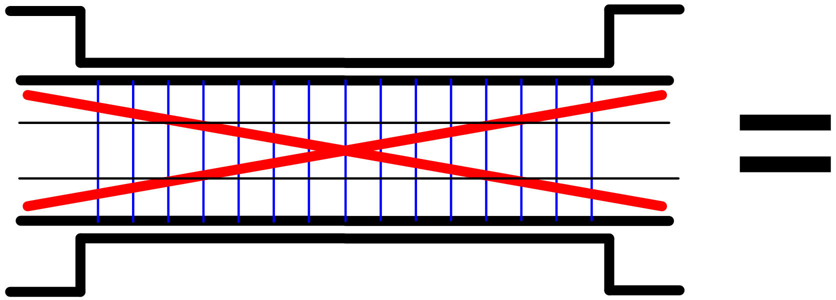


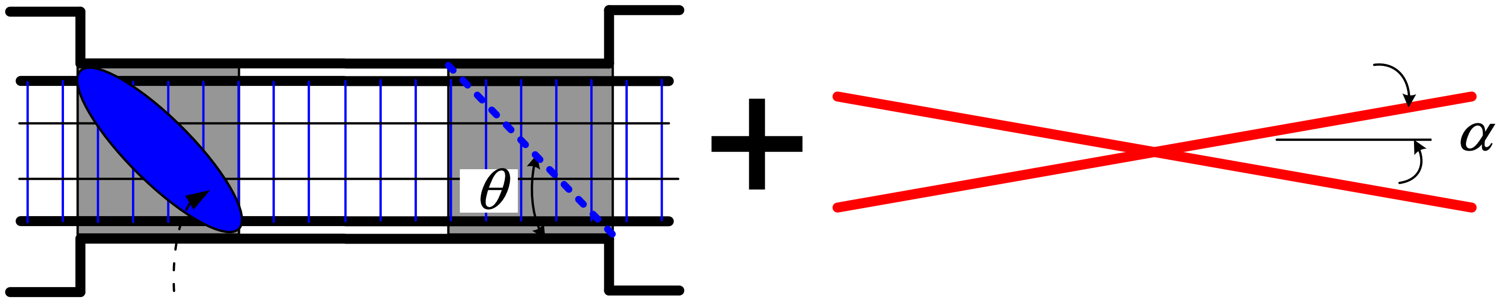
MCE作用下取$V_c = 0$，故需：$V_s = A_{vt} f_{yt} \frac{d}{s} \geq C_d \sin 45°$

剪拉強度足夠，確保壓桿剪壓強度充分發展

對角鋼筋配置梁

對角鋼筋配置連接梁之剪力強度評估



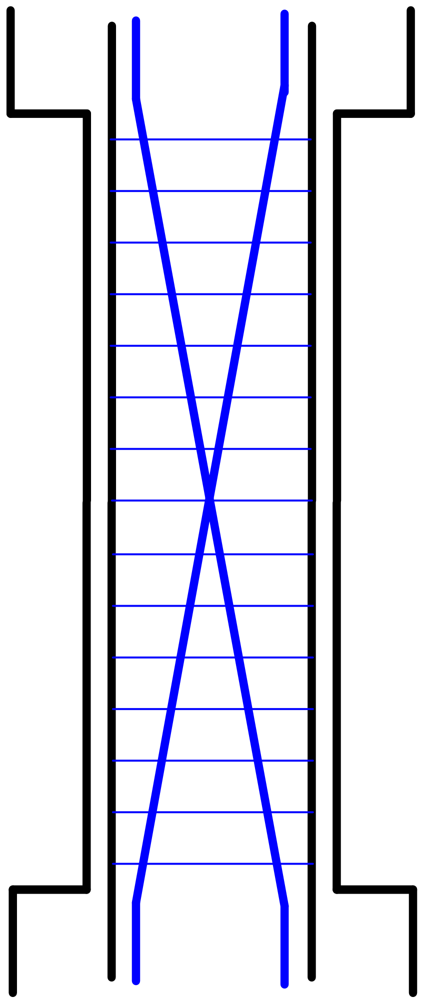


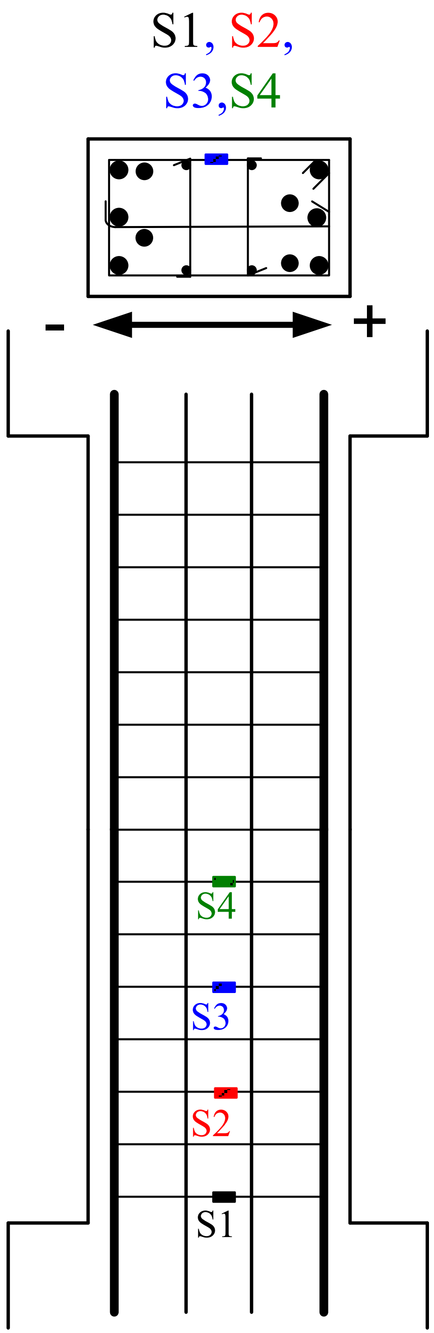
混凝土壓桿 + 對角鋼筋

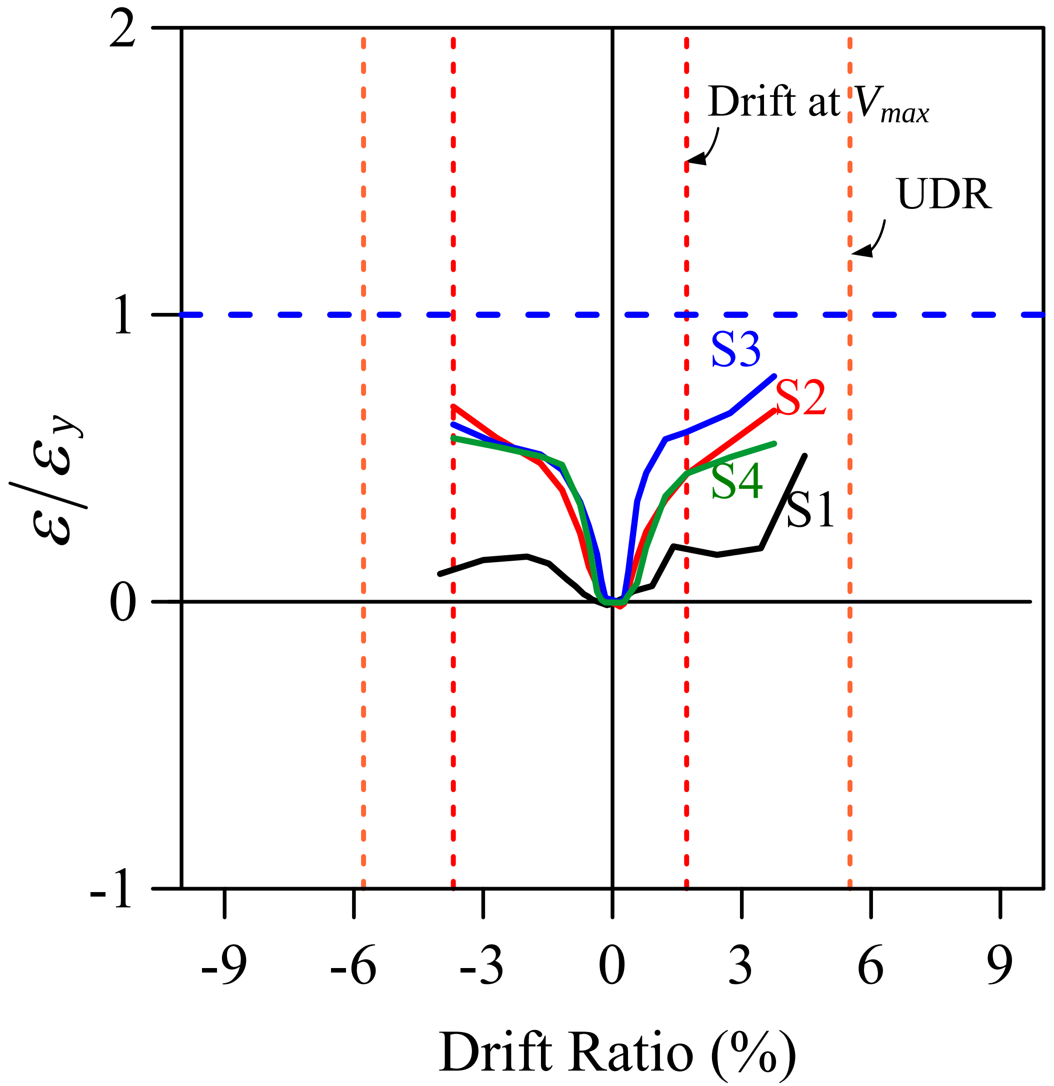
(需足夠內部支承) （應變量測驗證）

CB30-H

彈性箍筋支持壓桿強度充分發展

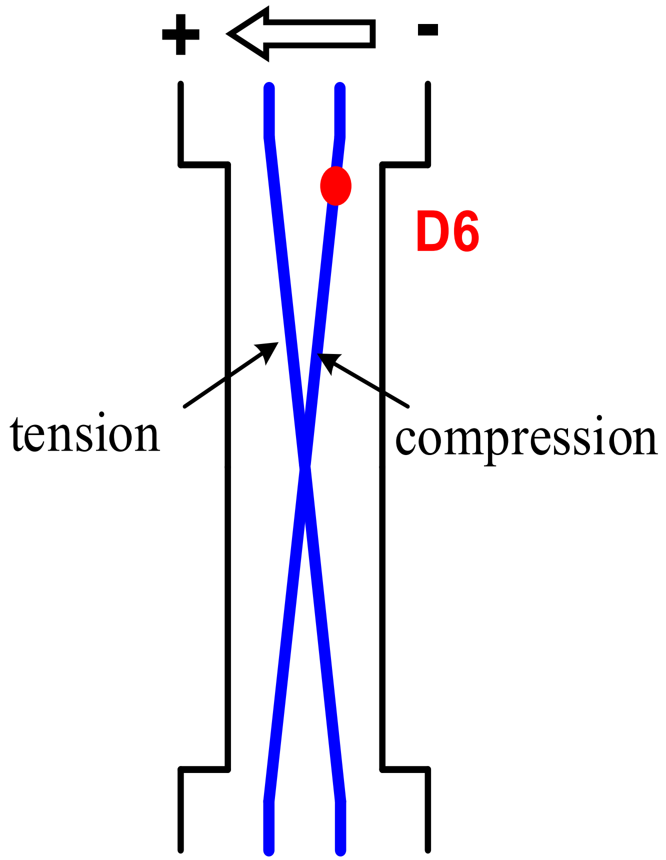


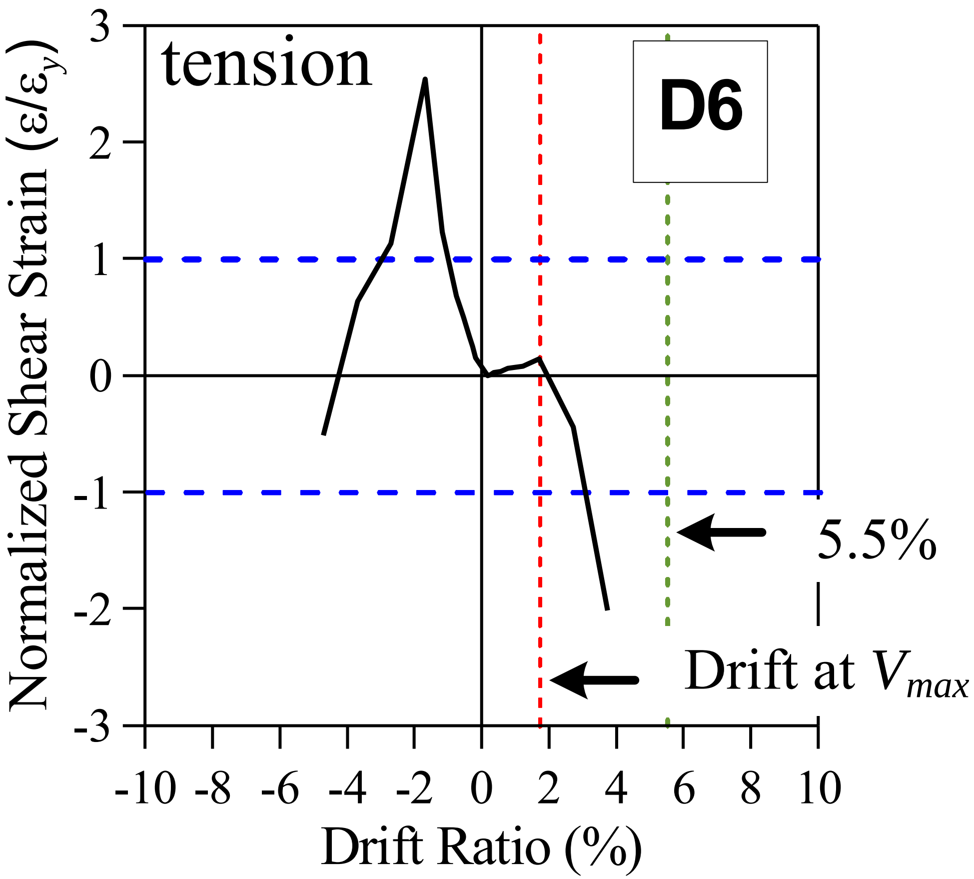




對角鋼筋降伏後之應變硬化

CB30-H



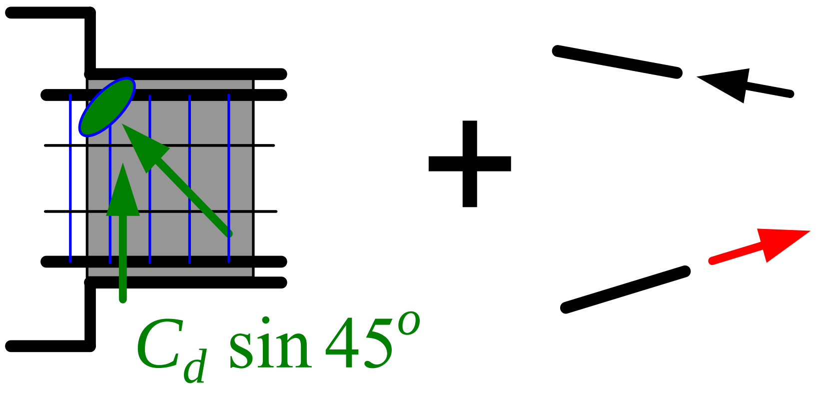


拉力鋼筋與壓力鋼筋均出現應變硬化之現象

需配置對角鋼筋之連接梁

$$C_d \sin 45^o < \frac{V_p}{\phi}$$

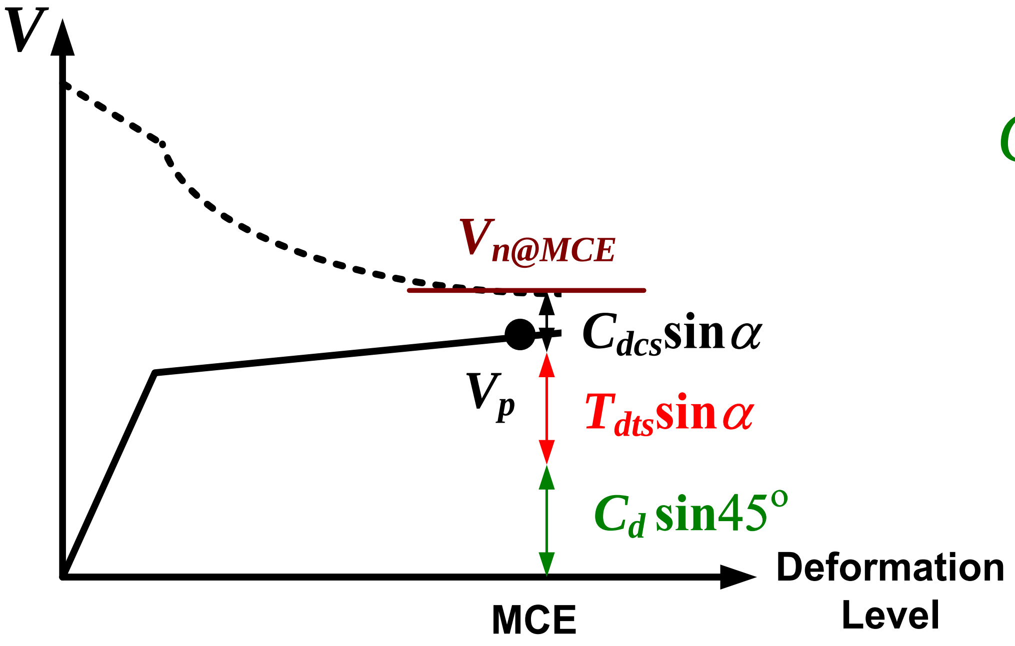
（需配置對角鋼筋）



$C_{dcs} = 1.25 A_{vd} f_y$

$T_{dts} = 1.25 A_{vd} f_y$

塑鉸應變硬化

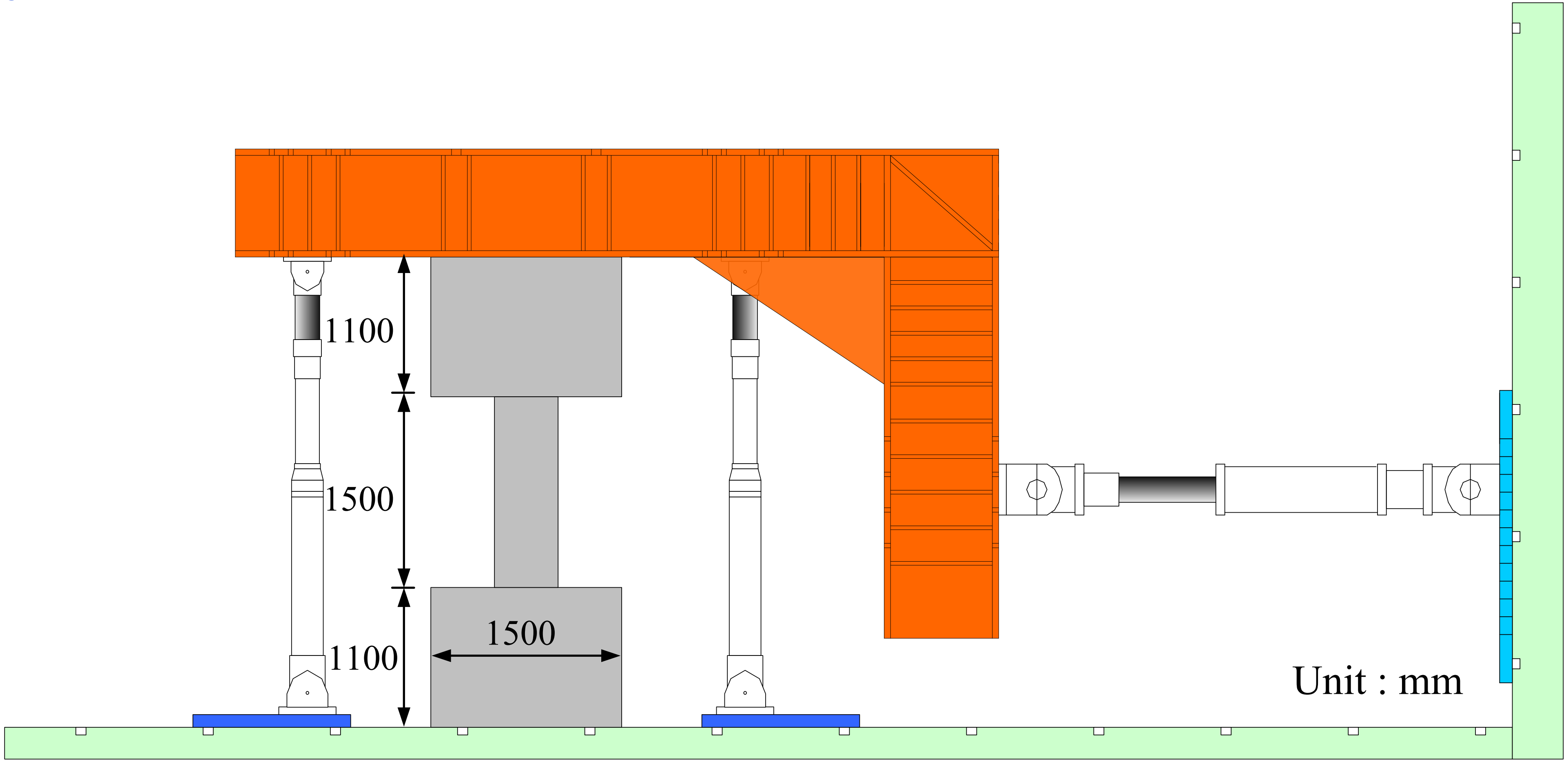


$$C_d \sin 45^o + (1.25 + 1.25) A_{vd} f_y \sin\alpha = \frac{V_p}{\phi}$$

$$\phi = 0.85$$

Solve for: A_{vd}

$$\frac{\ell_n}{h} = 3.0$$



Lim, E., Hwang, S. J., Cheng, C. H., and Lin, P. Y. (2016). “Cyclic Tests of Reinforced Concrete Coupling Beam with Intermediate Span-to-Depth Ratio,” ACI Structural Journal, V. 113, No. 3, May-June, pp. 515-524.

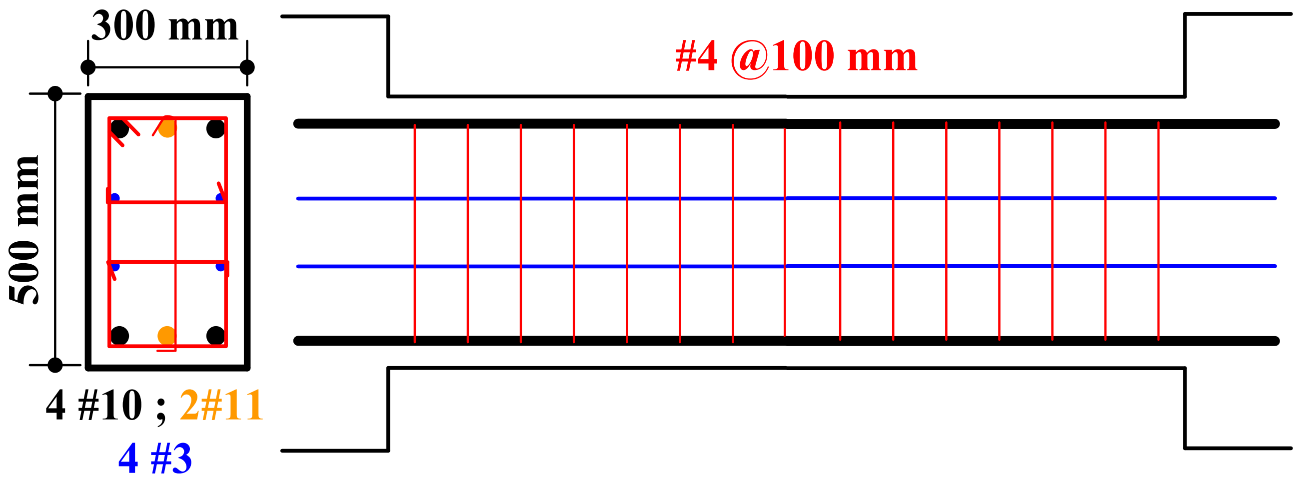
$$\ell_n / h = 3.0$$

實驗試體

CB30-C

$f_c' = 47.9\ MPa$

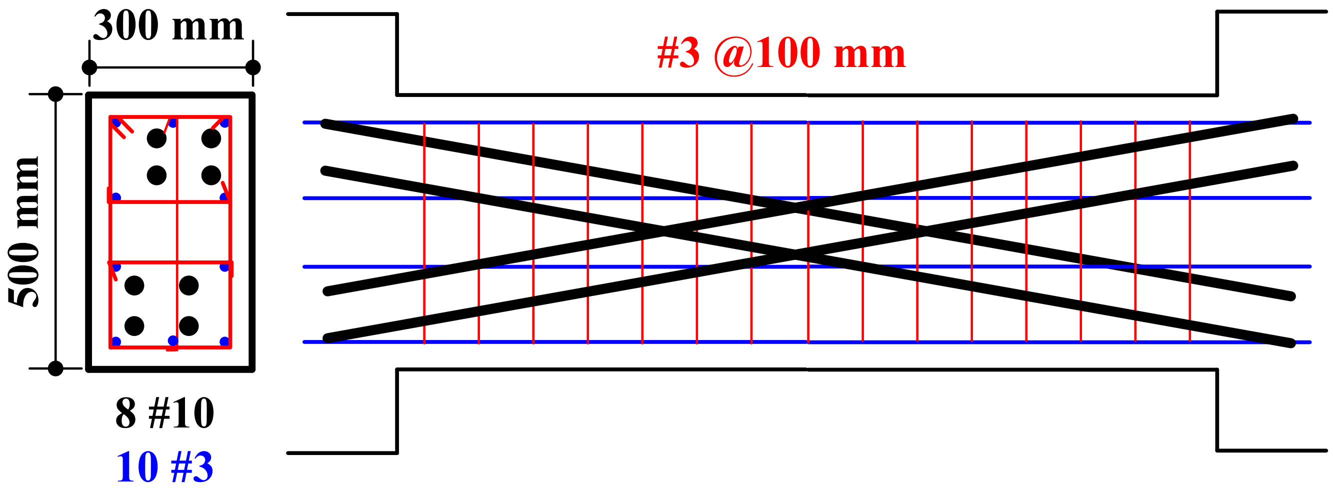
$\rho_f = 2.03\%$



CB30-DB

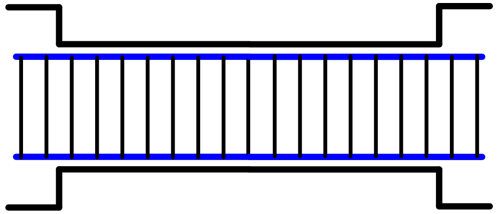
$f_c' = 38.4\ MPa$

$\rho_f = 2.86\%$



試驗結果

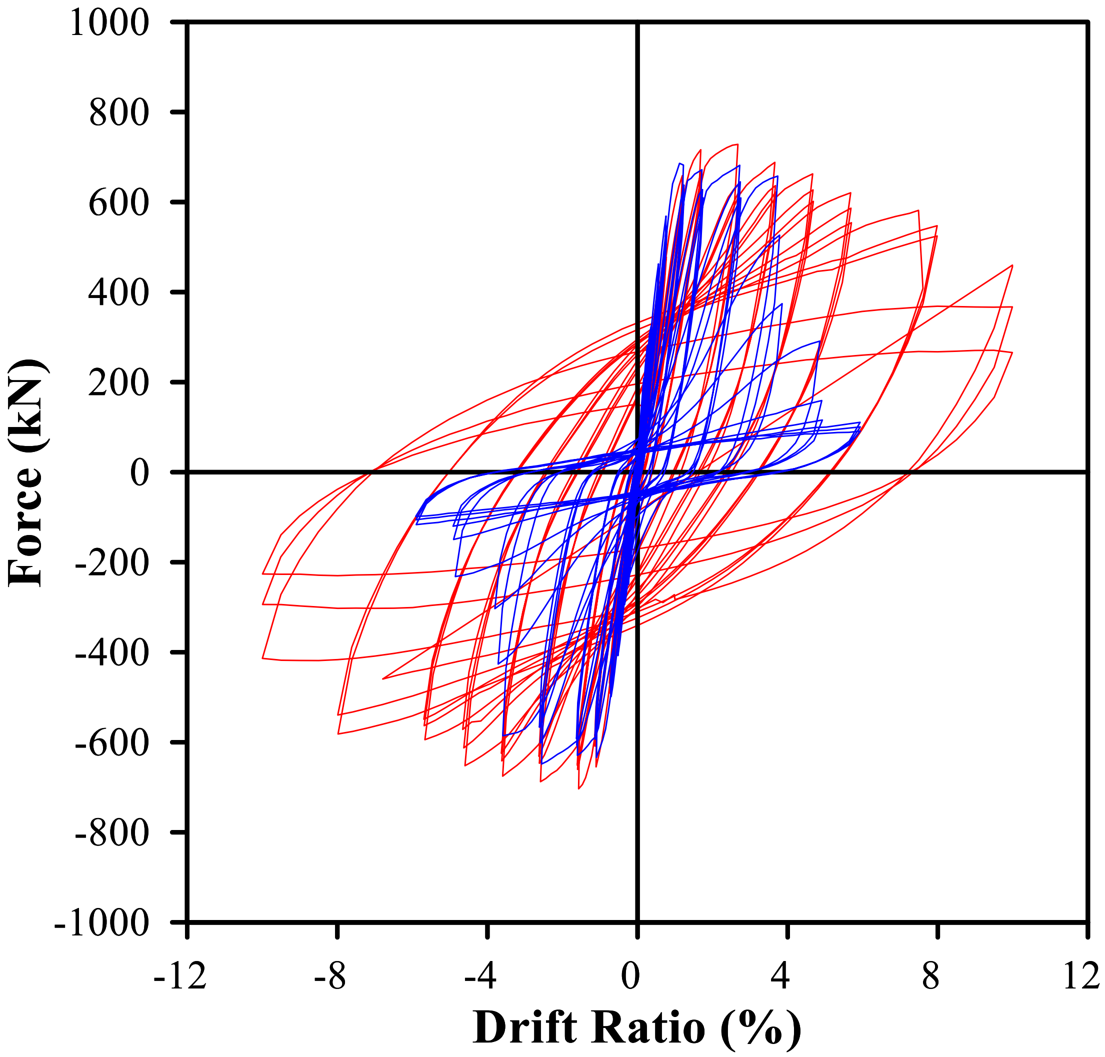
CB30-C



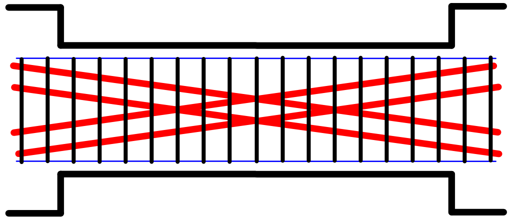
4.0%

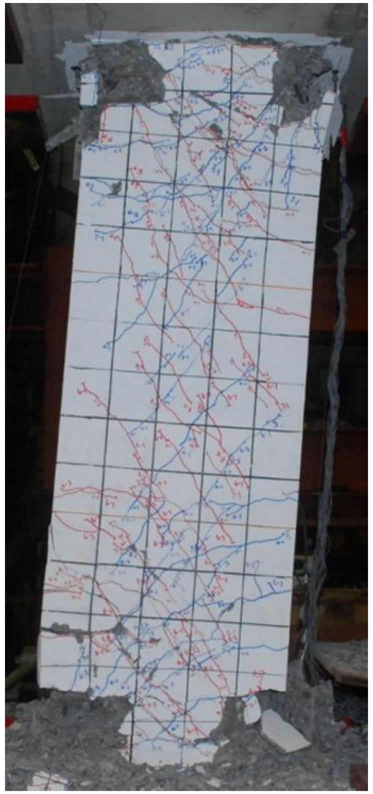
$$\upsilon = 0.75\sqrt{f_c'}$$

UDR = 3.8%



CB30-DB





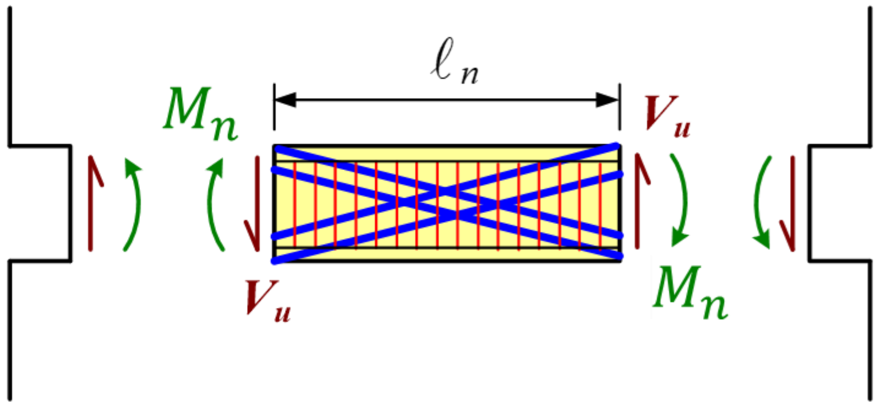
8.0%

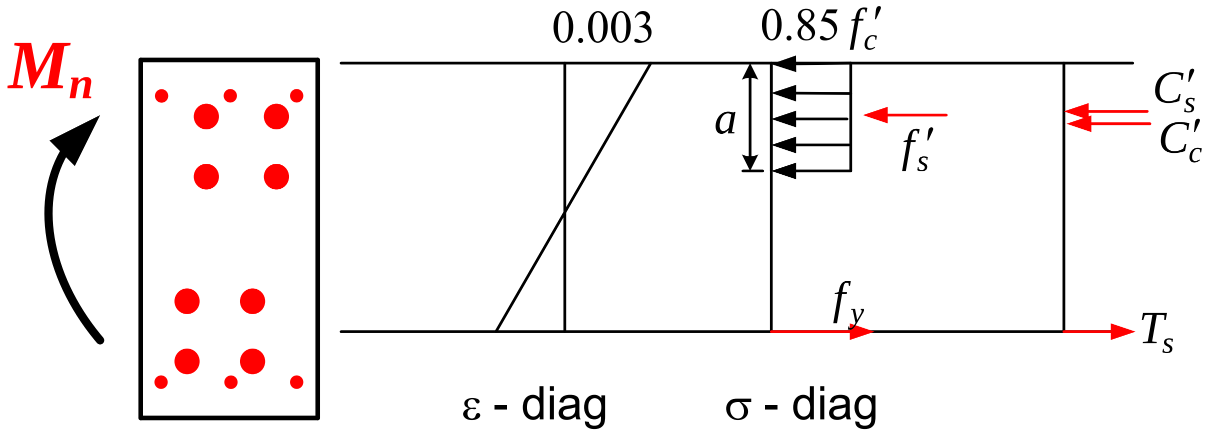
$$\upsilon = 1.04\sqrt{f_c'}$$

UDR = 7.5%

撓曲強度之計算流程

撓曲強度： 由 XTRACT 取得斷面標稱彎矩 $M_n \Rightarrow V_{mn} = \dfrac{2M_n}{\ell_n}$

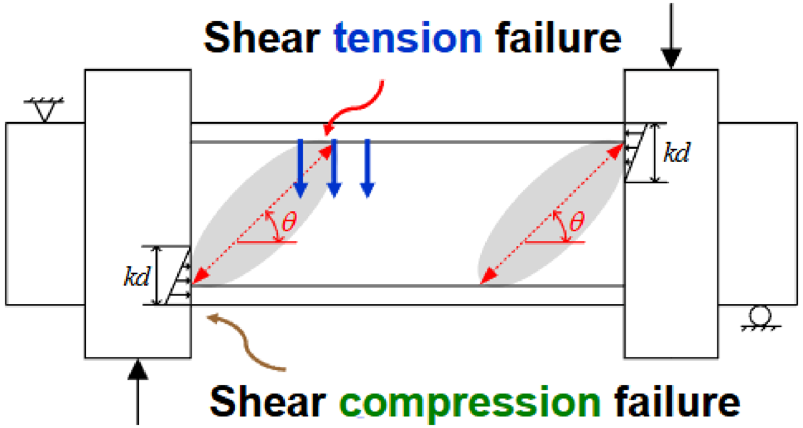




- 斷面分析 - XTRACT
- 平面維持平面

DBE剪力強度之計算流程

Strength (DBE):

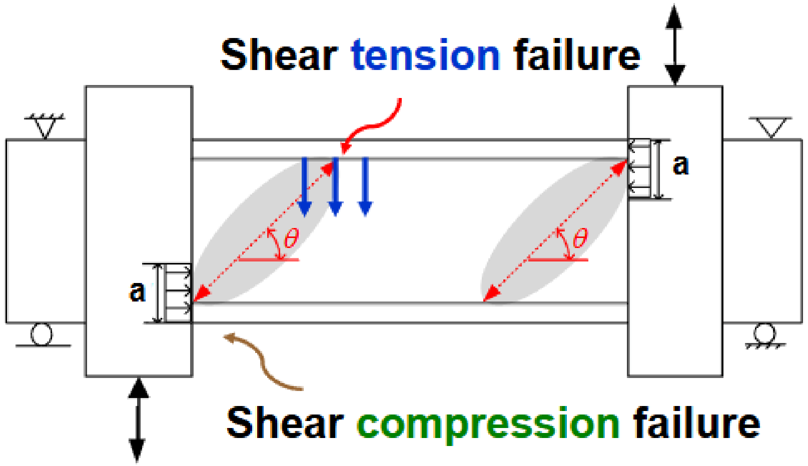


1. 彈性壓力區深度 $kd = \left(\sqrt{(\rho_f n)^2 + 2\rho_f n} - \rho_f n\right) d$
2. 壓拉桿指標 $K = \tan^A \theta + \cot^A \theta - 1 + 0.14B \leq 1.64$
3. 開裂鋼筋混凝土軟化係數 $\zeta = \frac{3.35}{\sqrt{f_c'}} \leq 0.52$
4. 計算剪壓強度 $V_{n,c} = K\zeta f_c' b_w kd \sin\theta$
5. 計算剪拉強度 $V_{n,t} = V_c + V_s = 0.17\sqrt{f_c'(MPa)}b_w d + \frac{A_{vt} f_{yt} d}{s}$; $V_c \neq 0$
6. 計算DBE需求下之剪力強度 $V_{n@DBE} = \min(V_{n,c}, V_{n,t}) + (1.0 + 1.0) A_{vd} f_{yd} \sin\alpha$

註：混凝土少量開裂，撓曲主筋降伏

MCE剪力強度之計算流程

Strength (MCE):

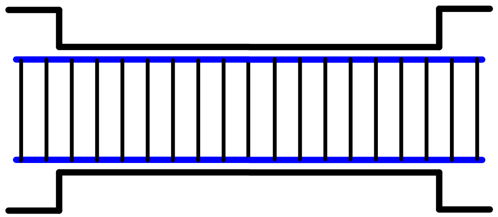


1. 壓力區深度 $a = \frac{1.25 f_y \times A_{st}}{0.85 f_c' \times b_w}$
2. 壓拉桿指標 $K = \tan^A \theta + \cot^A \theta - 1 + 0.14B \leq 1.64$
3. 開裂鋼筋混凝土軟化係數 $\zeta = \frac{3.35}{\sqrt{f_c'}} \leq 0.52$
4. 計算剪壓強度 $V_{n,c} = K\zeta f_c' b_w a \sin\theta$
5. 計算剪拉強度 $V_{n,t} = V_s = \frac{A_{vt} f_{yt} d}{s}$; $V_c = 0$
6. 計算MCE需求下之剪力強度 $V_{n@MCE} = \min(V_{n,c}, V_{n,t}) + (1.25 + 1.25) A_{vd} f_{yd} \sin\alpha$

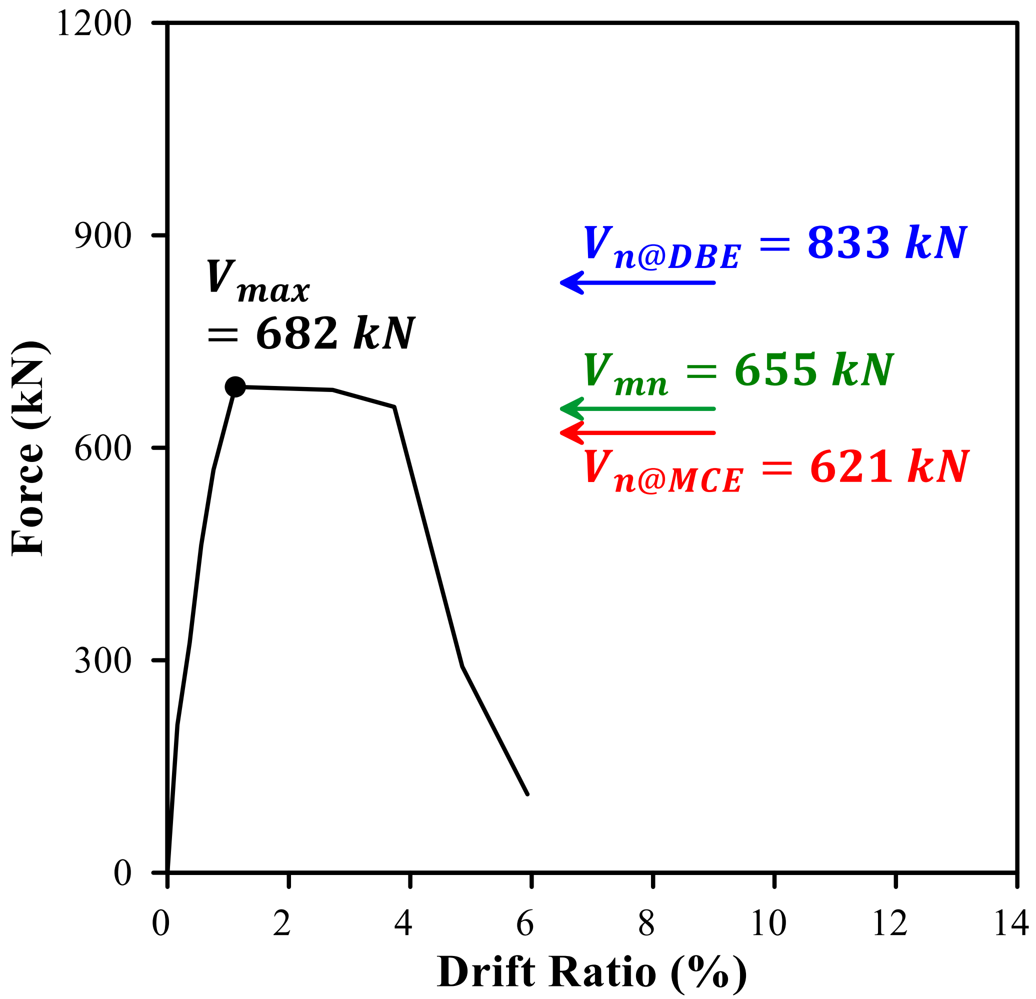
註：混凝土大量開裂，撓曲主筋應變硬化

$\ell_n / h = 3.0$

CB30-C



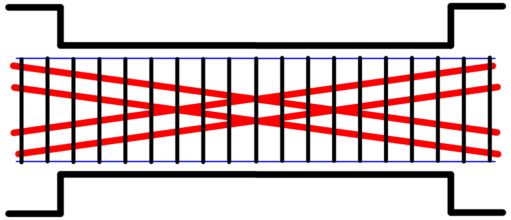
撓曲與剪力強度評估



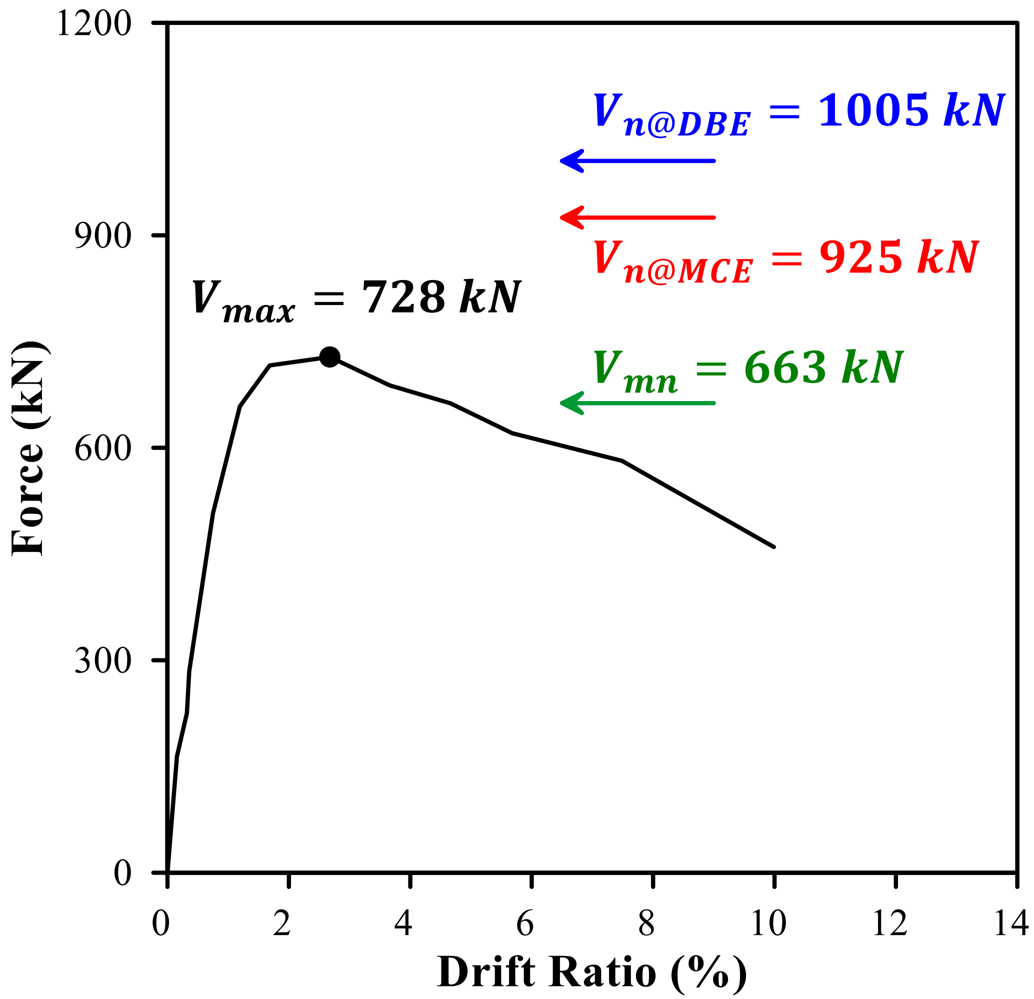
1. $V_{mn} < V_{n@DBE}$
2. 撓曲強度主控行為
3. $\frac{V_{max}}{V_{mn}} = 1.04$
4. $V_{n@MCE} < V_{max}$
5. $\frac{V_{n@MCE}}{V_{max}} = 0.91$
6. 撓曲塑鉸變形發展尚需強化

$\ell_n / h = 3.0$

CB30-DB



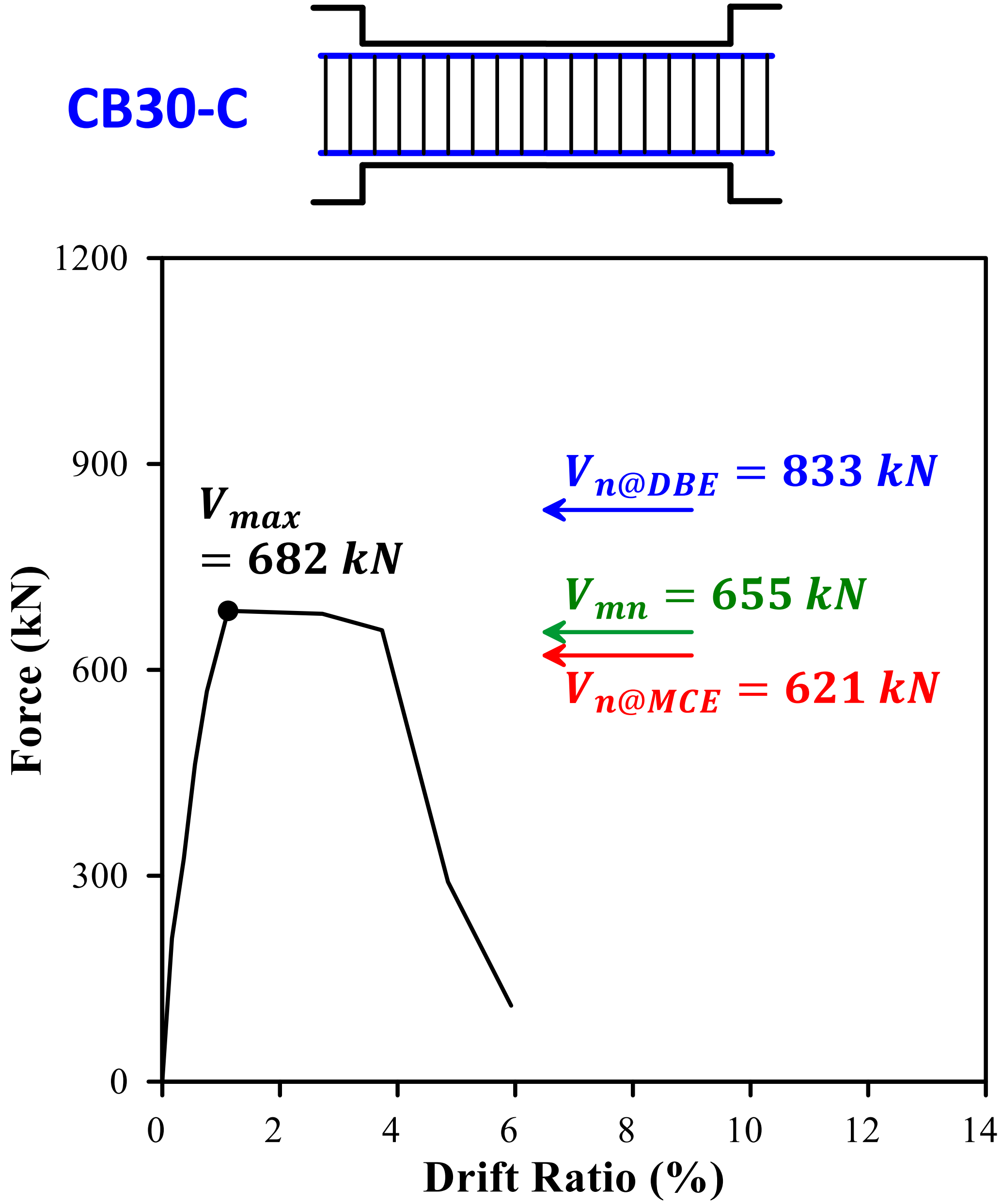
撓曲與剪力強度評估

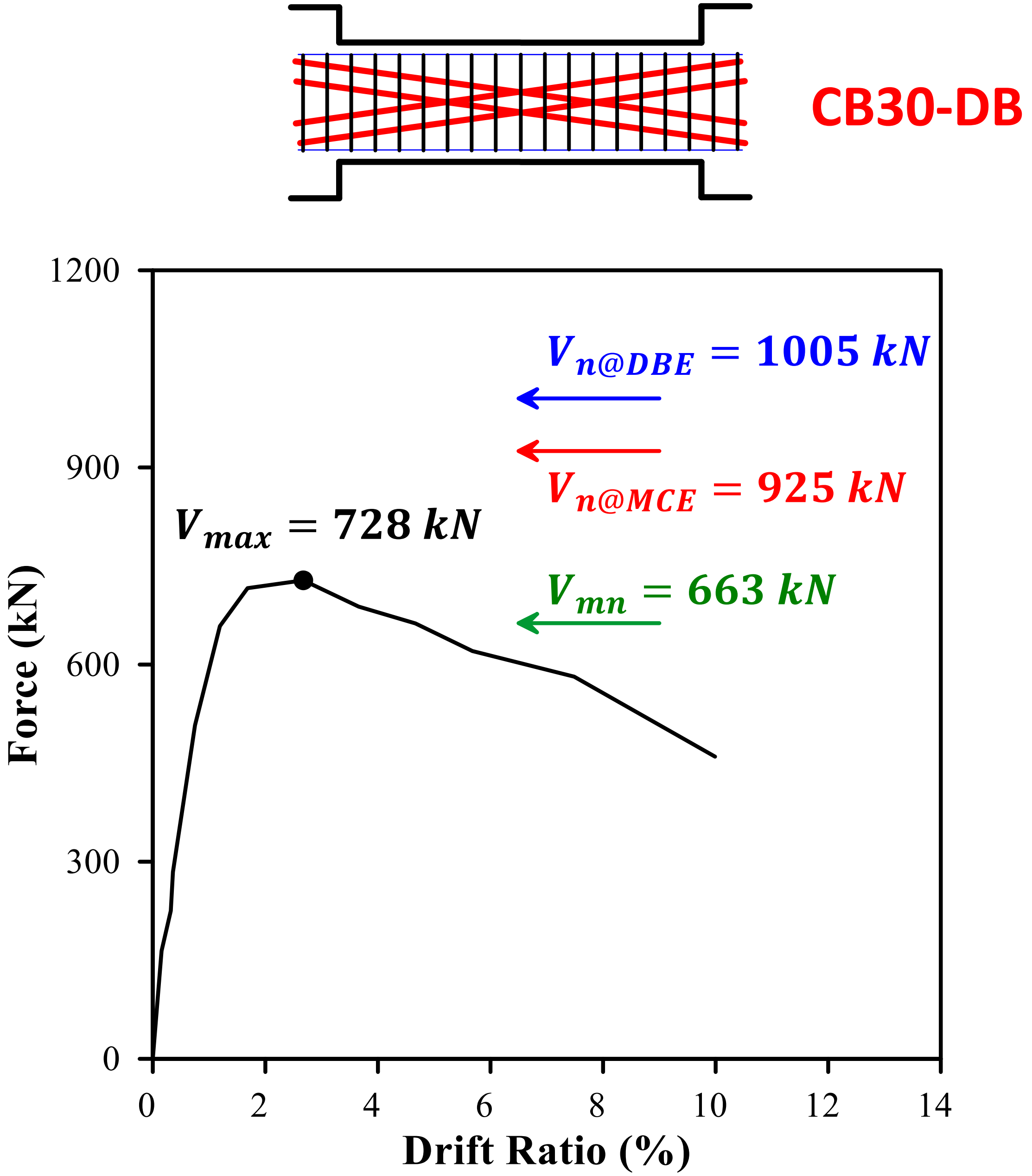


1. $V_{mn} < V_{n@DBE}$
2. 撓曲強度主控行為
3. $\frac{V_{max}}{V_{mn}} = 1.10$
4. $V_{n@MCE} > V_{max}$
5. $\frac{V_{n@MCE}}{V_{max}} = 1.27$
6. 撓曲塑鉸變形充分發展

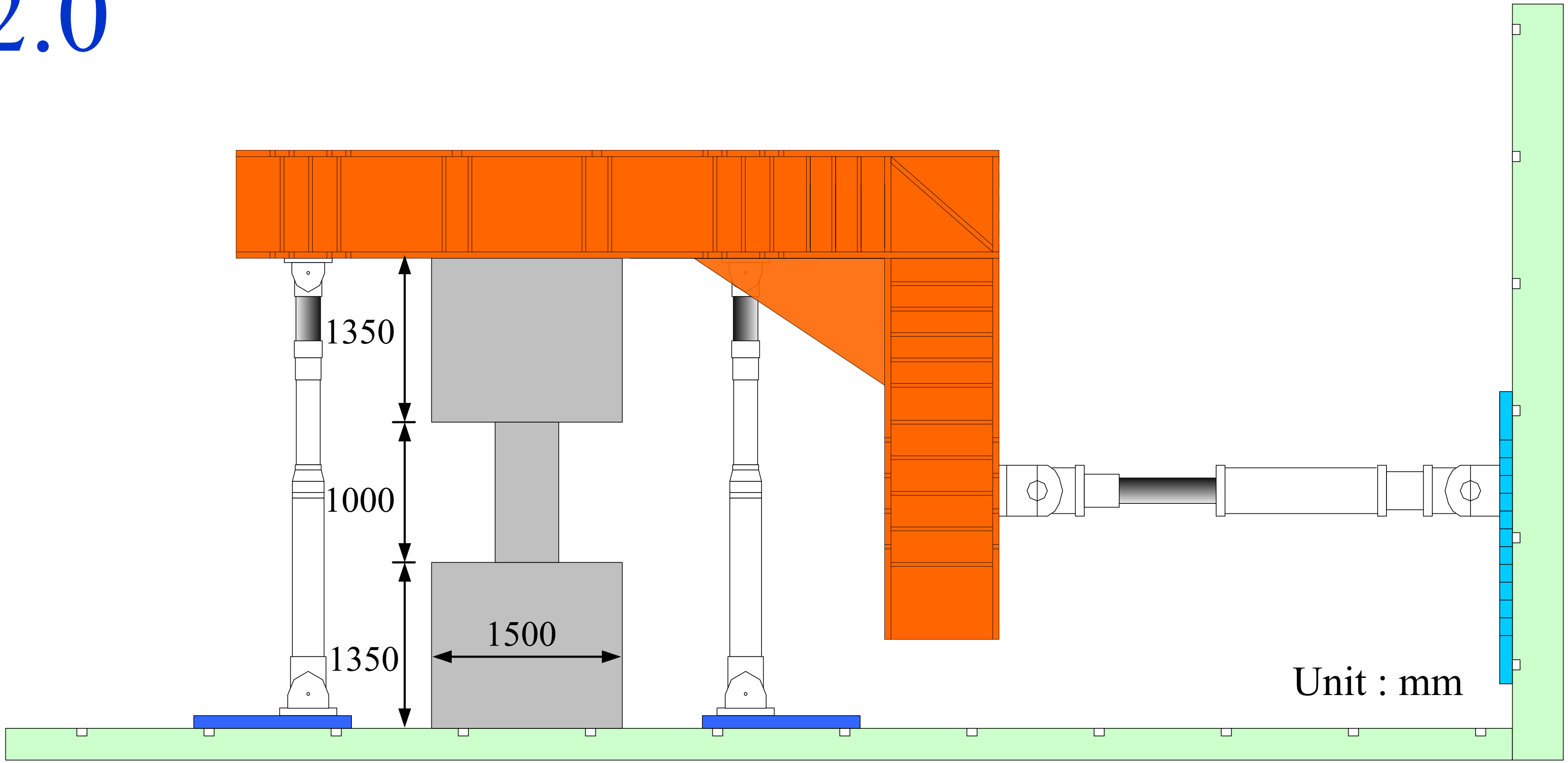
$\ell_n / h = 3.0$

測試行為評估





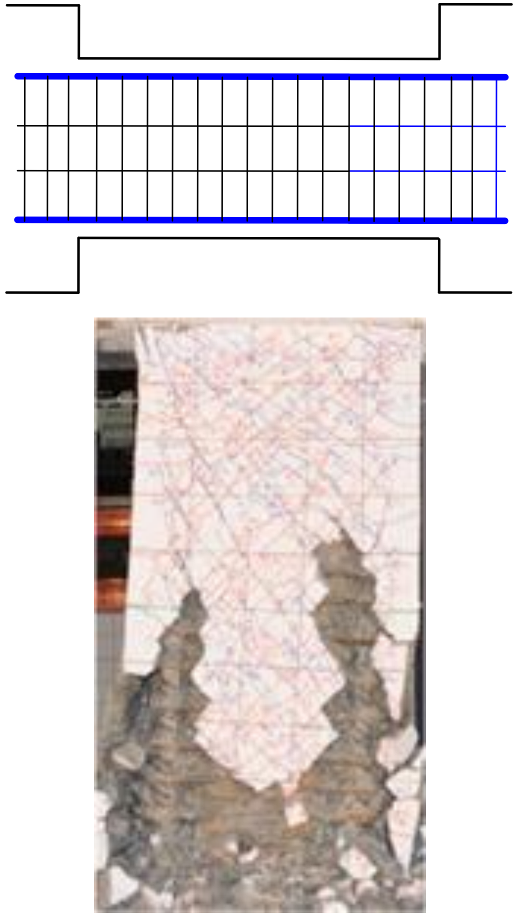
$$\frac{\ell_n}{h} = 2.0$$



Lim, E., Hwang, S. J., Wang, T. W., and Chang, Y. H. (2016). “An Investigation on the Seismic Behavior of Deep Reinforced Concrete Coupling Beams,” ACI Structural Journal, V. 113, No. 2, March-April, pp. 217-226.

試驗結果

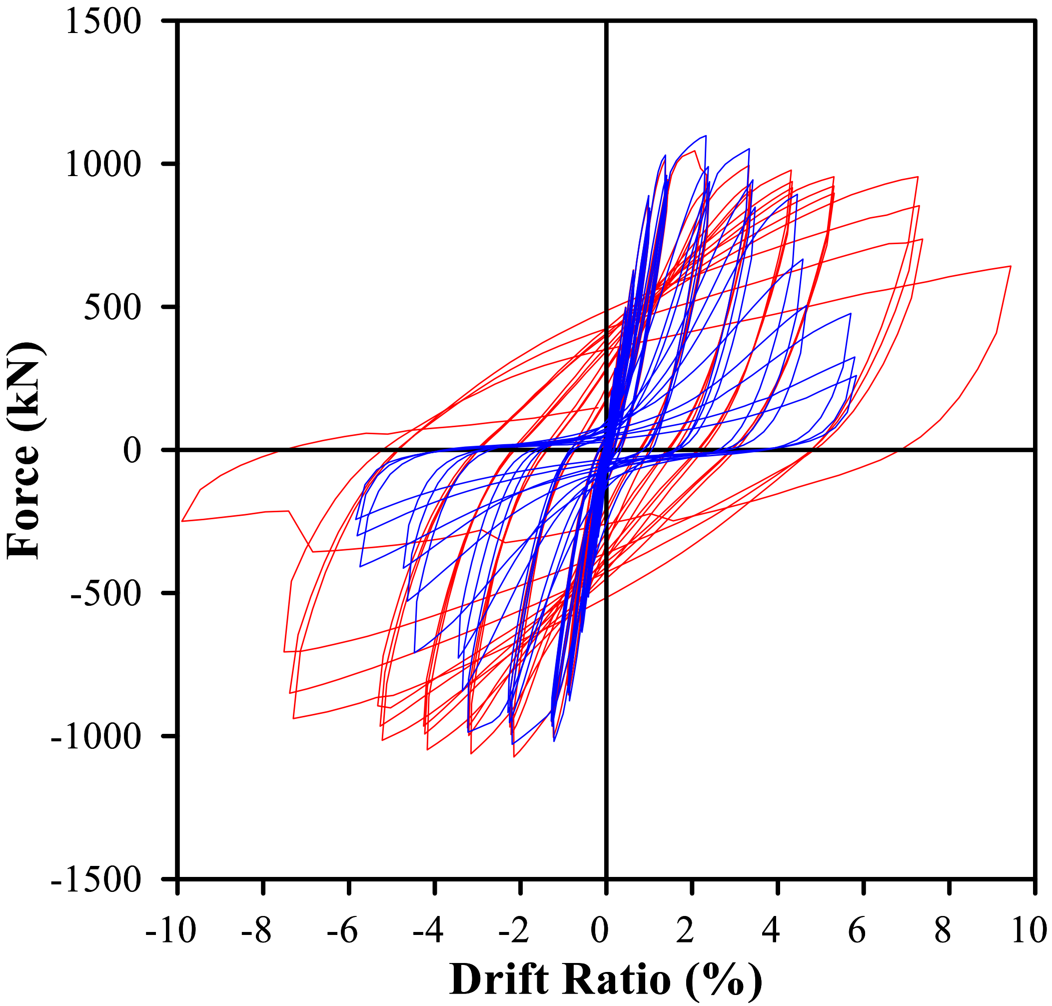
CB20-2



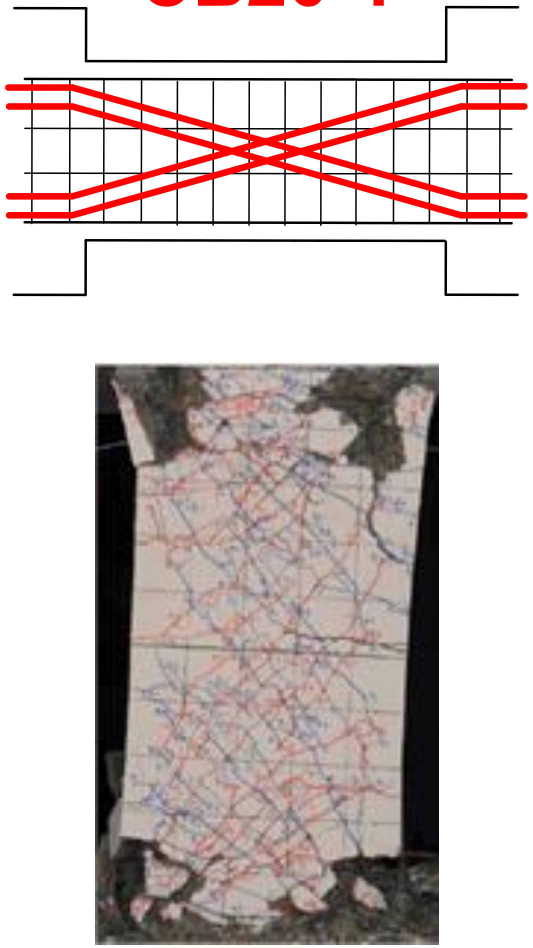
5.7%

$$\upsilon = 1.2\sqrt{f_c'}$$

UDR = 4.4%



CB20-1



7.3%

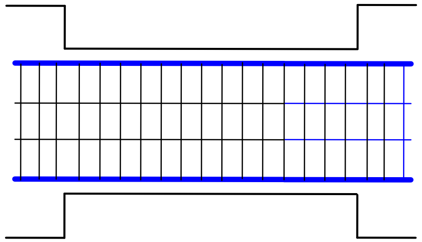
$$\upsilon = 1.3\sqrt{f_c'}$$

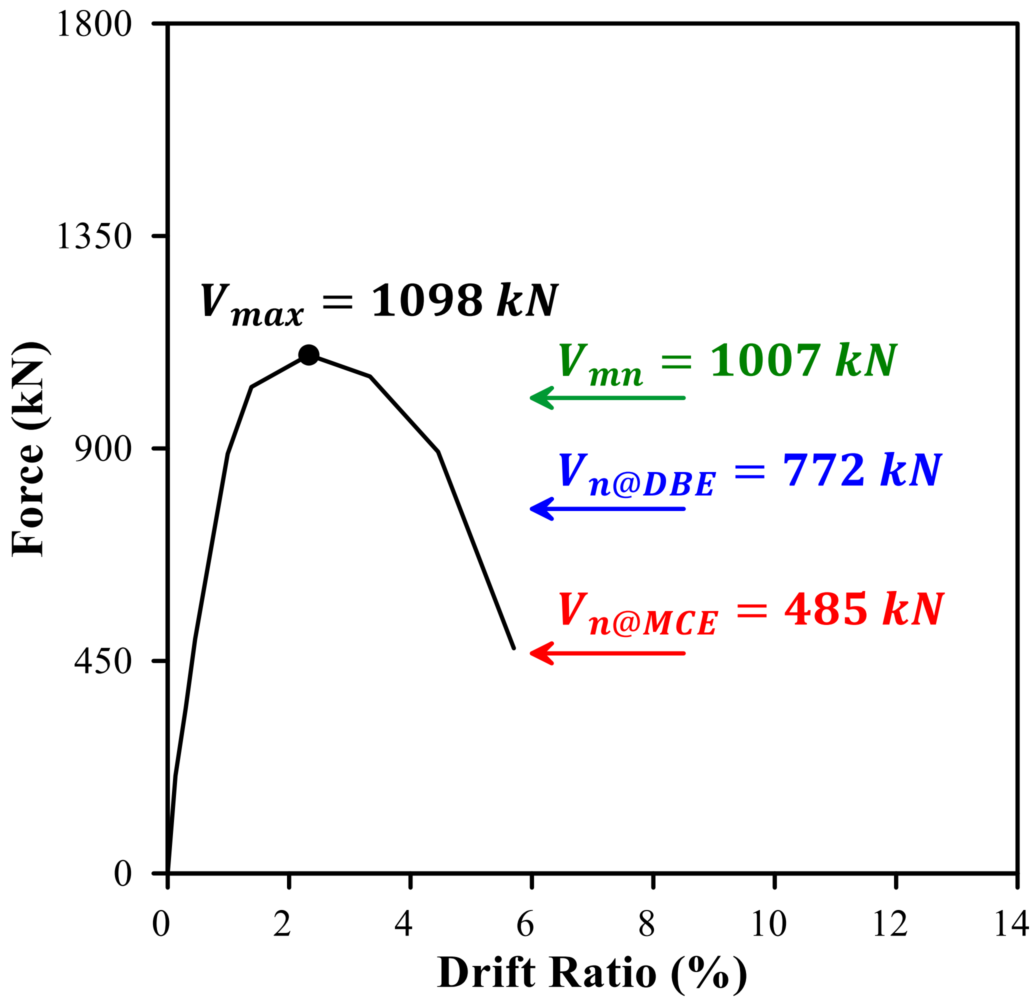
UDR = 7.2%

$\ell_n / h = 2.0$

以D-區域構件評估測試行為

CB20-2

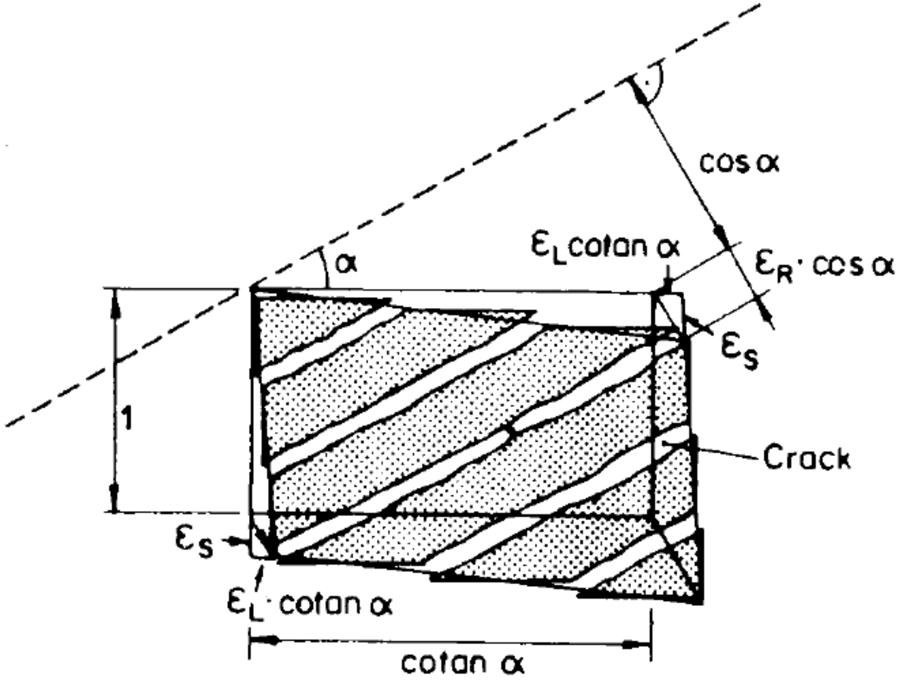


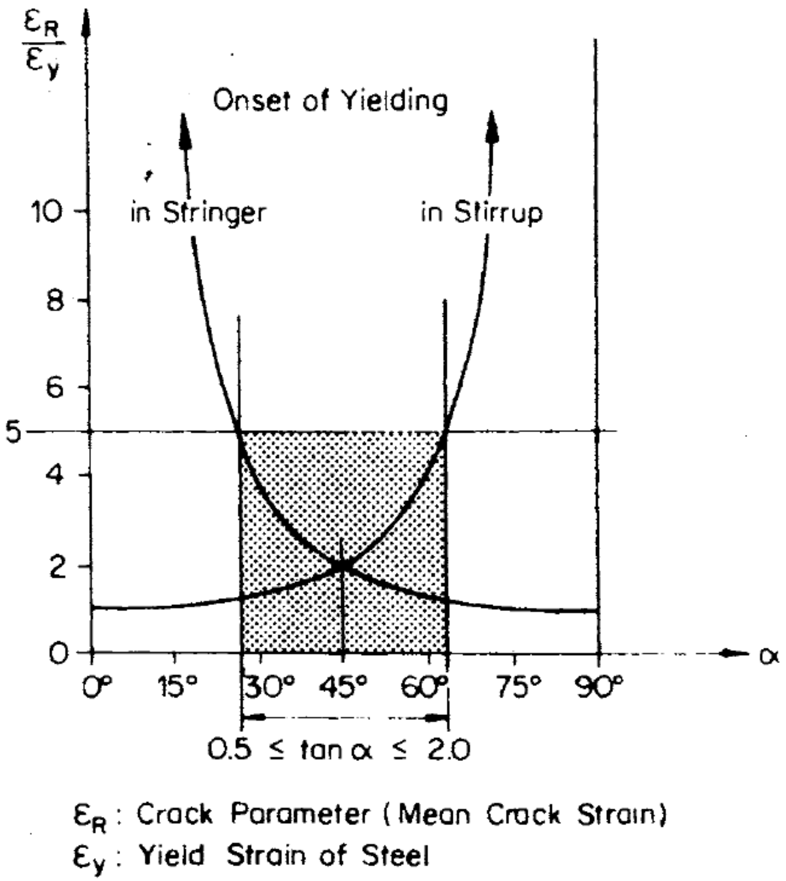


1. 試體撓曲強度已發展
2. $V_{n@DBE}$ 以D-區域評估，得剪力強度主控之結果，似有不當
3. 剪力傳遞模型為D-區域或是DBD-區域，宜再檢討

混凝土開裂角度研究

Plastic strain



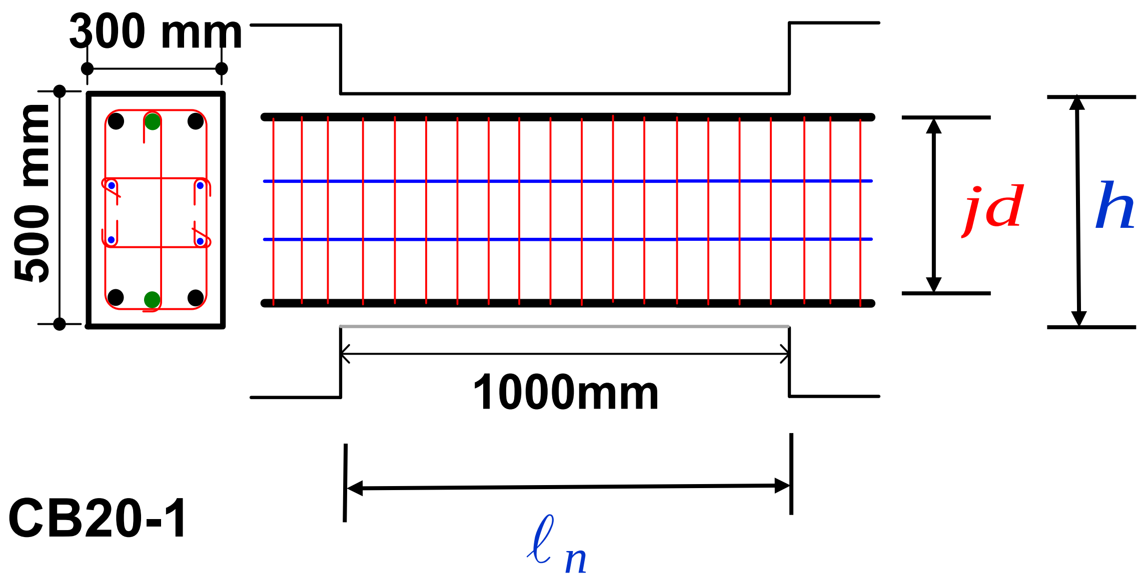


- 根據Thurlimann's kinematic study (1979), 混凝土開裂角θ應在範圍($\tan^{-1}(1/2) = 26.5°$, $\tan^{-1}(2) = 63.4°$)之間。

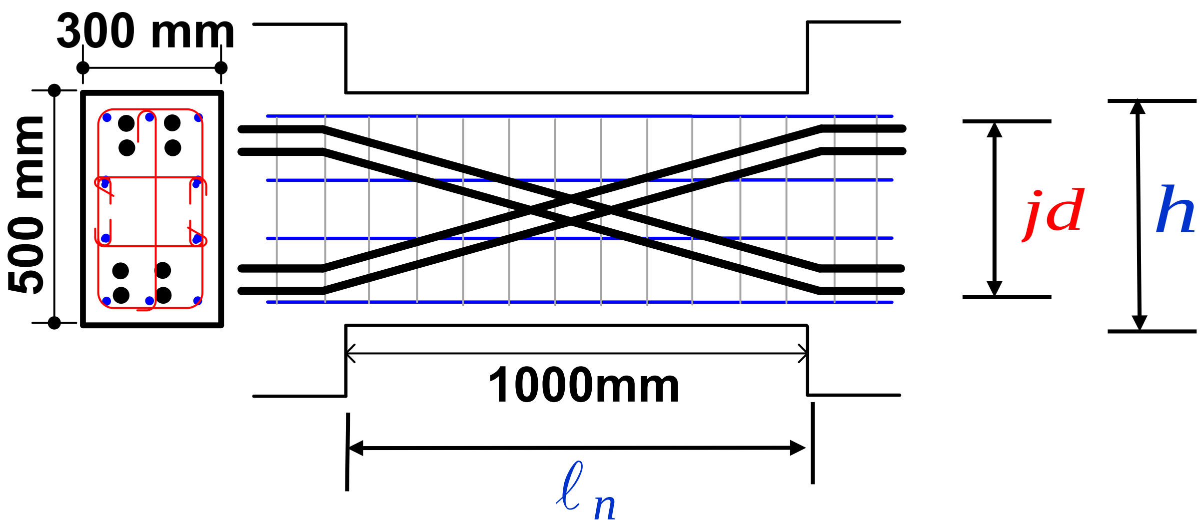
Thürlimann, B., "Shear Strength of Reinforced and Prestressed Concrete-CEB Approach," *ACI Special Publication*, SP 59, 1979, pp. 93-116.

D-區域構件或DBD-區域構件之判定

CB20-2



CB20-1



$$\frac{\ell_n}{h} = 2\ ;\ \ \tan^{-1}\left(\frac{h}{\ell_n}\right) = 26.5°$$

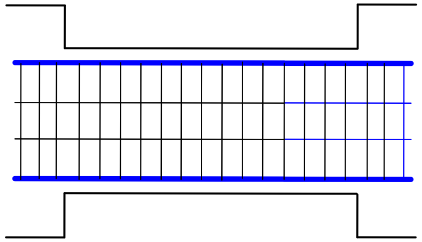
$$\theta = \tan^{-1}\left(\frac{jd}{\ell_n}\right) < \tan^{-1}\left(\frac{1}{2}\right) = 26.5°$$

$$\theta < 26.5°$$

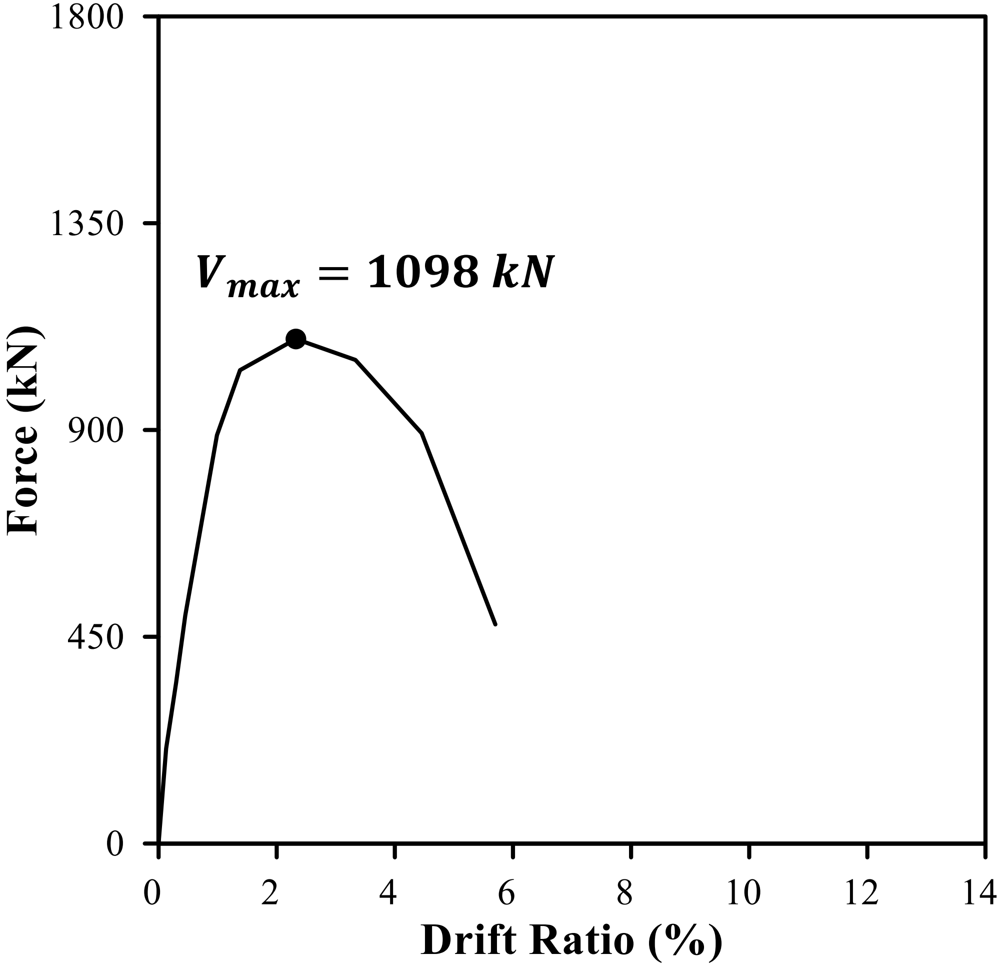
其為DBD傳力模式，亦即應以中短連接梁模擬較佳

$\ell_n / h = 2.0$

CB20-2



撓曲與剪力強度評估



D-區域構件

$V_{mn} = 1007\ kN$

$V_{n@DBE} = 772\ kN$

$V_{n@MCE} = 485\ kN$

DBD-區域構件

$V_{n@DBE} = 1066\ kN$

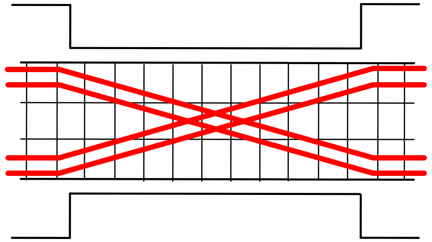
$V_{mn} = 1007\ kN$

$V_{n@MCE} = 657\ kN$

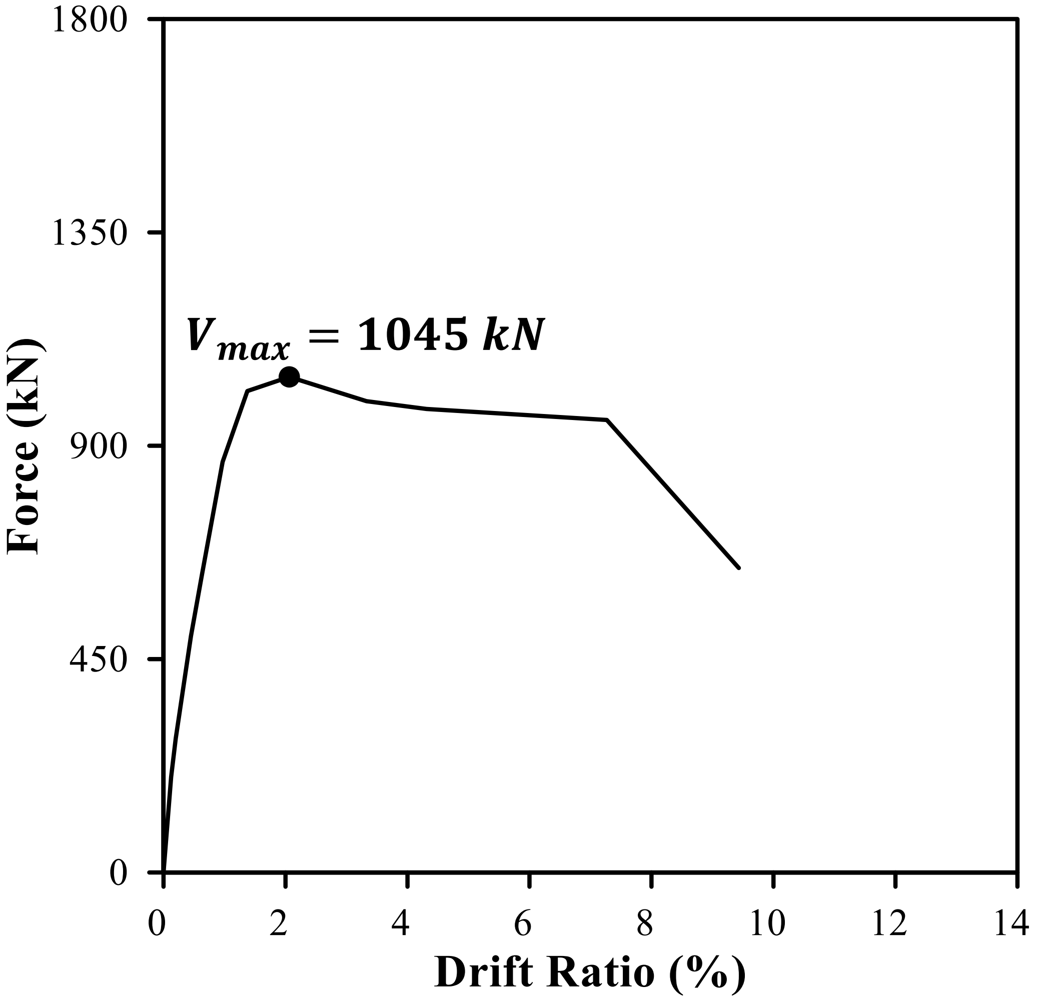
- 因取θ=45°，DBD-區域評估之剪力強度較高
- DBD-區域評估之$V_{n@DBE}$較為合理
- $V_{n@DBE}$應大於V_{max}才為合理

$\ell_n / h = 2.0$

CB20-1



撓曲與剪力強度評估

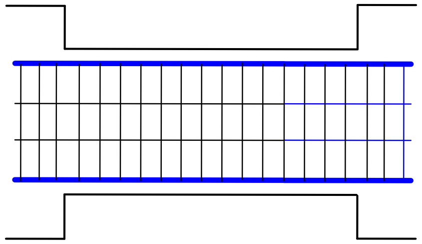


D-區域構件	DBD-區域構件
$V_{n@DBE} = 1418\ kN$	$V_{n@DBE} = 1577\ kN$
$V_{n@MCE} = 1160\ kN$	$V_{n@MCE} = 1293\ kN$
$V_{mn} = 978\ kN$	$V_{mn} = 978\ kN$

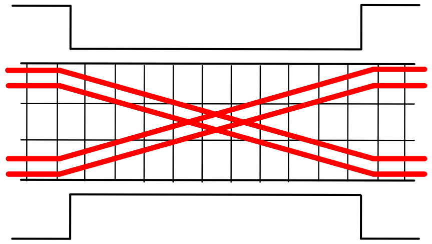
- 因取θ=45°，DBD-區域評估之剪力強度稍高
- 兩者相差不多

以DBD-區域構件評估測試行為

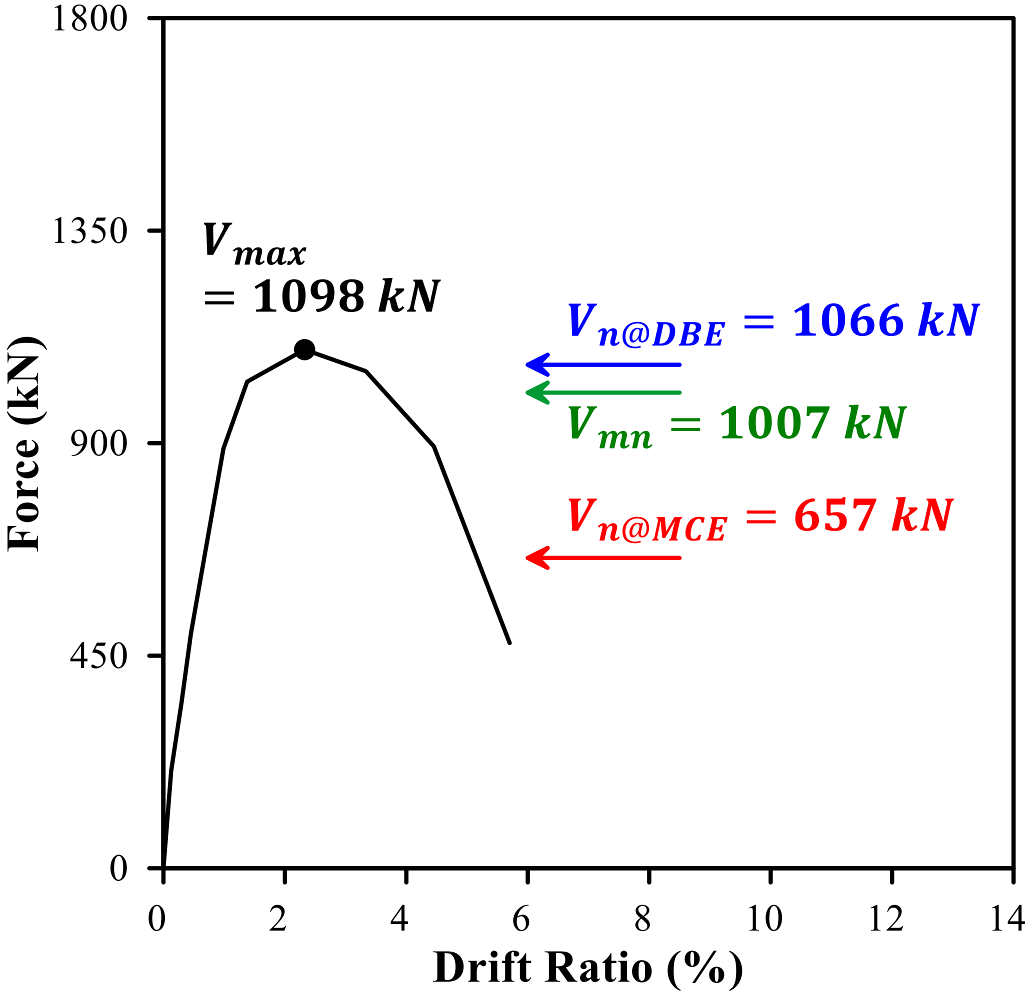
CB20-2

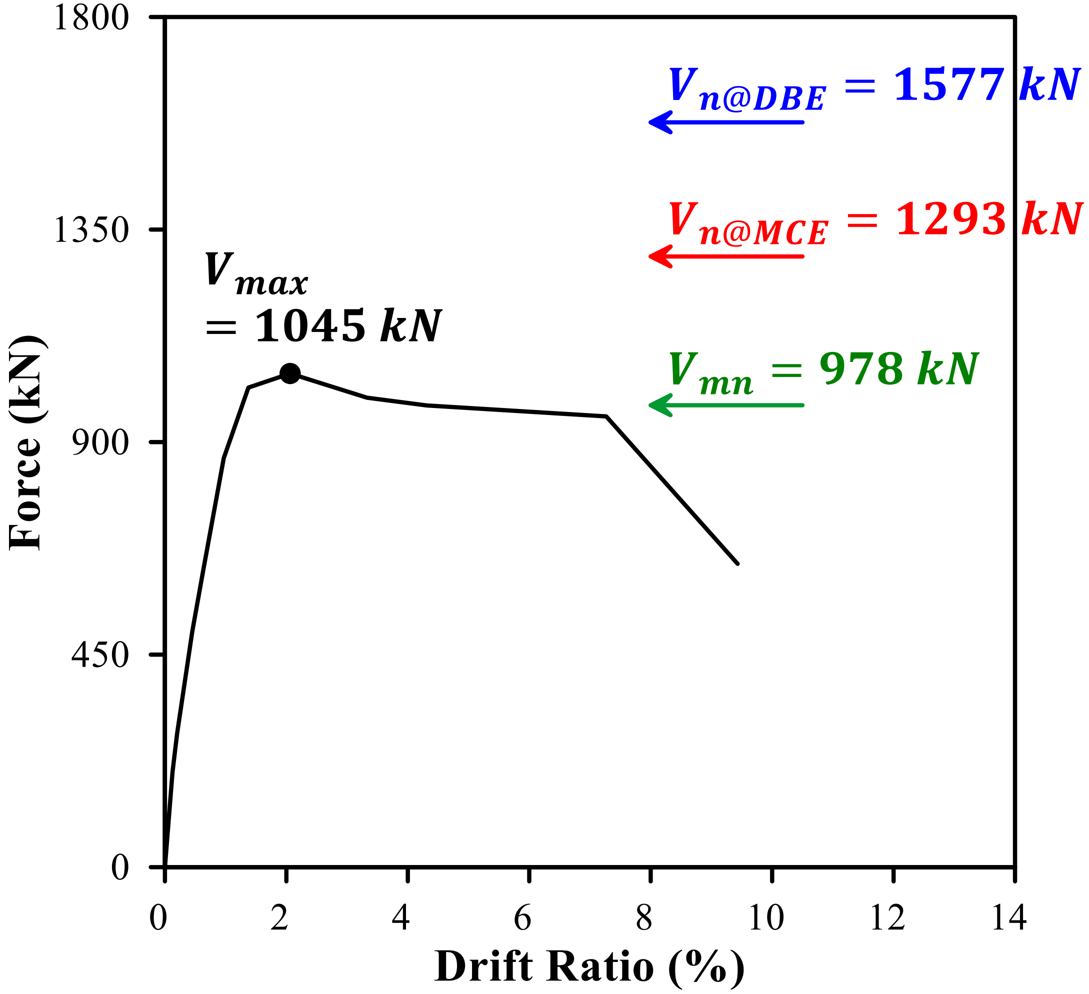


$\ell_n / h = 2.0$

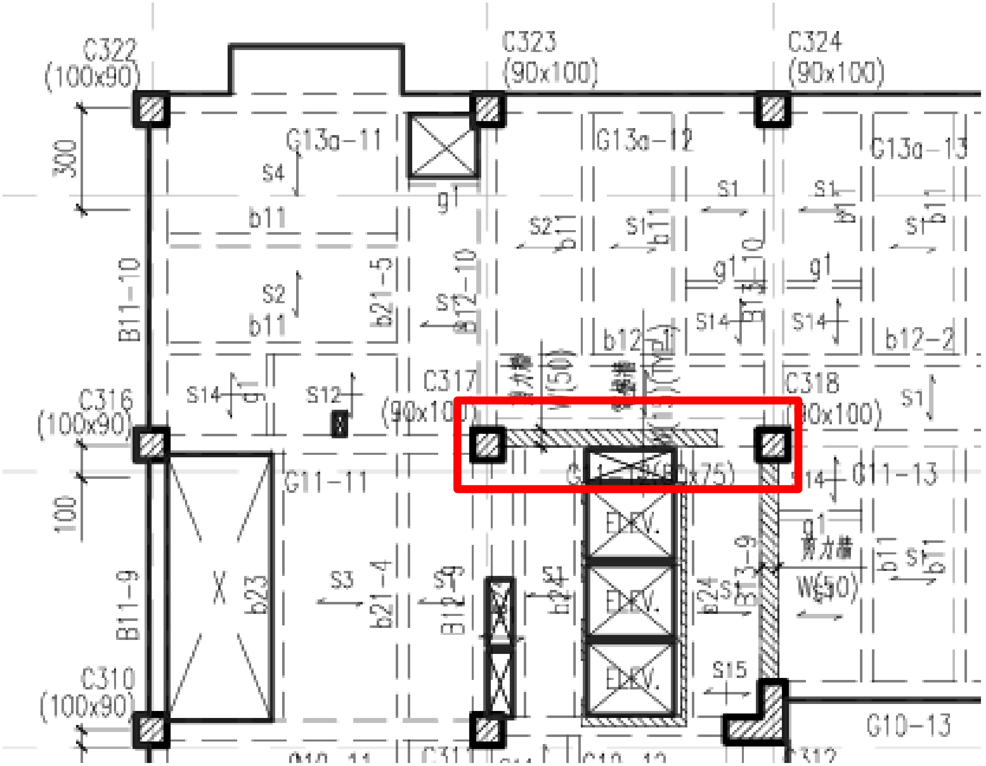


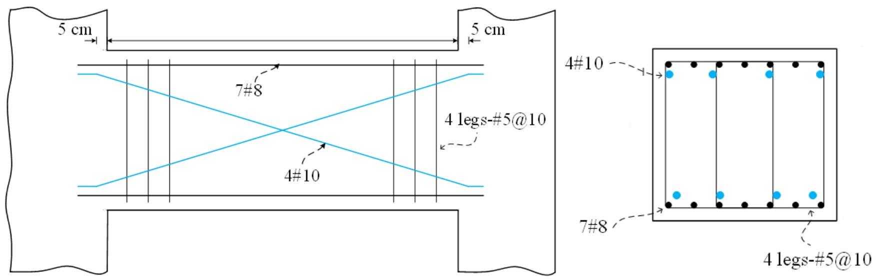
CB20-1





非結構RC牆所造成中短梁之剪力強度設計





- 中小度地震剪力強度設計

彈性壓力區深度 $kd = \left(\sqrt{(\rho_f n)^2 + 2\rho_f n} - \rho_f n\right) d$

$$V_n = \min\left(V_{n,c}, V_{n,t}\right) + (1.0 + 1.0) A_{vd} f_{yd} \sin\alpha$$

$$\phi V_n \geq V_u$$

- 增加梁之剪力強度與轉動能力

壓力區深度 $c = \dfrac{f_{yd} \times A_{vd} \cos\alpha}{0.85 f_c' \times b_w}$　　塑鉸少量轉動

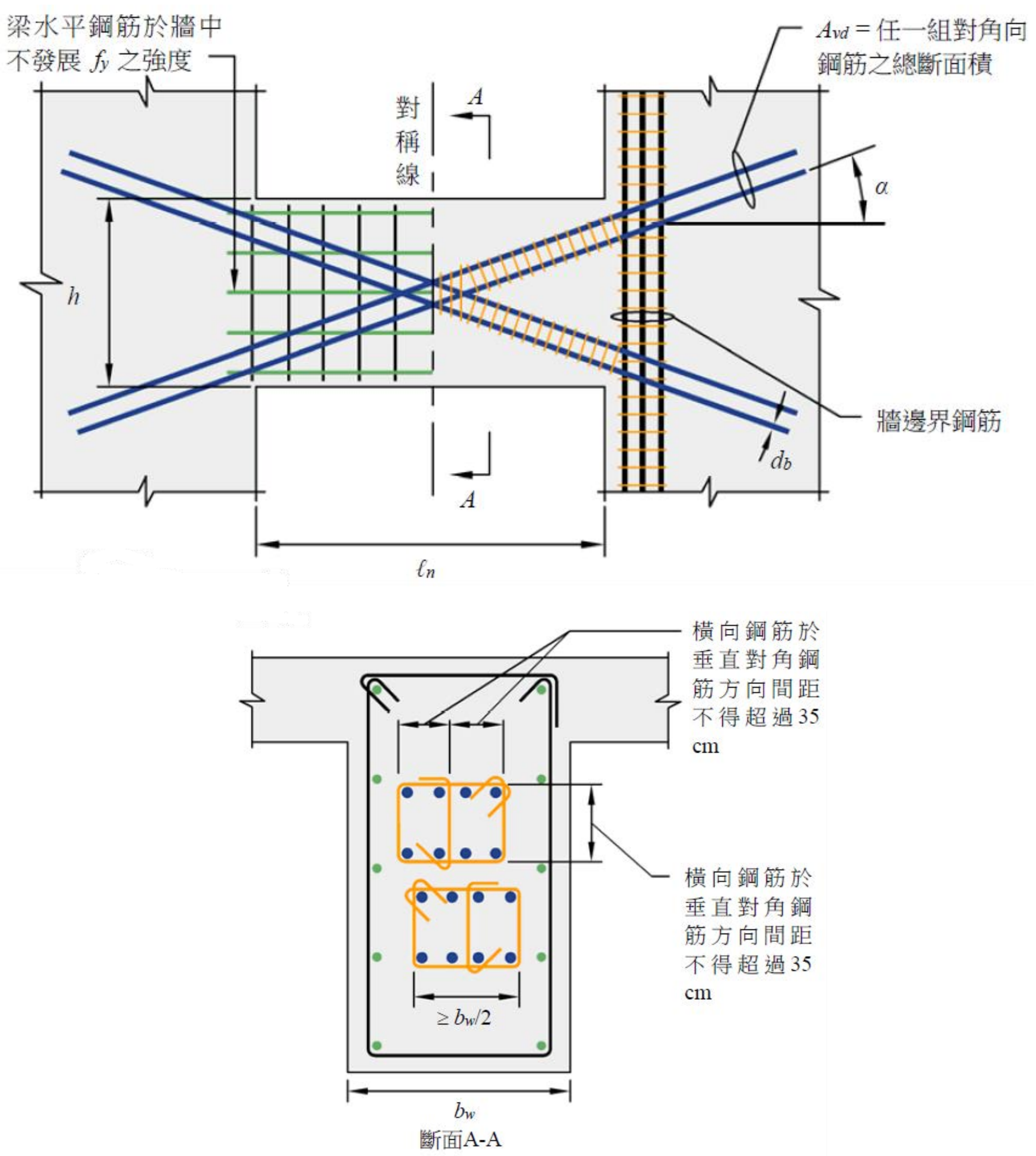
$$V_n = \min\left(V_{n,c}, V_{n,t}\right) + (1.0 + 1.0) A_{vd} f_{yd} \sin\alpha$$

$$\phi V_n \geq \frac{2 \times M_n}{\ell_n}$$

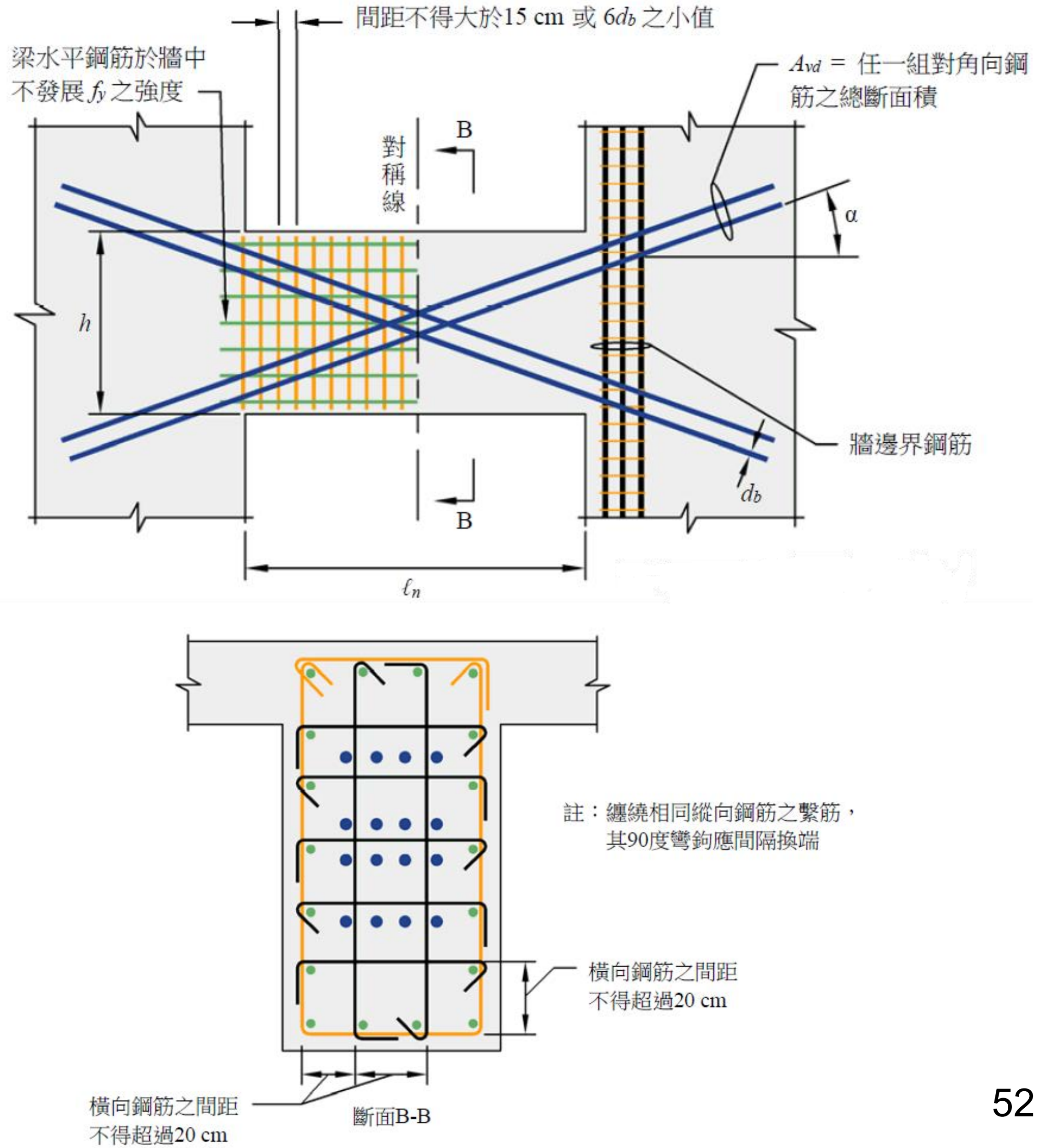
ACI 318-25規範討論

橫向鋼筋圍束效應之探討

圍束對角鋼筋



連接梁斷面全圍束



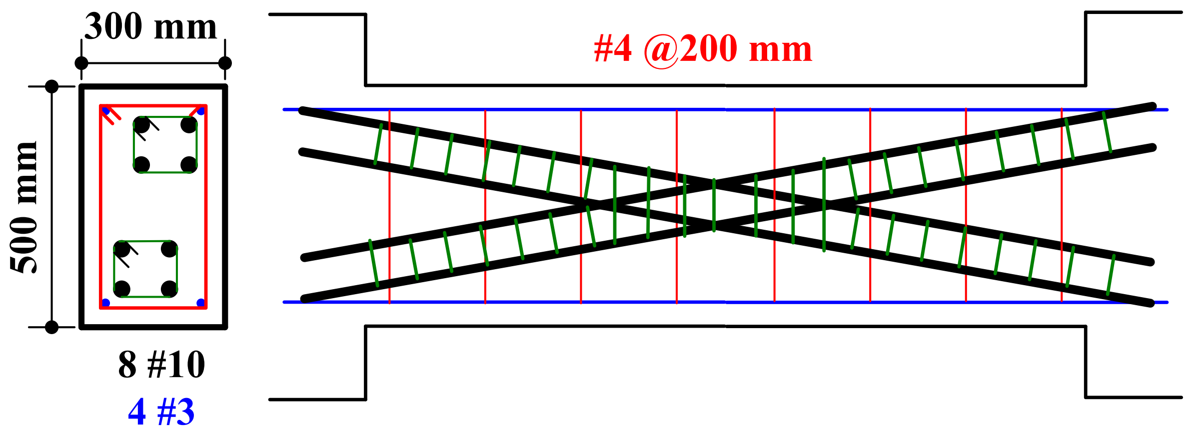
$\ell_n / h = 3.0$

實驗試體

CB30-DA

$f'_c = 39.7\ MPa$

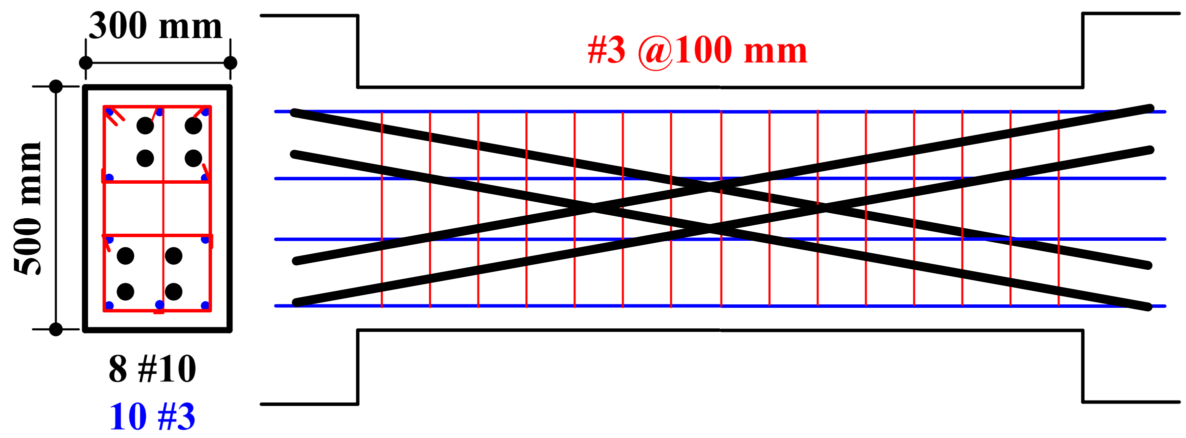
$\rho_f = 2.89\%$



CB30-DB

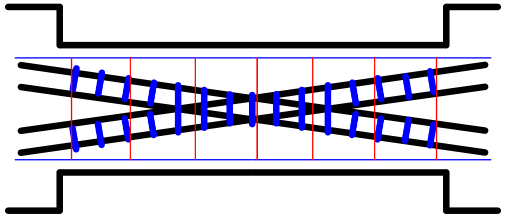
$f'_c = 38.4\ MPa$

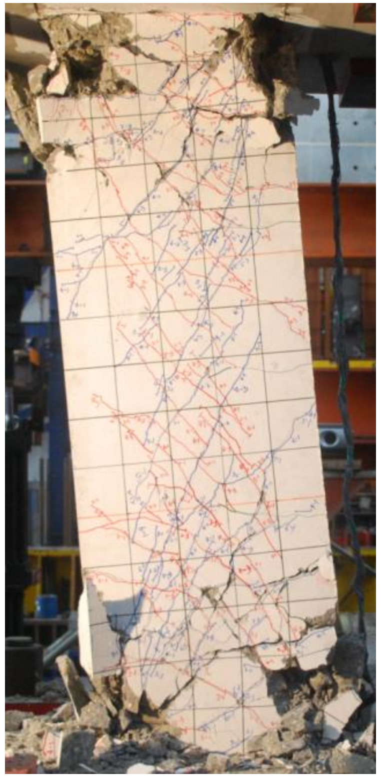
$\rho_f = 2.86\%$



試驗結果

CB30-DA

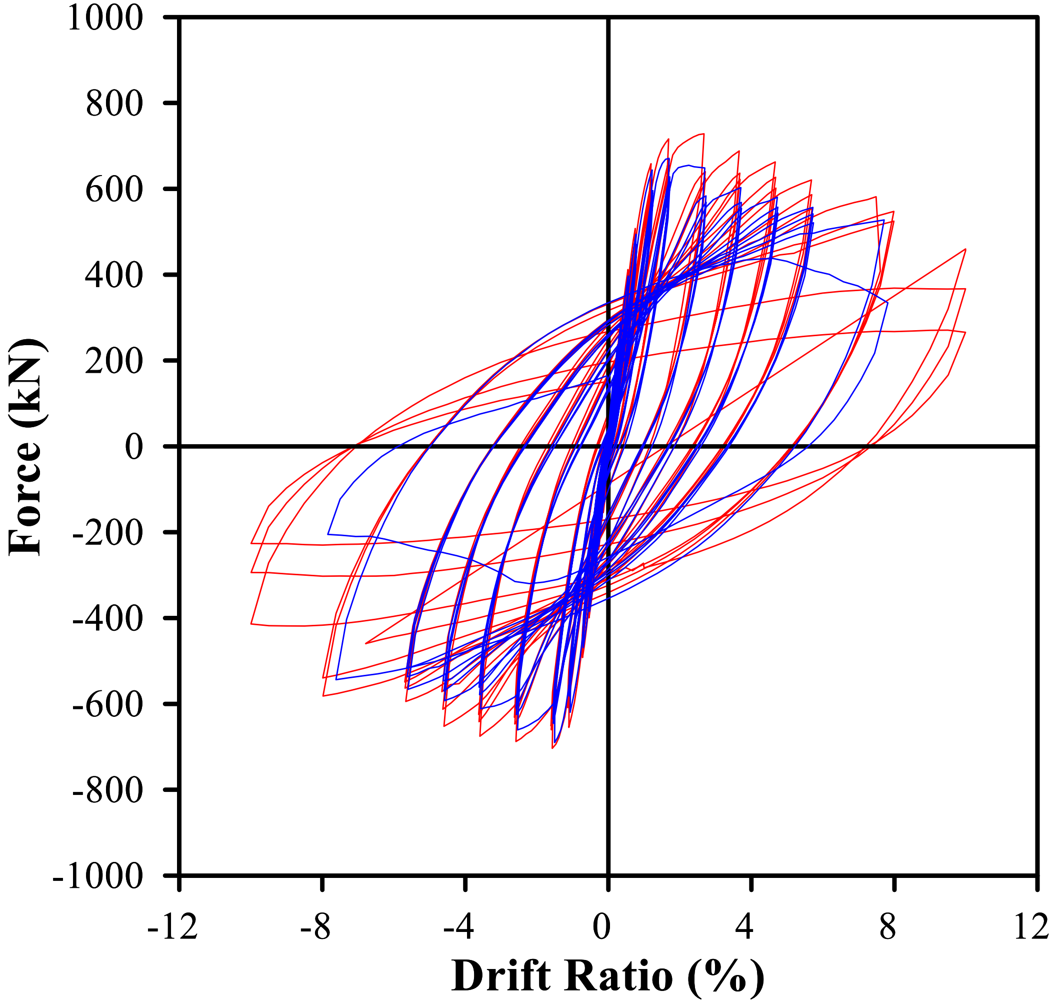




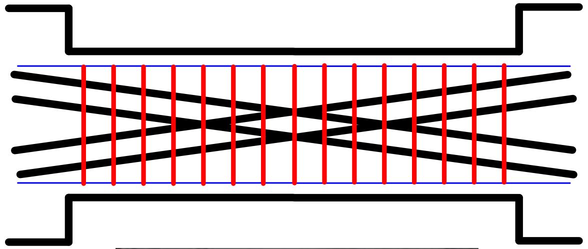
7.7%

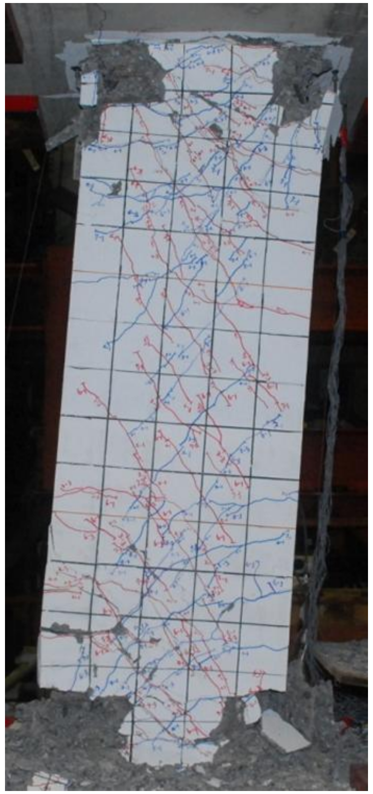
$$\upsilon = 0.98\sqrt{f_c'}$$

UDR=7.0%



CB30-DB



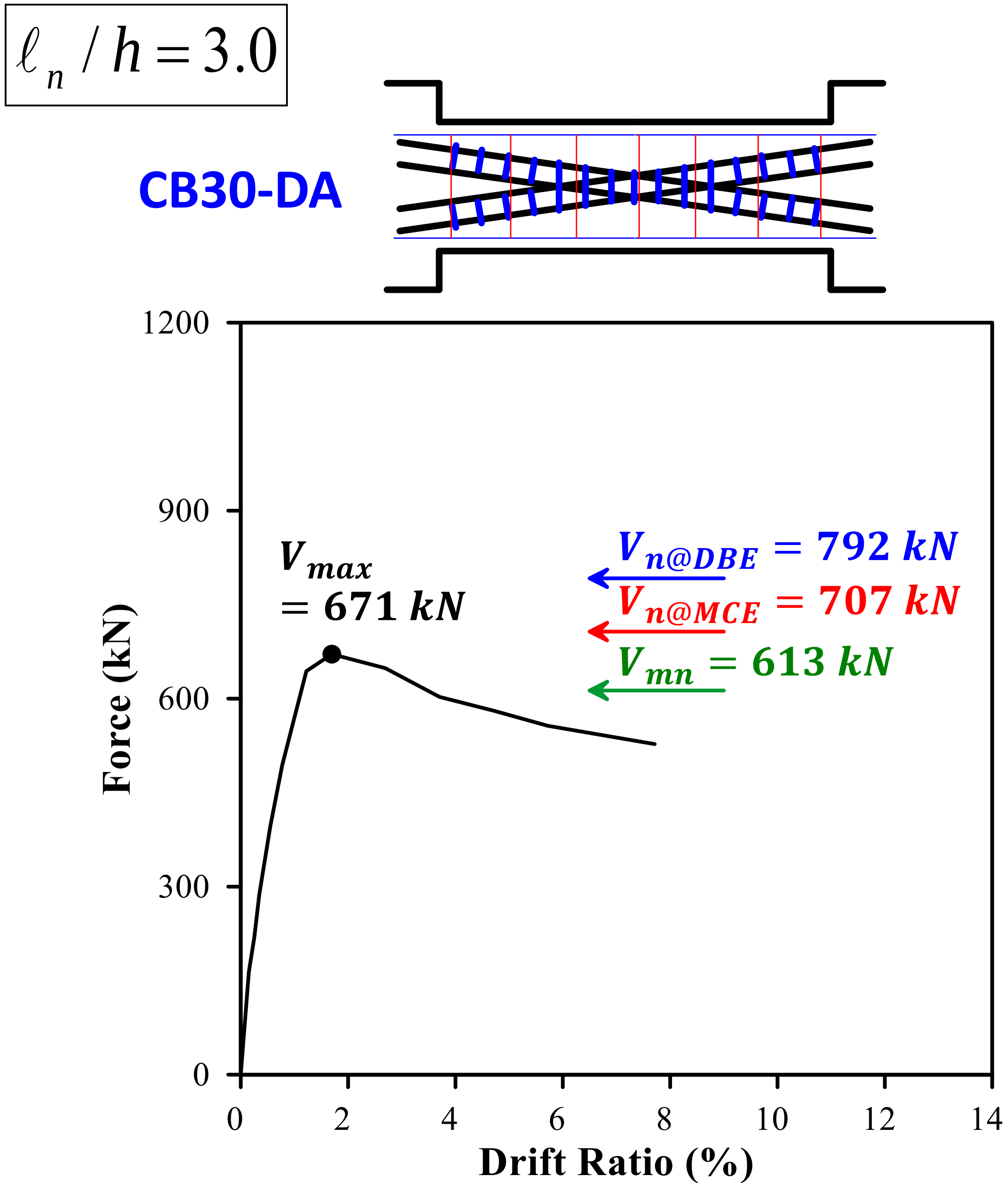


8.0%

$$\upsilon = 1.04\sqrt{f_c'}$$

UDR=7.4%

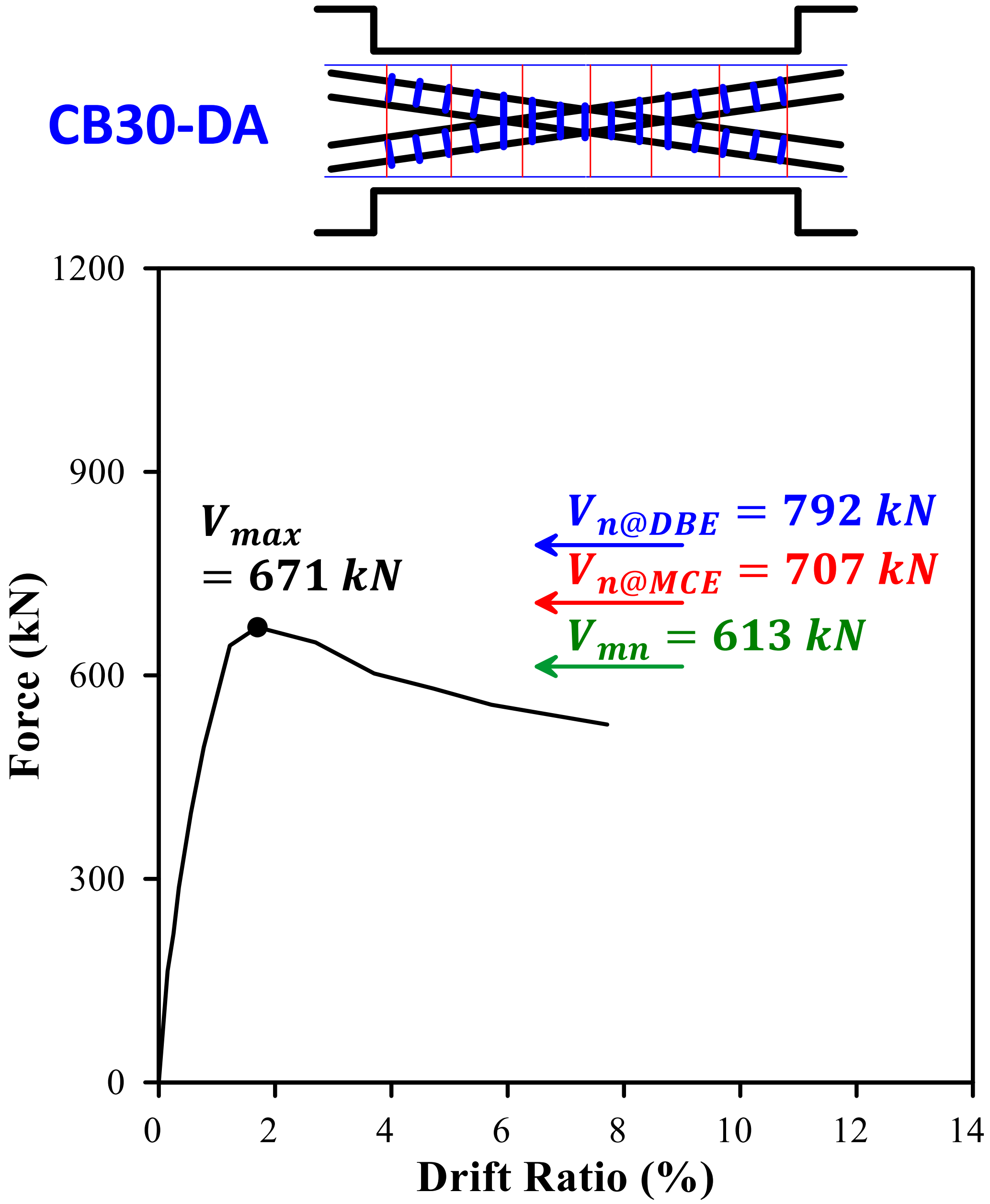
撓曲與剪力強度評估

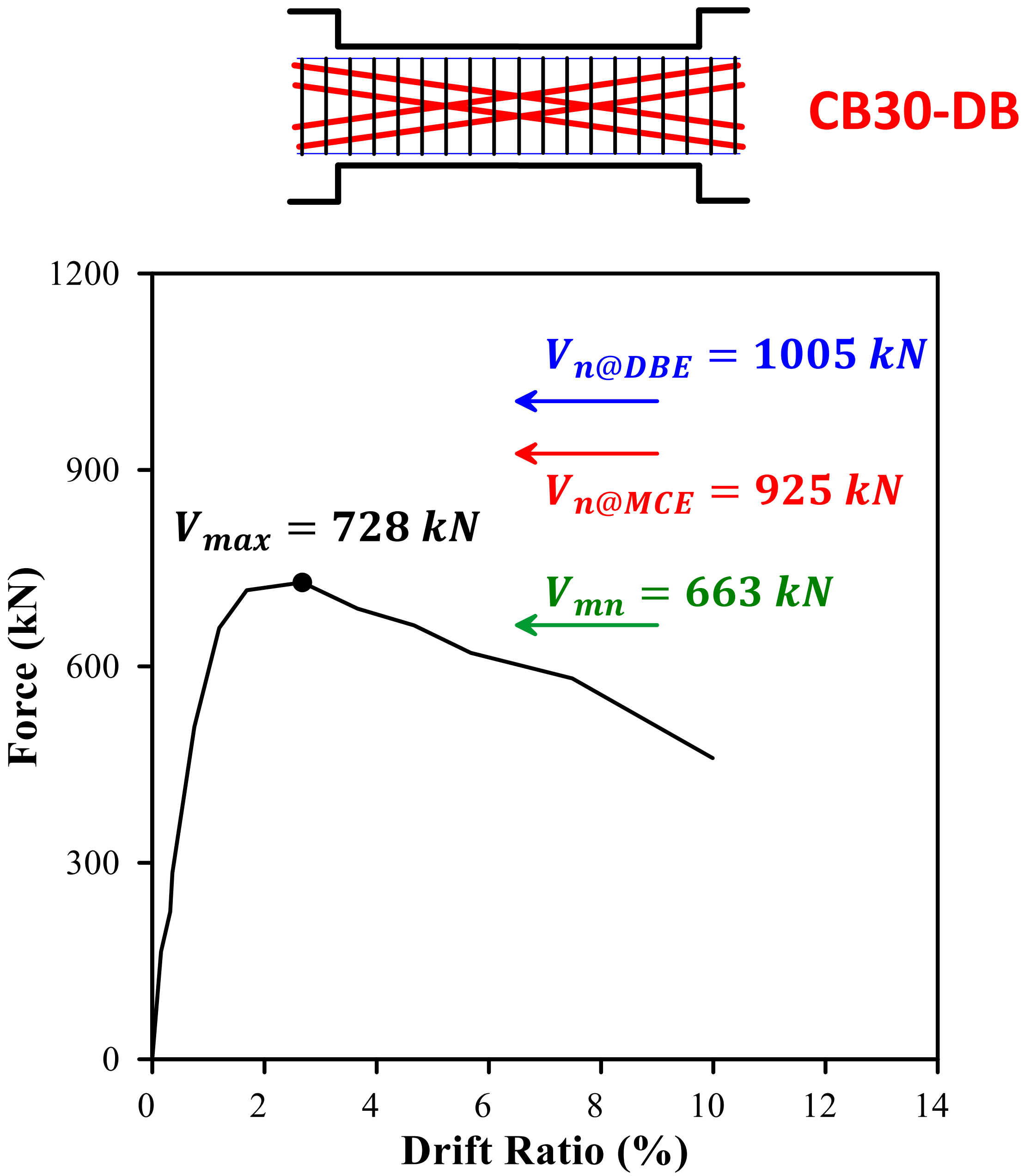


1. $V_{mn} < V_{n@DBE}$
2. 撓曲強度主控行為
3. $\frac{V_{max}}{V_{mn}} = 1.09$
4. $V_{n@MCE} > V_{max}$
5. $\frac{V_{n@MCE}}{V_{max}} = 1.05$
6. 撓曲塑鉸變形充分發展

$\ell_n / h = 3.0$

測試行為評估





強度預測之缺陷

ACI 318-25 強度設計之缺陷

1. 忽略混凝土強度貢獻，提供超量之對角鋼筋量

   → 引致較大之作用力，可能產生非預期之破壞

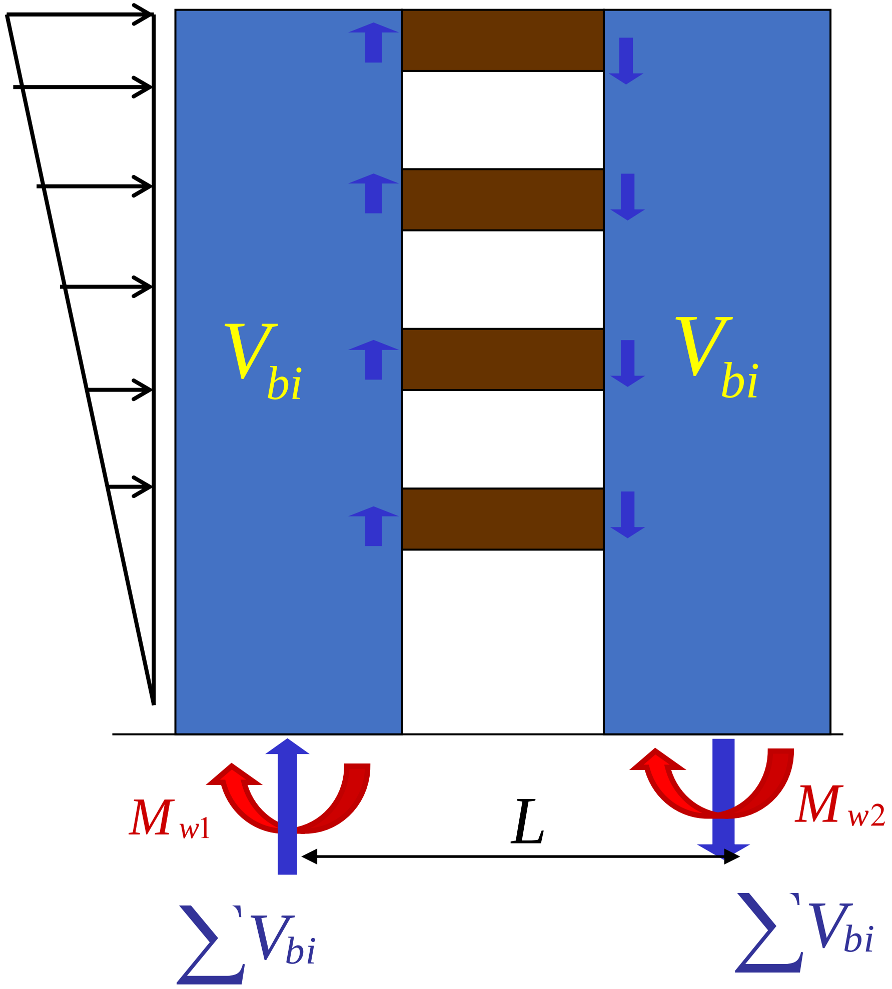
   → 造成鋼筋壅塞，施工困難

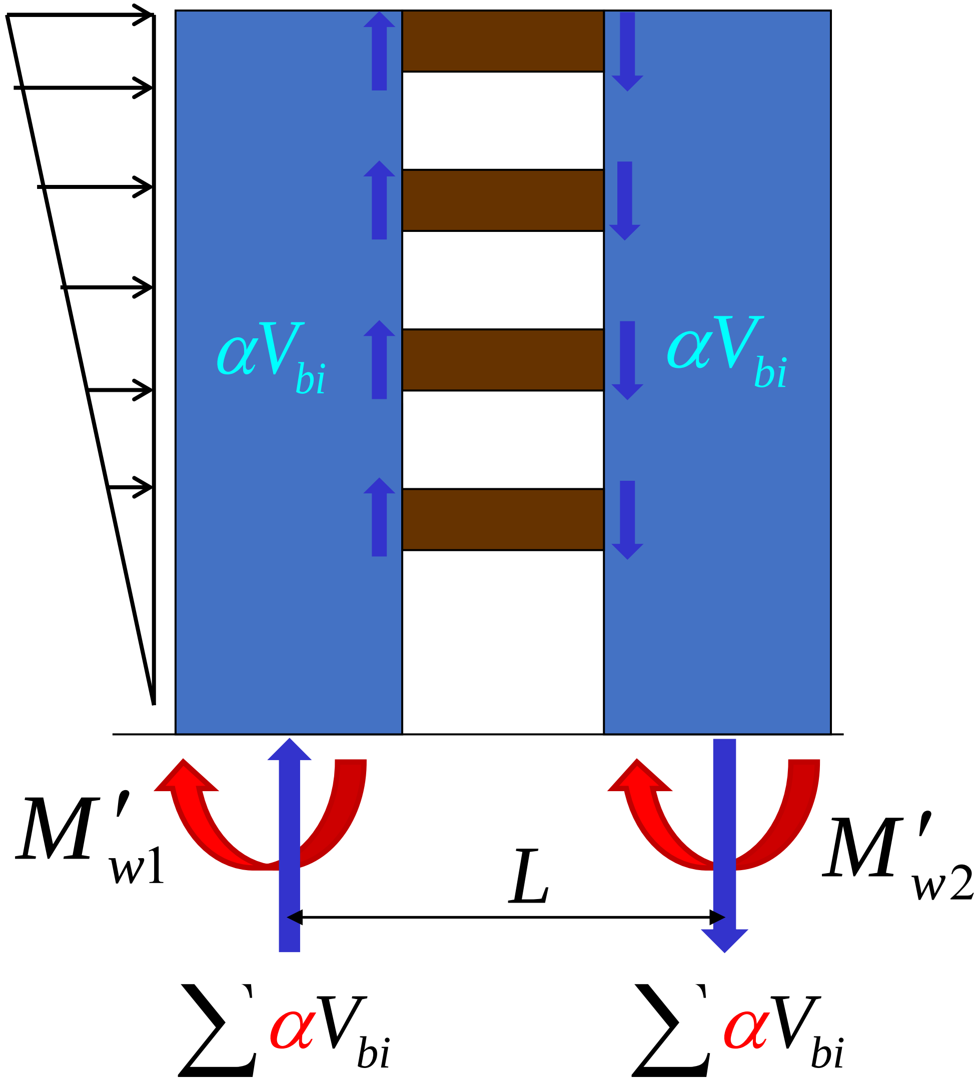
2. 足夠之對角鋼筋量，混凝土已無擠碎破壞之疑慮，

   剪力強度上限應無必要， ~~$V_n \le 0.83\sqrt{f_c'}bd$~~

國立臺灣大學 National Taiwan University

超出預期之作用力易引致非預期之破壞





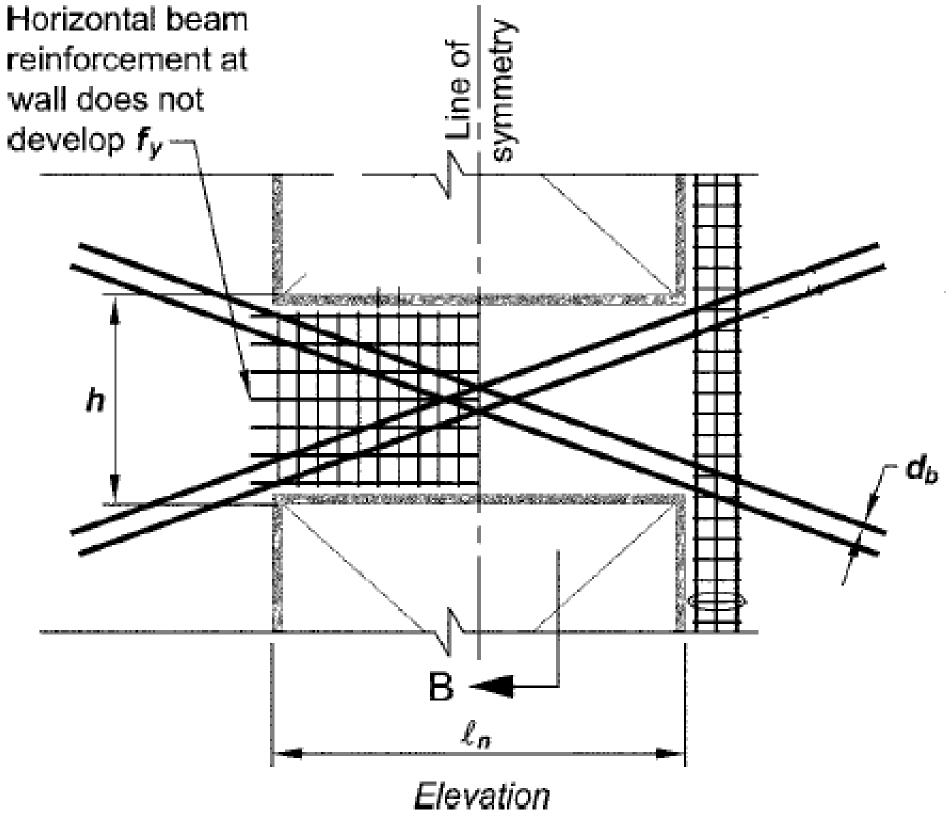
超量之對角鋼筋配置

Figure 1 Showing the intricate, labor-intensive steel reinforcing bar congestion in a typical high-rise coupling beam. Photo credit: Remy Lequesne, The University of Michigan

過多之對角鋼筋量 → 鋼筋壅塞 → 施工困難

鋼筋配置之改善

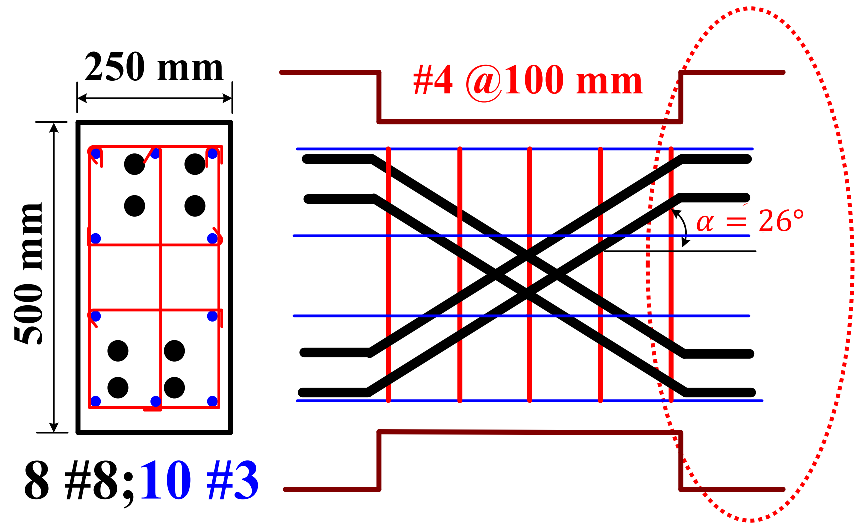
ACI 對角鋼筋之配置



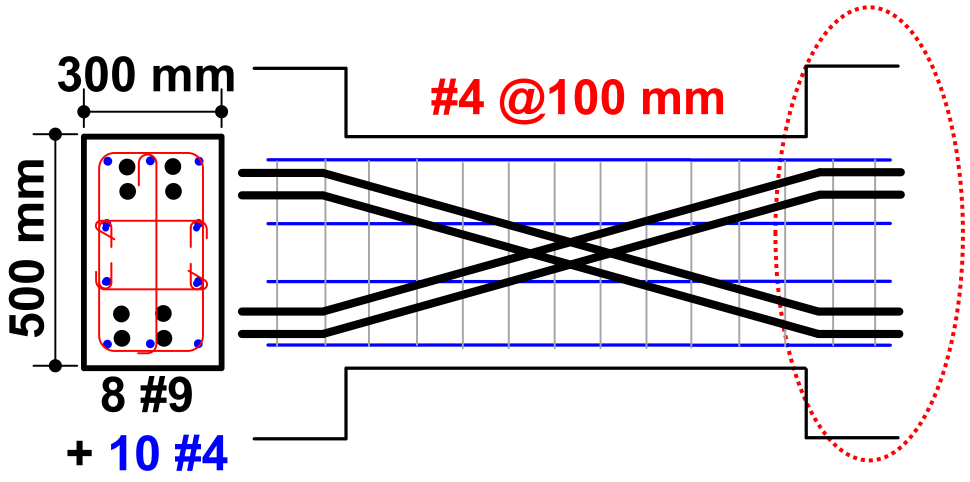
- 對角鋼筋採斜向配置於柱及剪力牆邊界構材內
- 柱及邊界構材內鋼筋壅塞，對角鋼筋綁紮困難

對角鋼筋配置之改善建議

CB10-1

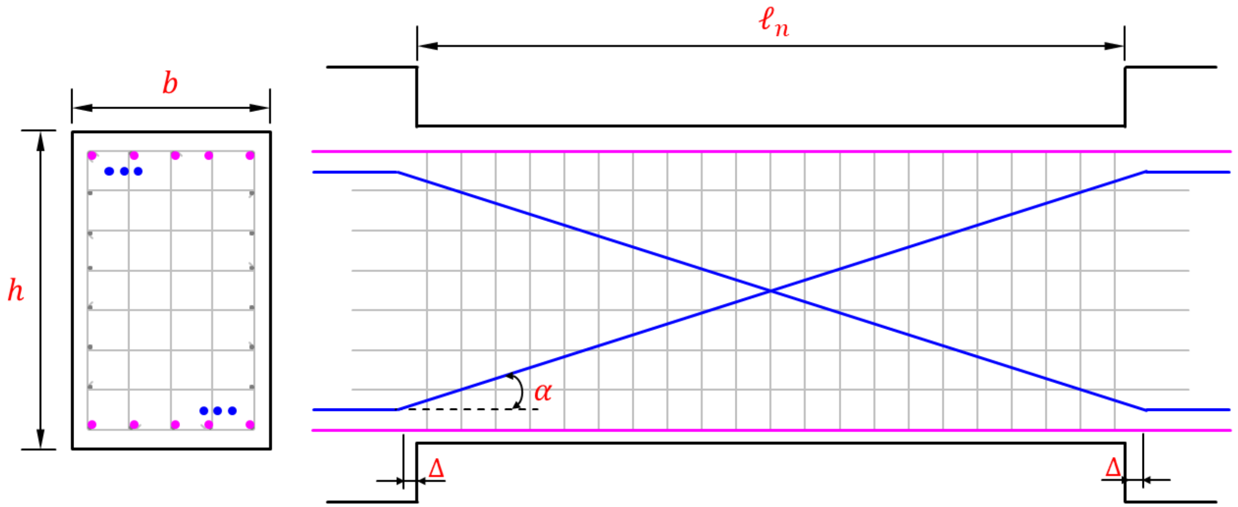


CB20-1

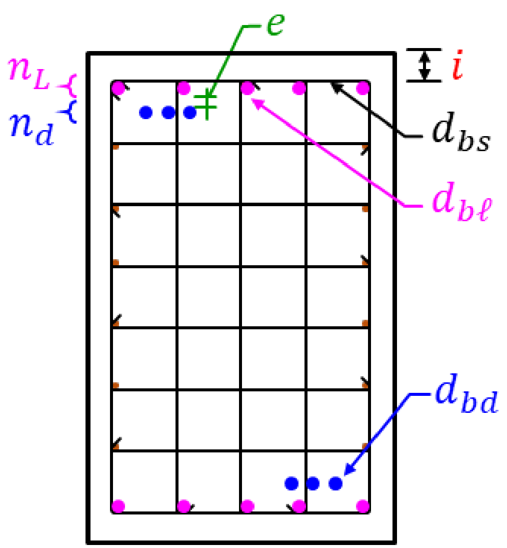


- 對角鋼筋於圍束核心內作偏折處理，採水平向配置於柱及剪力牆邊界構材內
- 對角鋼筋於柱及邊界構材內之鋼筋綁紮較為容易，適合預鑄組件施作

中短連接梁對角鋼筋夾角α計算公式



$$\alpha = \tan^{-1}\left(\frac{h-2\left(i+d_{bs}+n_L d_{b\ell}+2.5(n_L-1)+e+d_{bd}\frac{n_d}{2}+e\frac{(n_d-1)}{2}\right)}{\ell_n+2\Delta}\right)$$



Δ：對角鋼筋延伸進牆體長度的水平分量，至少取5公分

i：混凝土保護層厚度

d_{bs}：箍筋直徑

$d_{b\ell}$：縱向鋼筋直徑

n_L：縱向鋼筋層數

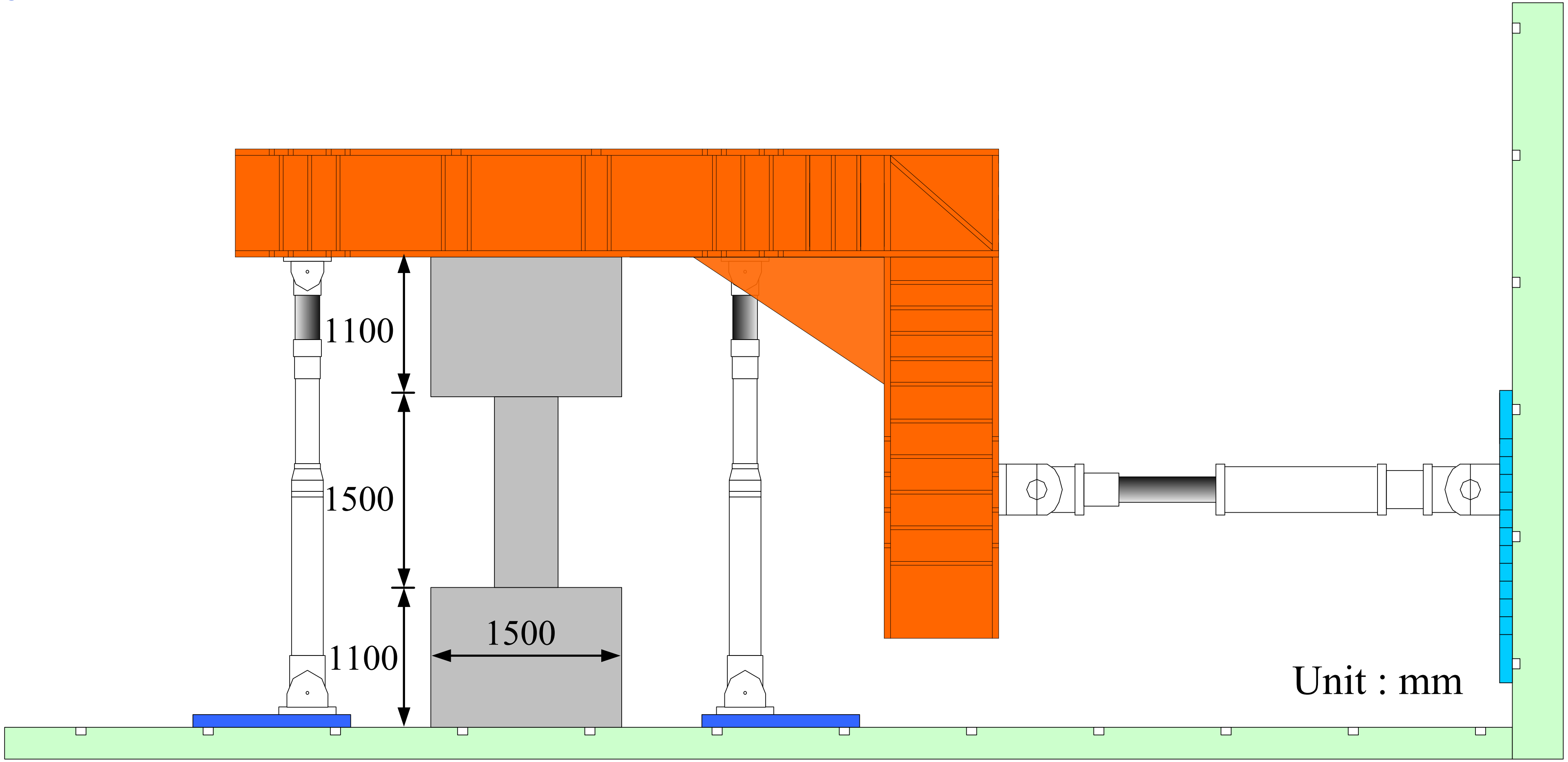
d_{bd}：對角鋼筋直徑

n_d：對角鋼筋層數

$e = \max(2.5\,, d_{bd})$：對角鋼筋層與層之排置間距，或與縱向主筋之淨間距

鋼筋混合配置之建議

$$\frac{\ell_n}{h} = 3.0$$

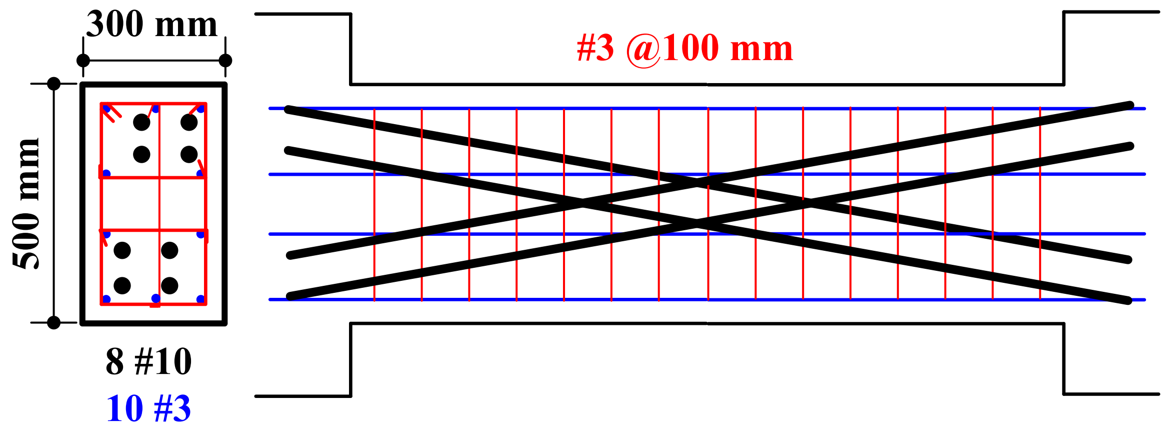


Lim, E., Hwang, S. J., Cheng, C. H., and Lin, P. Y. (2016). “Cyclic Tests of Reinforced Concrete Coupling Beam with Intermediate Span-to-Depth Ratio,” ACI Structural Journal, V. 113, No. 3, May-June, pp. 515-524.

測試試體

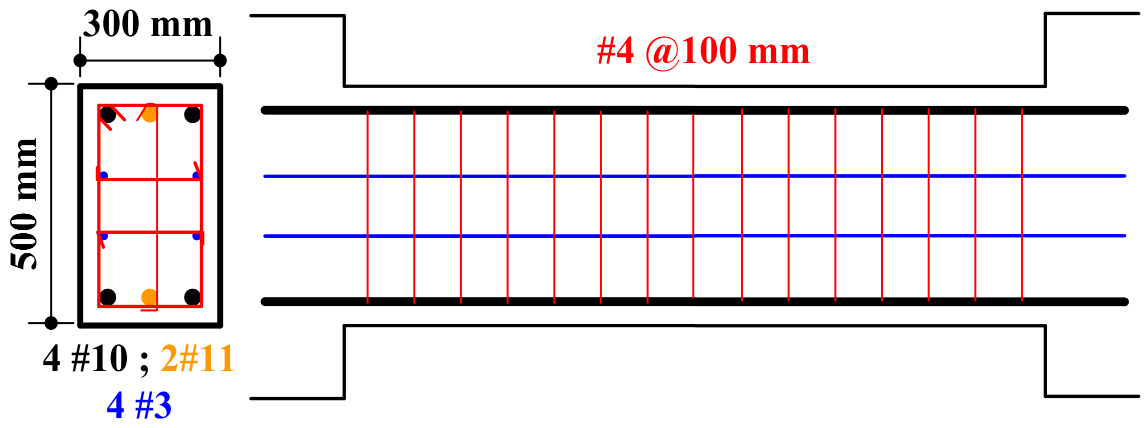
$\ell_n / h = 3.0$

CB30-DB



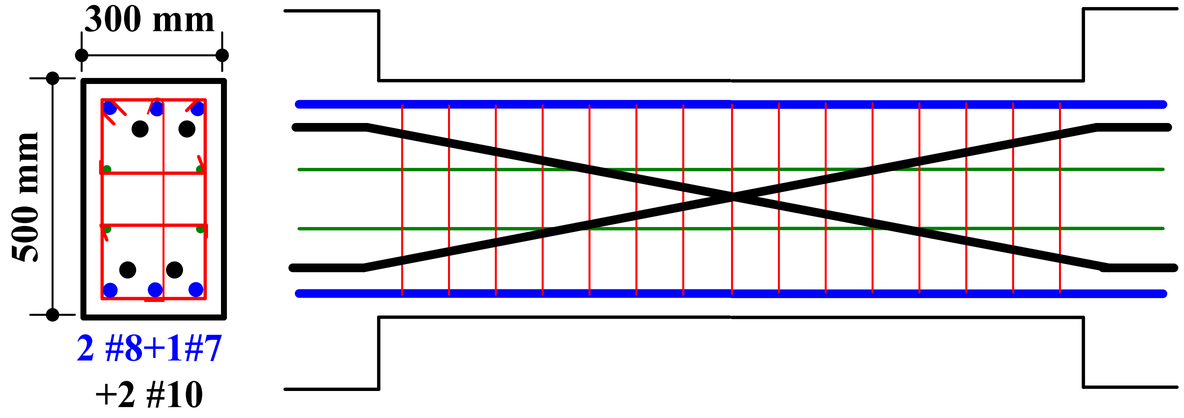
$f_c' = 38.4\ MPa$
$\rho_f = 2.86\%$

CB30-C



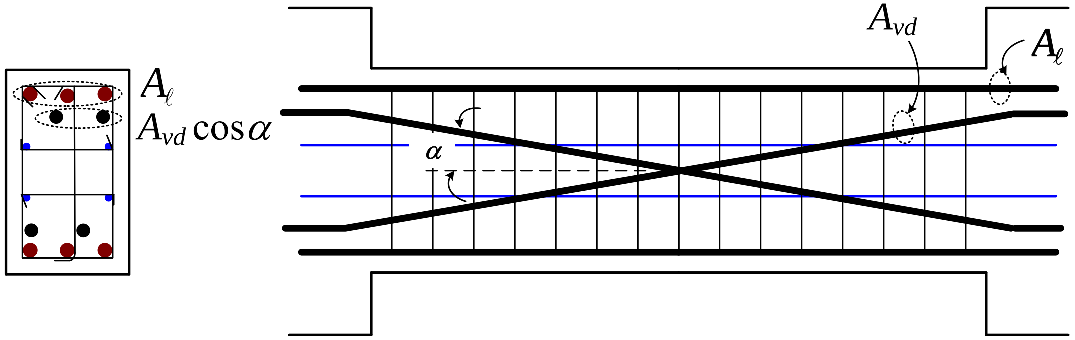
$f_c' = 47.9\ MPa$
$\rho_f = 2.03\%$

CB30-H



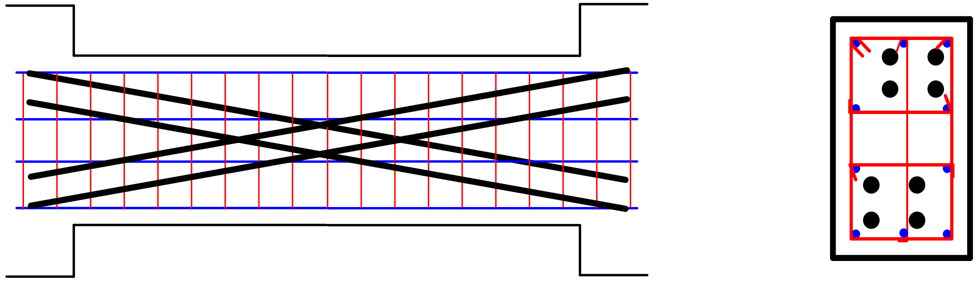
$f_c' = 58.0\ MPa$
$\rho_f = 2.51\%$

混合型配筋



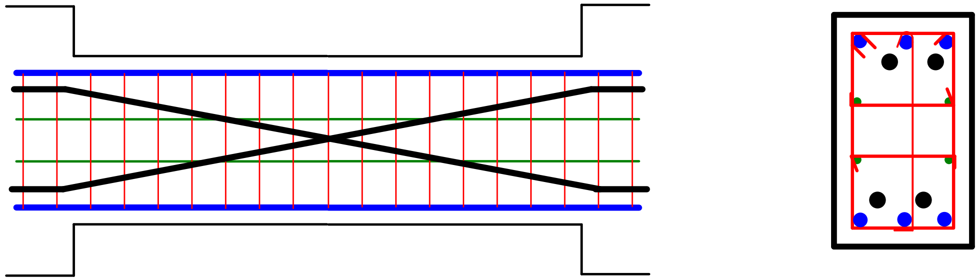
$$\eta = \frac{A_{vd}\cos\alpha\, f_{yd}}{A_{vd}\, f_{yd}\cos\alpha + A_\ell\, f_{y\ell}}$$

CB30-DB



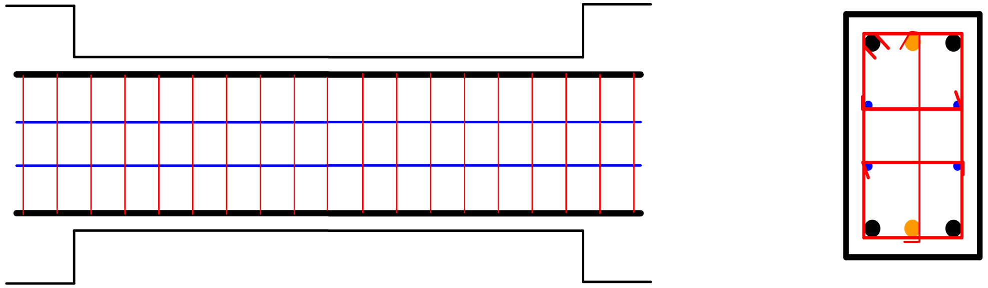
$\eta = 90\%$

CB30-H



$\eta = 54\%$

CB30-C

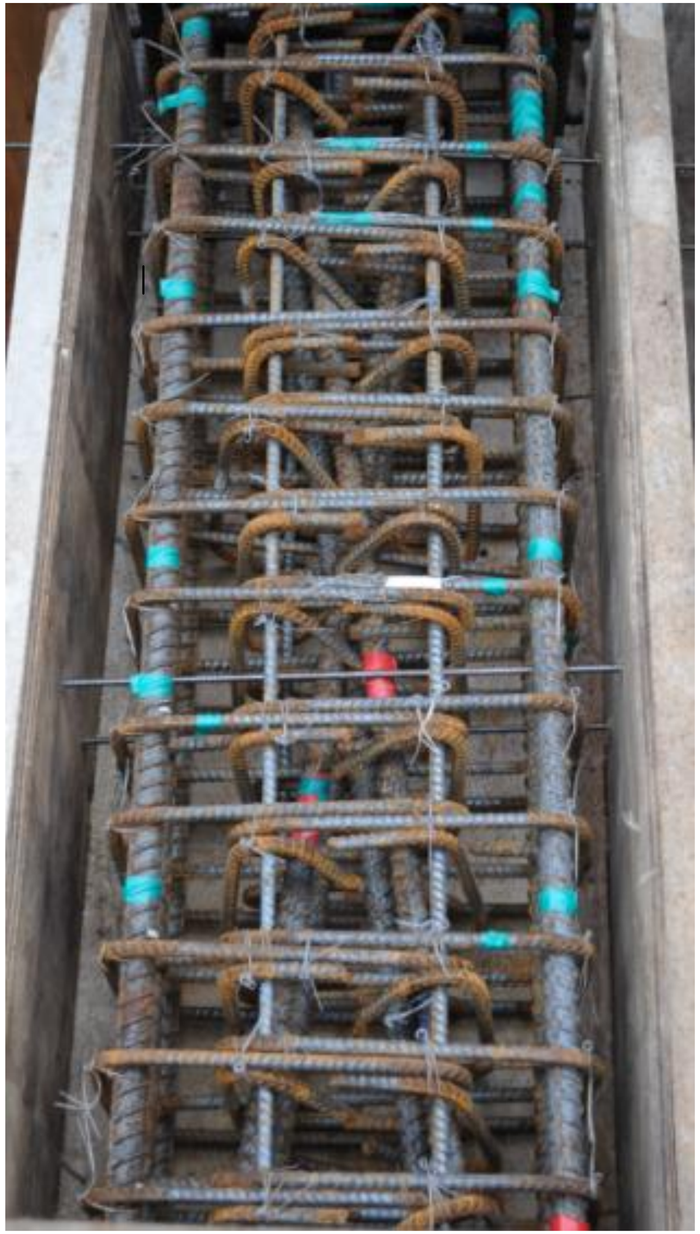


$\eta = 0\%$

連接梁鋼筋配置之施工性

CB30-DB

全對角鋼筋



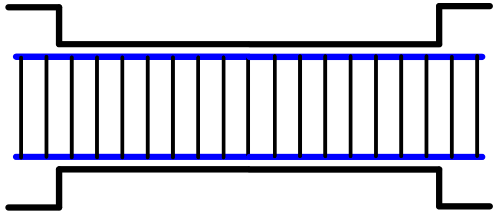
CB30-H

混合型配置

施工性較佳

試驗結果

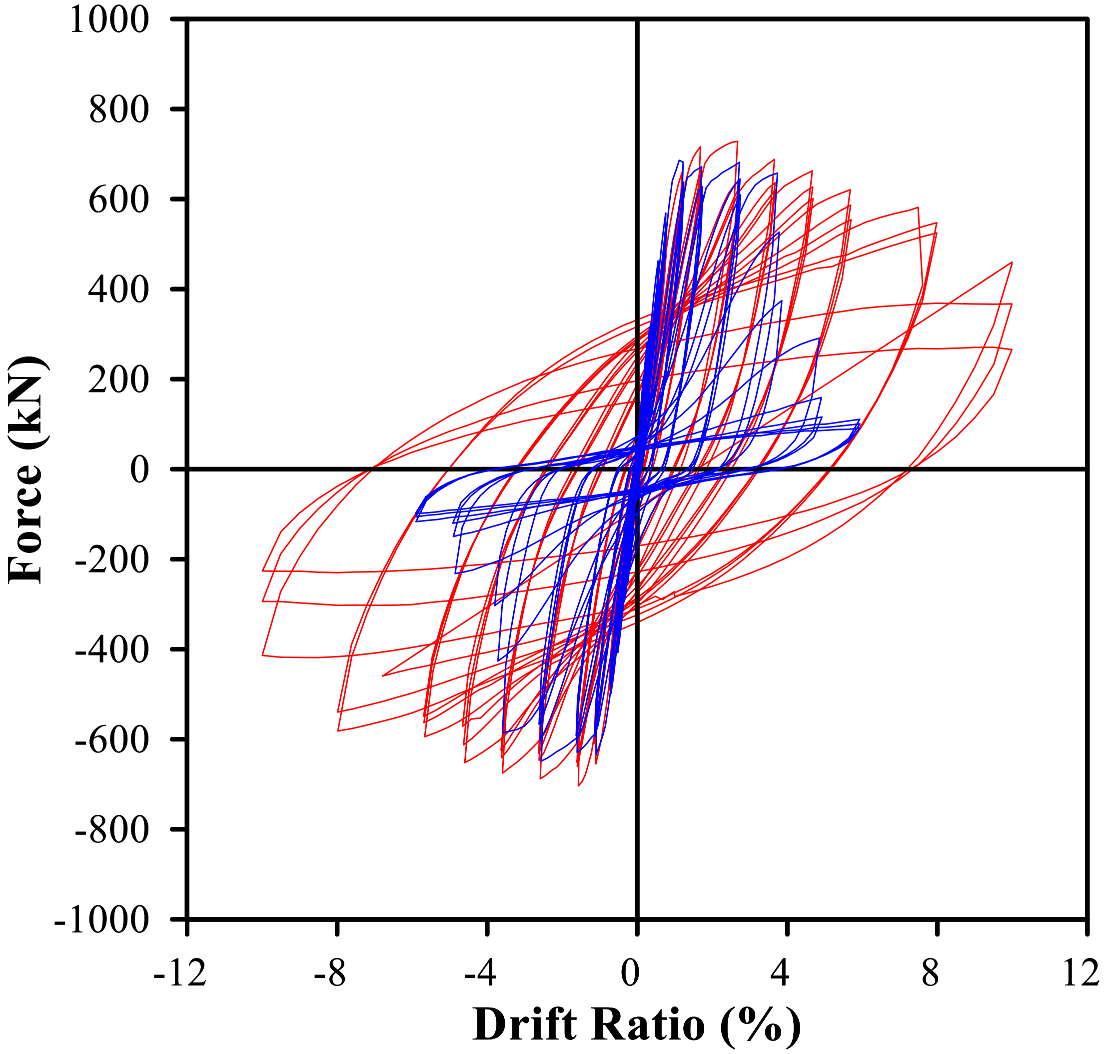
CB30-C



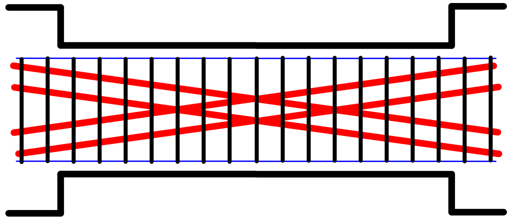
4.0%

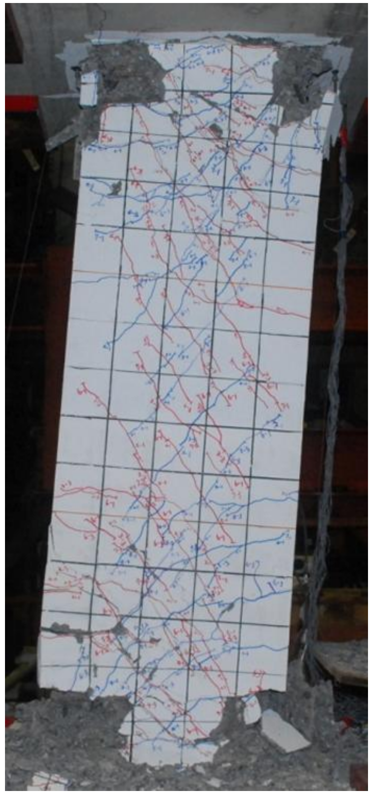
$$\upsilon = 0.75\sqrt{f_c'}$$

UDR = 3.8%



CB30-DB





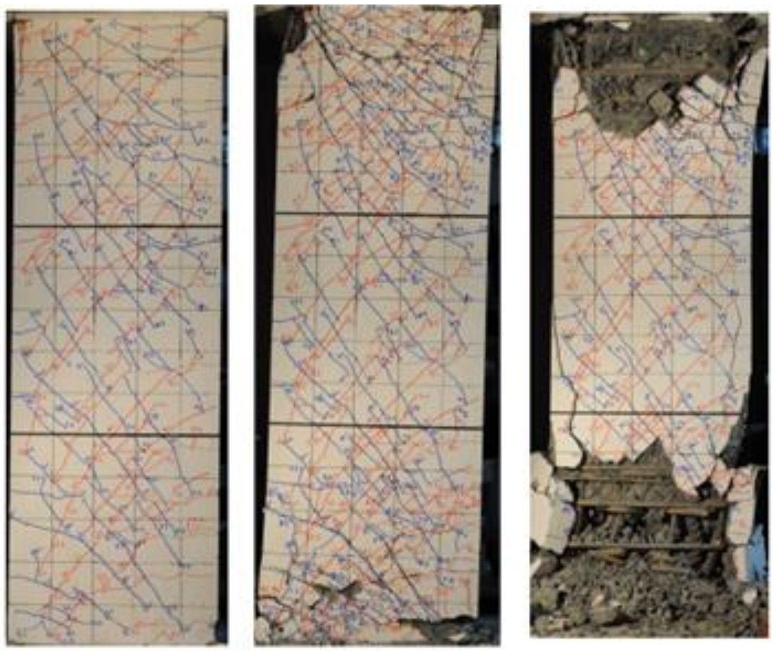
8.0%

$$\upsilon = 1.04\sqrt{f_c'}$$

UDR = 7.5%

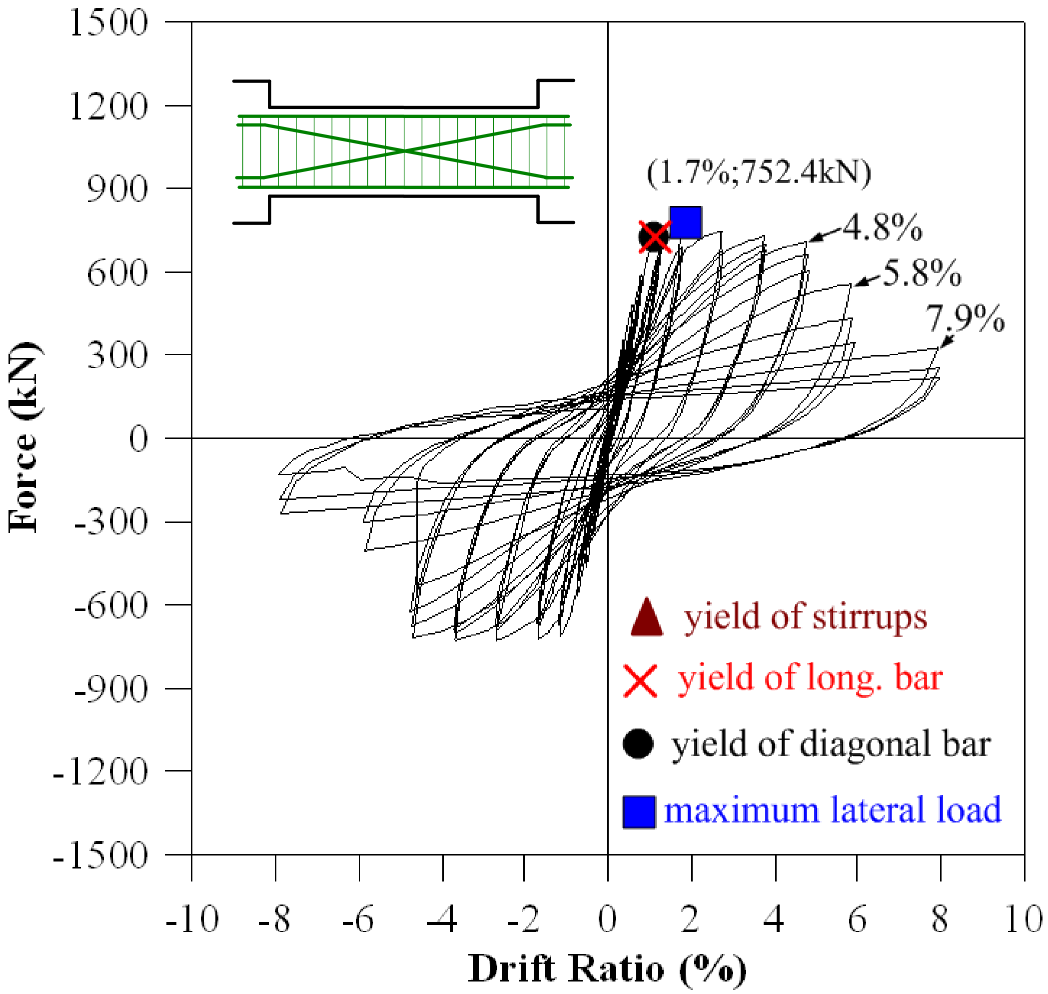
混合型鋼筋配置梁之測試結果

CB30-H

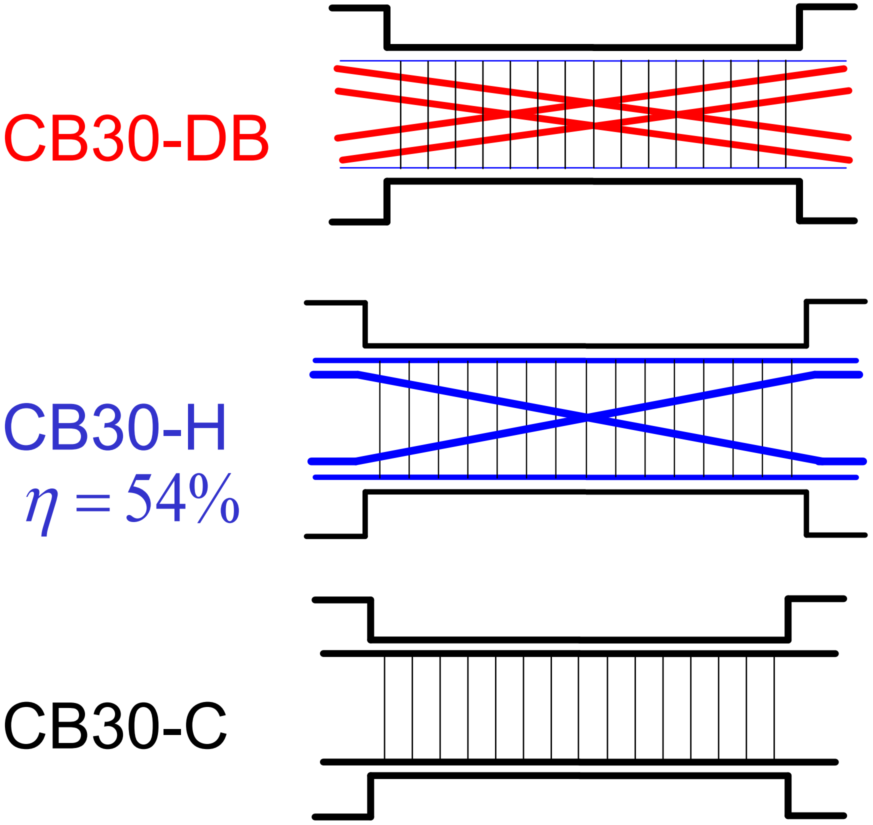


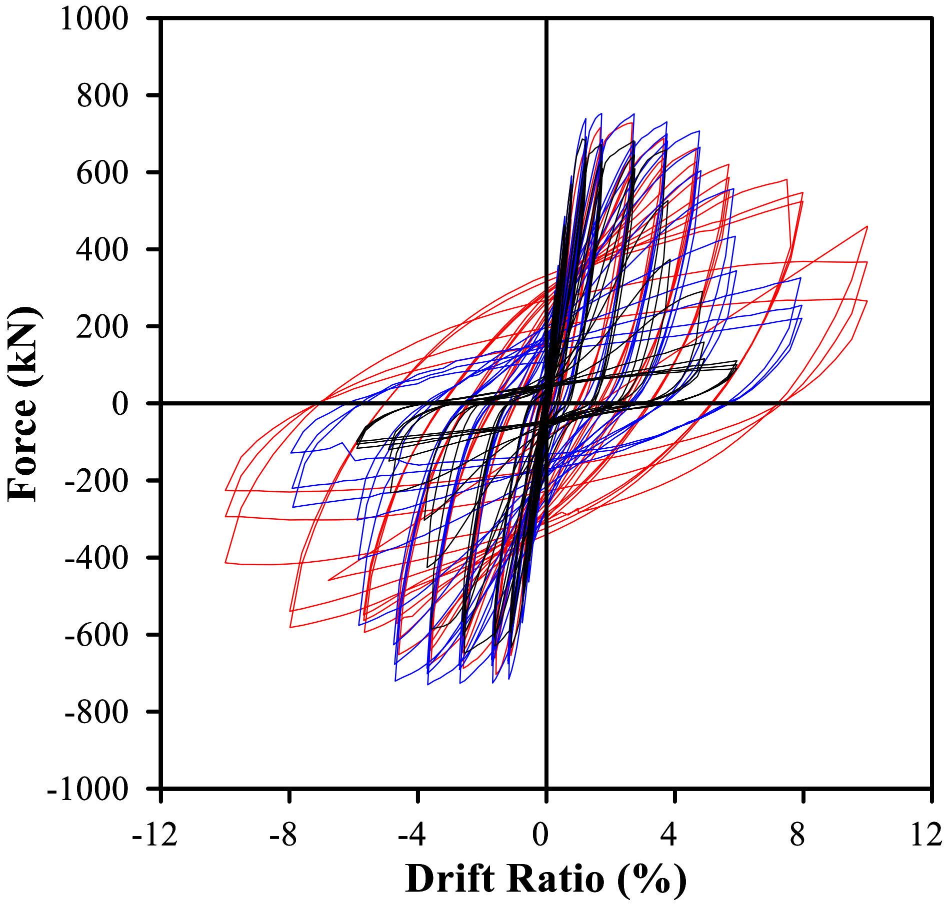
$$\upsilon = 0.81\sqrt{f_c'}$$

UDR=5.7%



Flexure shear failure



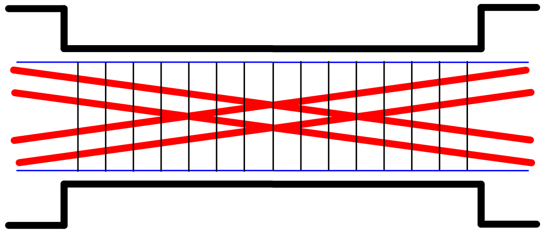


	F.M	Ult. DR	$\upsilon_{max}[MPa]$
CB30-DB	Flexure	7.5%	$0.99\sqrt{f_c'}$
CB30-H	Flexure Shear	5.7%	$0.81\sqrt{f_c'}$
CB30-C	Flexural Shear	3.8%	$0.77\sqrt{f_c'}$

連接梁轉角性能之比較

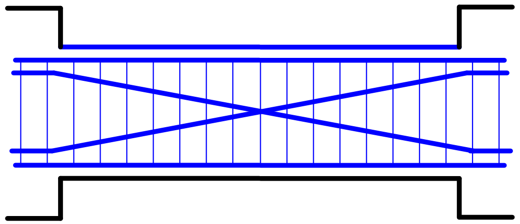
$\eta = 90\%$

全對角



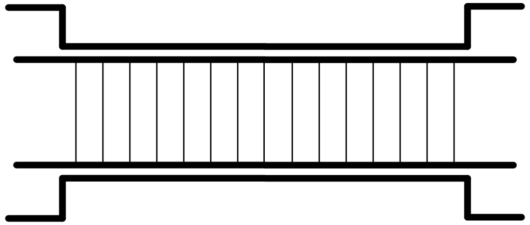
$\eta = 54\%$

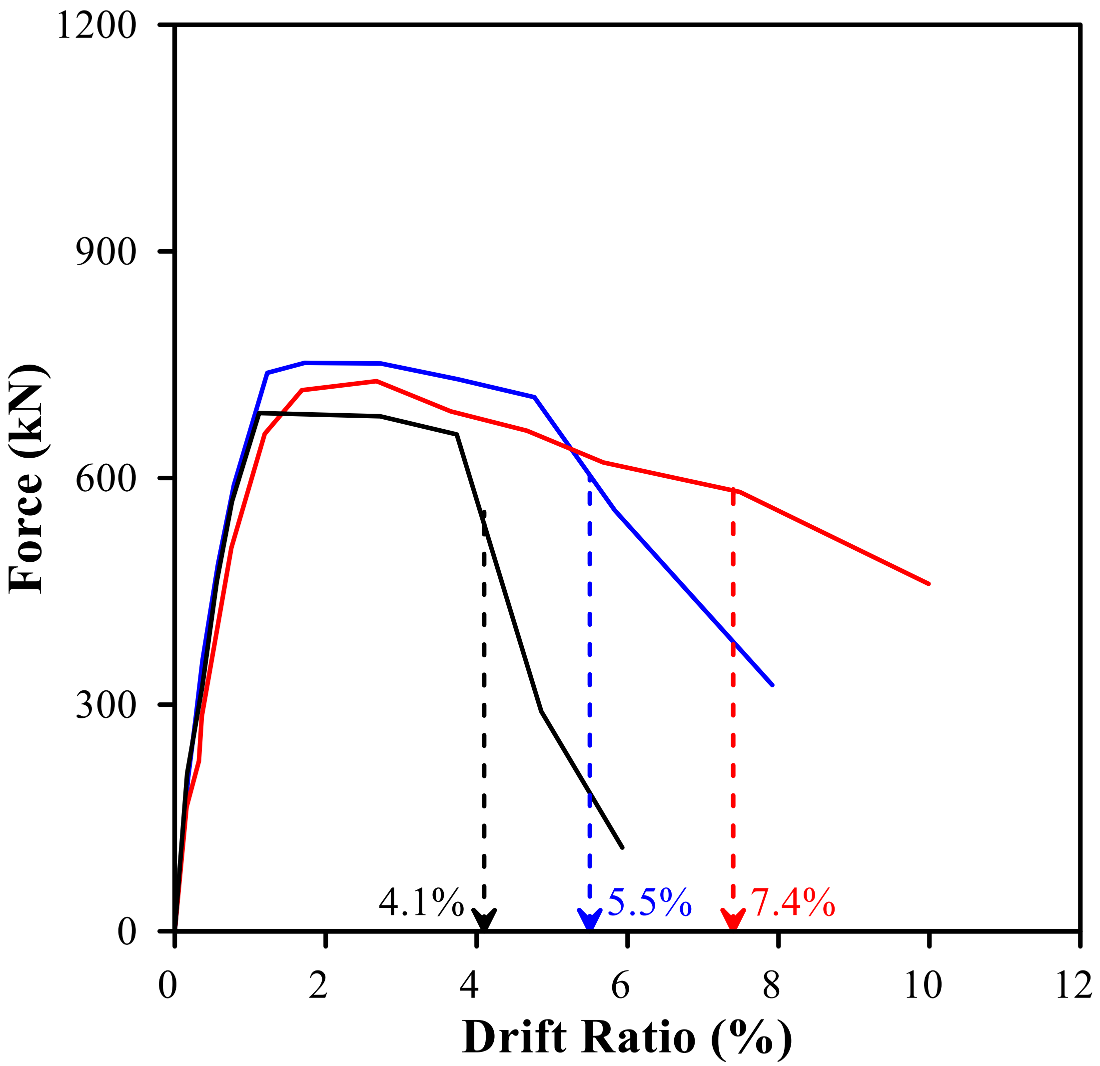
混合



$\eta = 0\%$

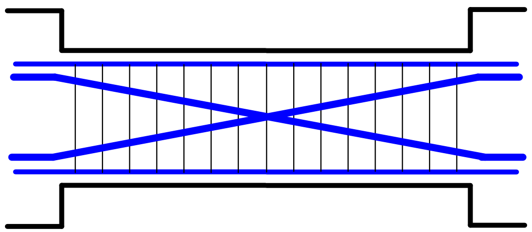
傳統

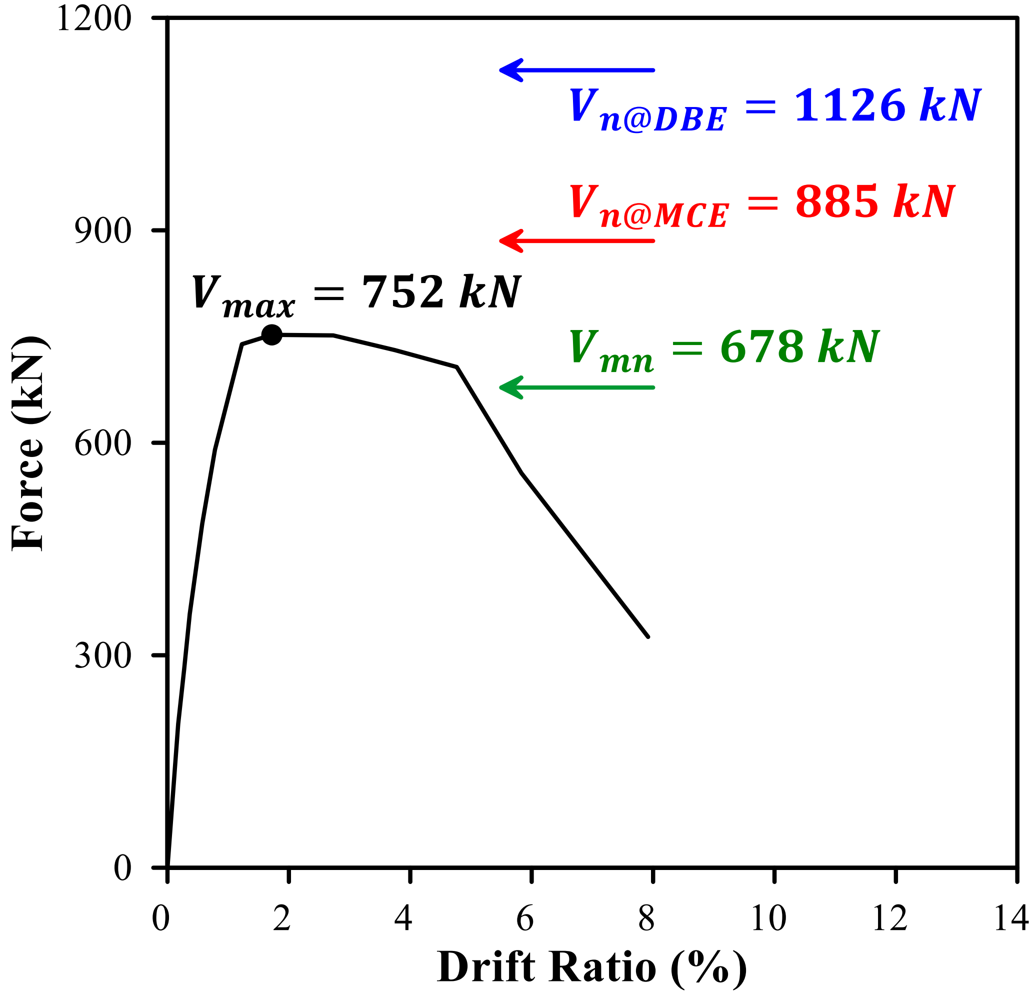




$\ell_n / h = 3.0$

CB30-H



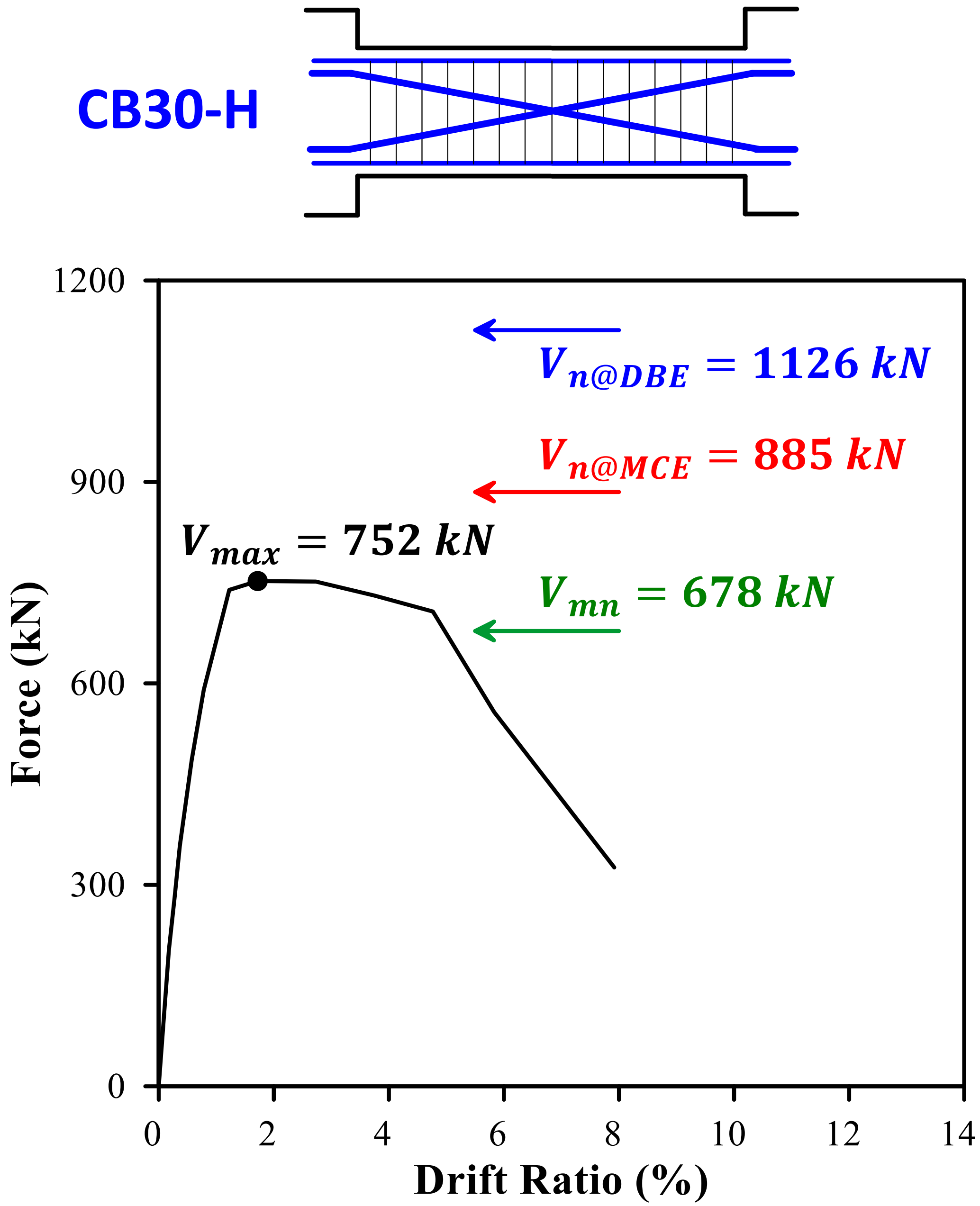


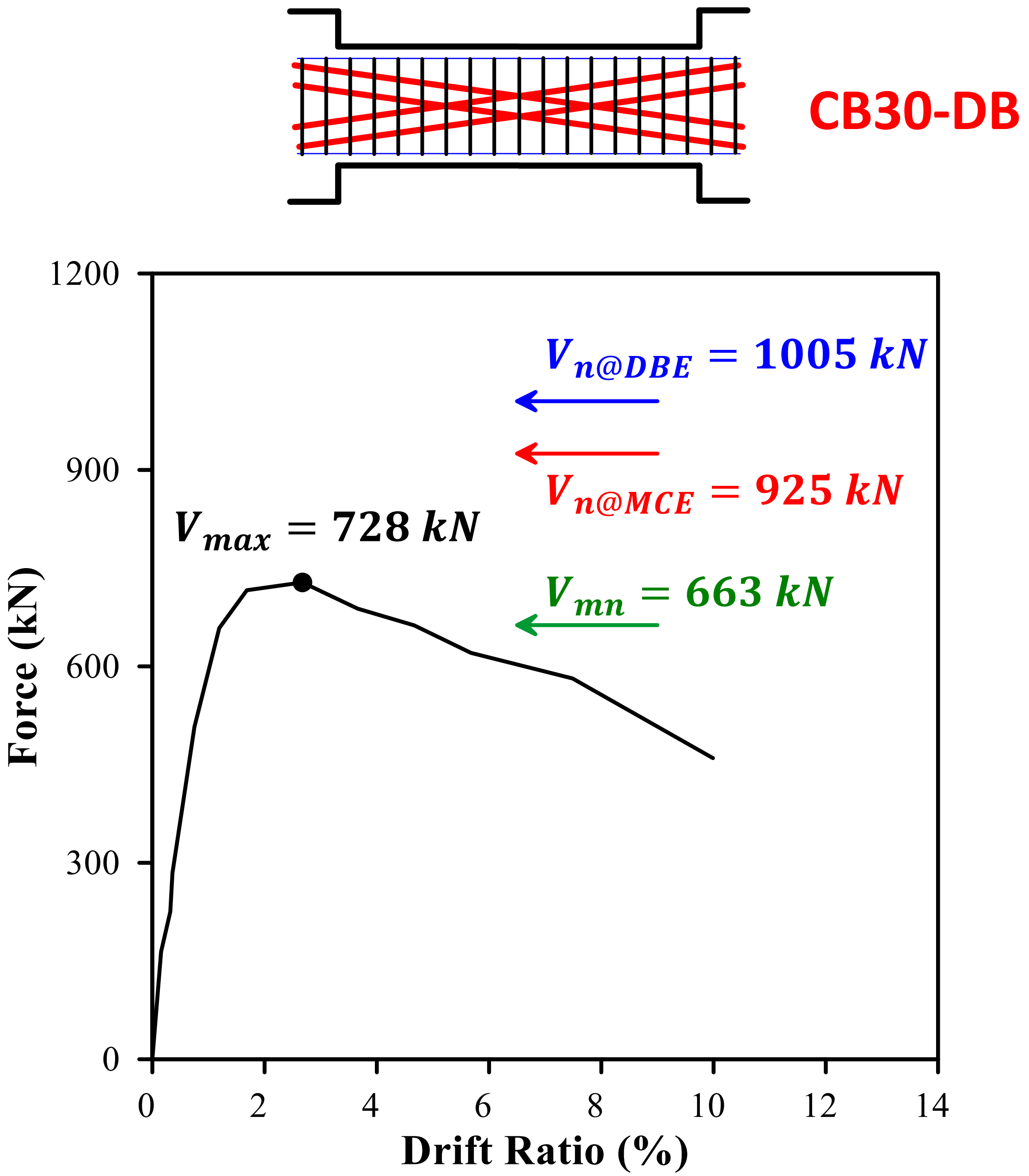
撓曲與剪力強度評估

1. $V_{mn} < V_{n@DBE}$
2. 撓曲強度主控行為
3. $\frac{V_{max}}{V_{mn}} = 1.11$
4. $V_{n@MCE} > V_{max}$
5. $\frac{V_{n,@MCE}}{V_{max}} = 1.18$
6. 撓曲塑鉸變形充分發展

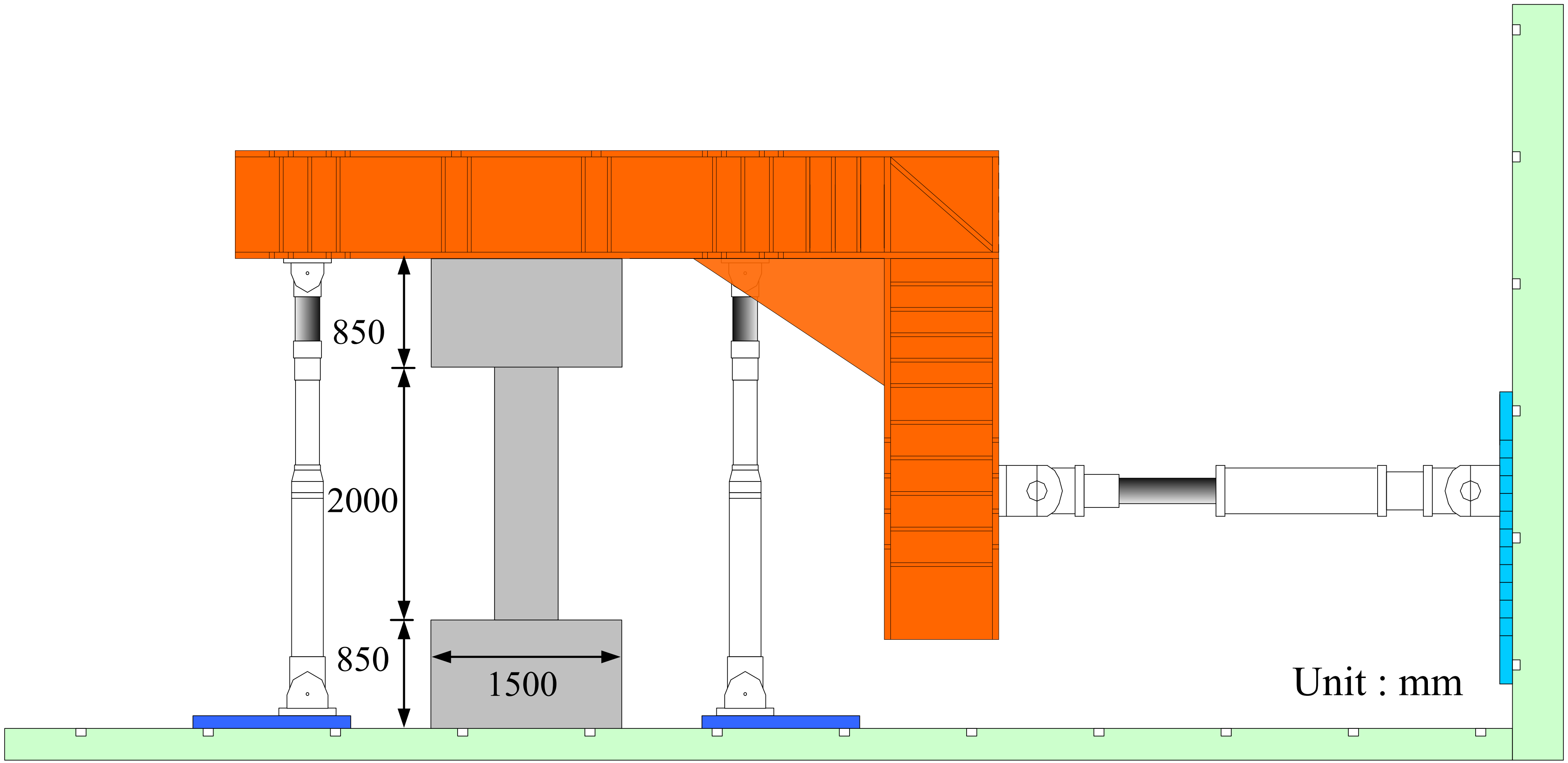
$\ell_n / h = 3.0$

測試行為評估





$$\frac{\ell_n}{h} = 4.0$$

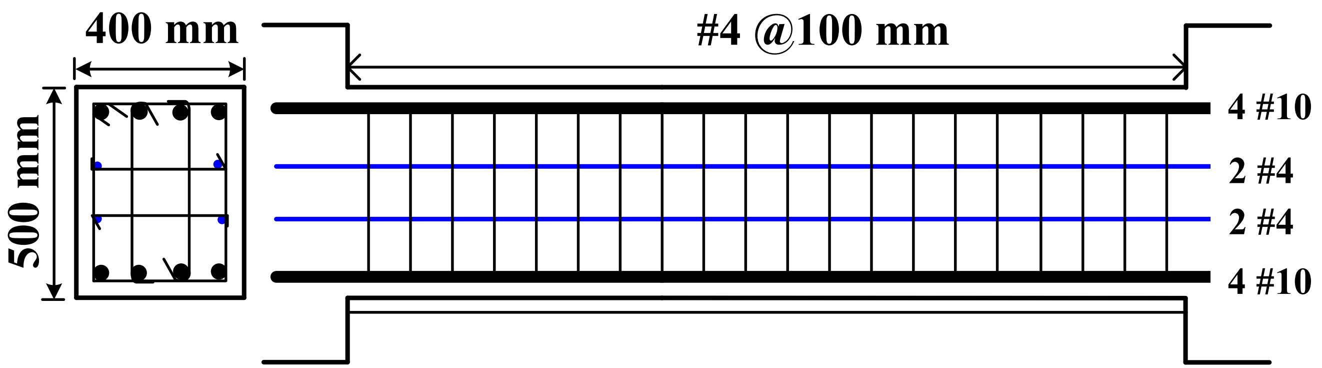


Lim, E., Hwang, S. J., Cheng, C. H., and Lin, P. Y. (2016). “Cyclic Tests of Reinforced Concrete Coupling Beam with Intermediate Span-to-Depth Ratio,” ACI Structural Journal, V. 113, No. 3, May-June, pp. 515-524.

$\ell_n / h = 4.0$

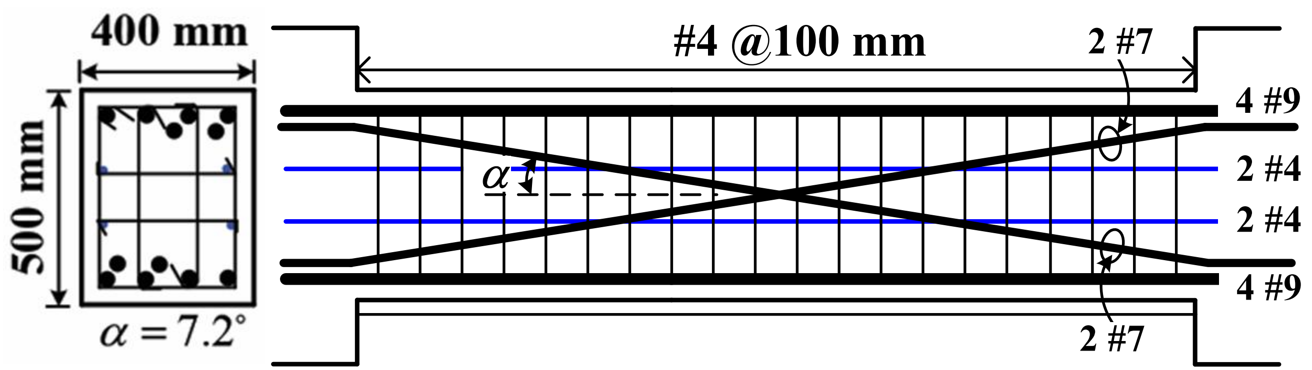
測試試體

CB40-C



$f_c' = 58.1\ MPa$
$\rho_f = 1.18\%$

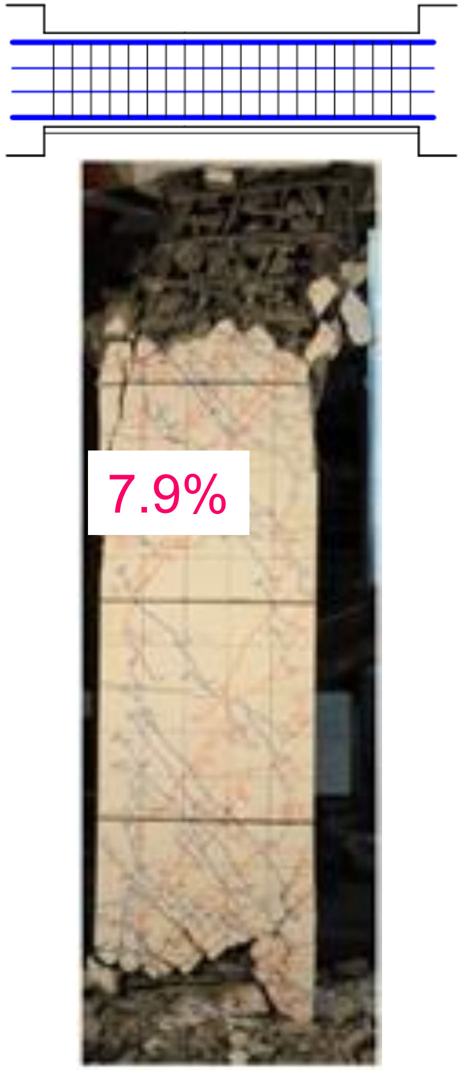
CB40-H



$f_c' = 58.5\ MPa$
$\rho_f = 1.29\%$

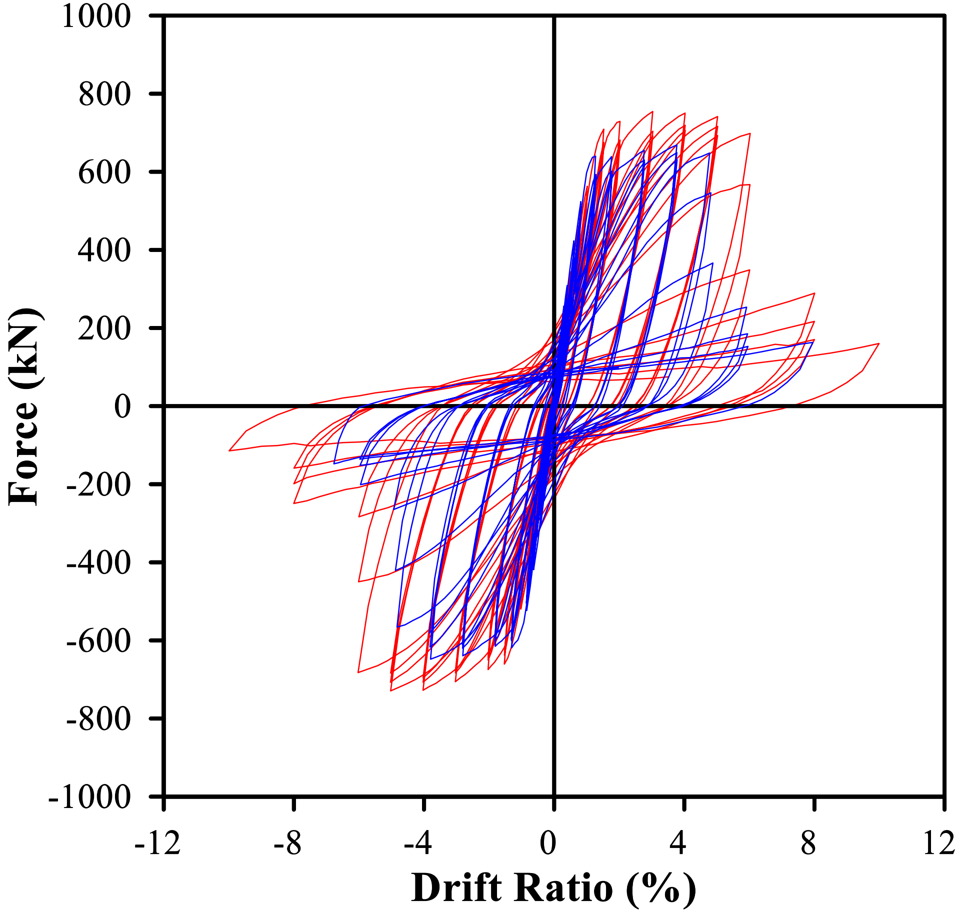
試驗結果

CB40-C

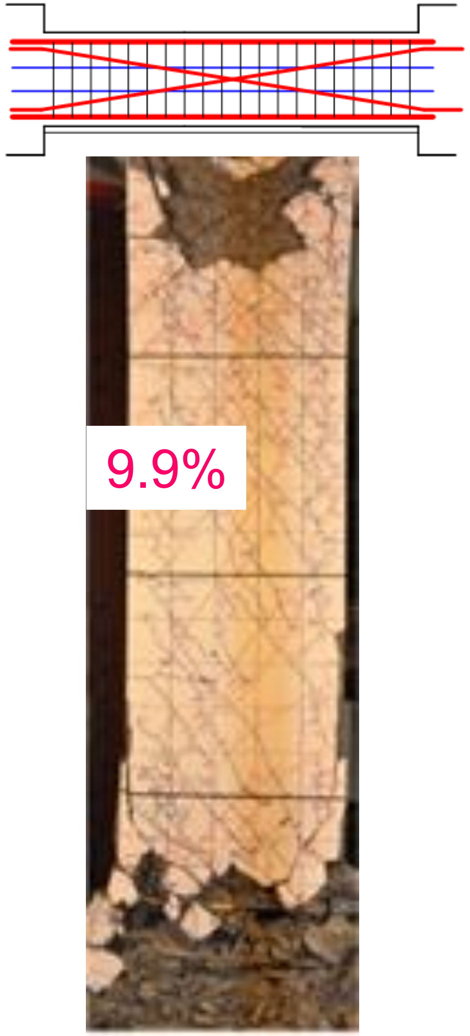


$$\upsilon = 0.51\sqrt{f_c'}$$

UDR=5.1%



CB40-H



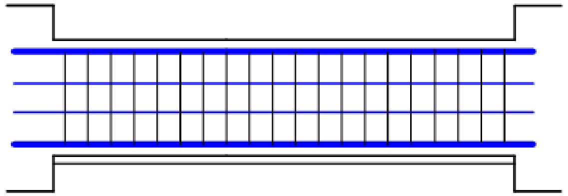
$$\upsilon = 0.6\sqrt{f_c'}$$

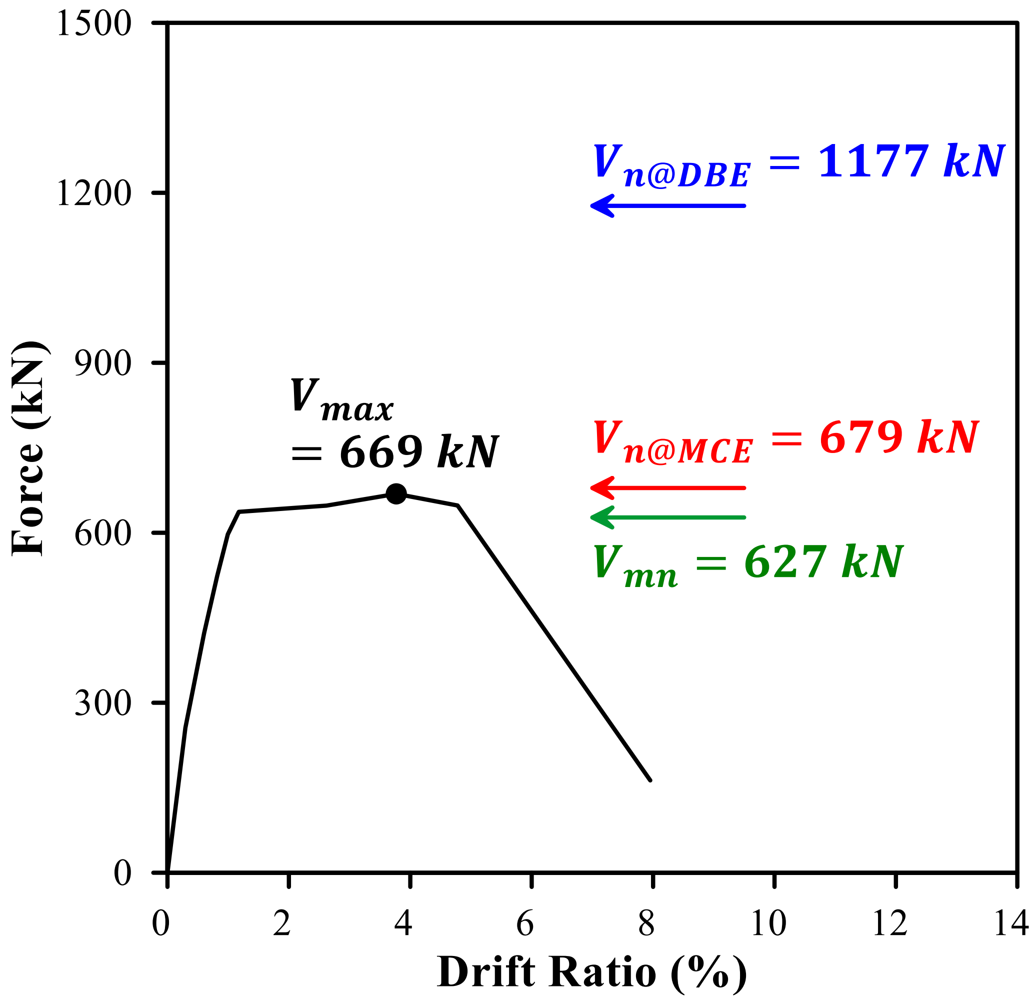
UDR=6.3%

撓曲與剪力強度評估

$\ell_n/h = 4.0$

CB40-C

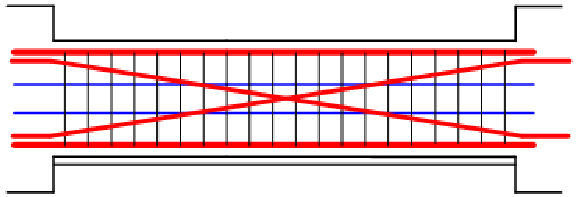


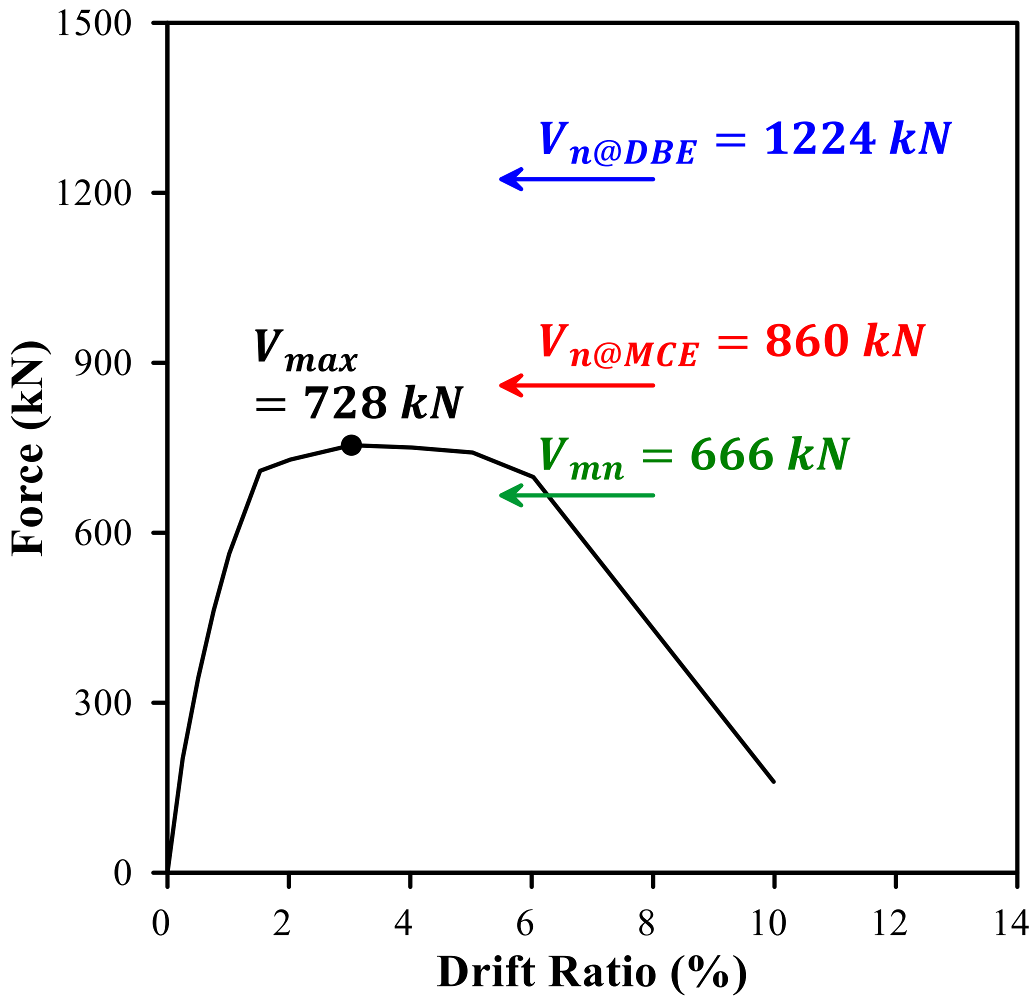


1. $V_{mn} < V_{n@DBE}$
2. 撓曲強度主控行為
3. $\dfrac{V_{max}}{V_{mn}} = 1.07$
4. $V_{n@MCE} > V_{max}$
5. $\dfrac{V_{n,@MCE}}{V_{max}} = 1.02$
6. 撓曲塑鉸變形可以發展

撓曲與剪力強度評估

$\ell_n/h = 4.0$

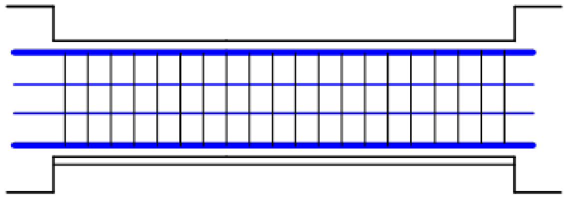


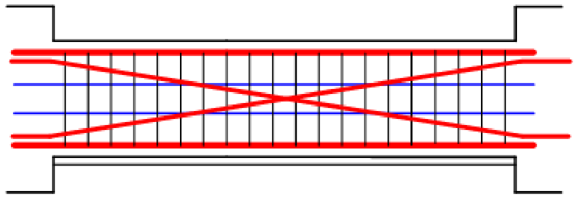


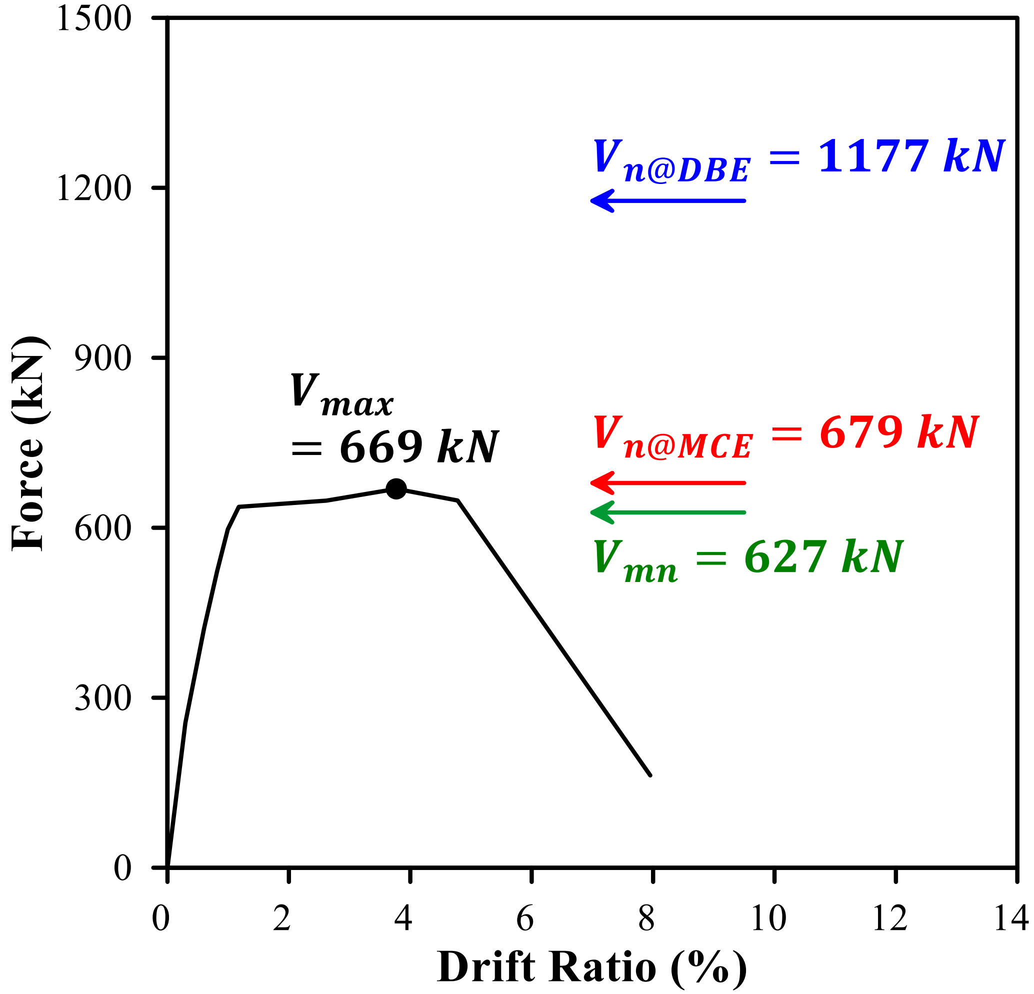
1. $V_{mn} < V_{n@DBE}$
2. 撓曲強度主控行為
3. $\frac{V_{max}}{V_{mn}} = 1.09$
4. $V_{n@MCE} > V_{max}$
5. $\frac{V_{n,@MCE}}{V_{max}} = 1.18$
6. 撓曲塑鉸變形充分發展

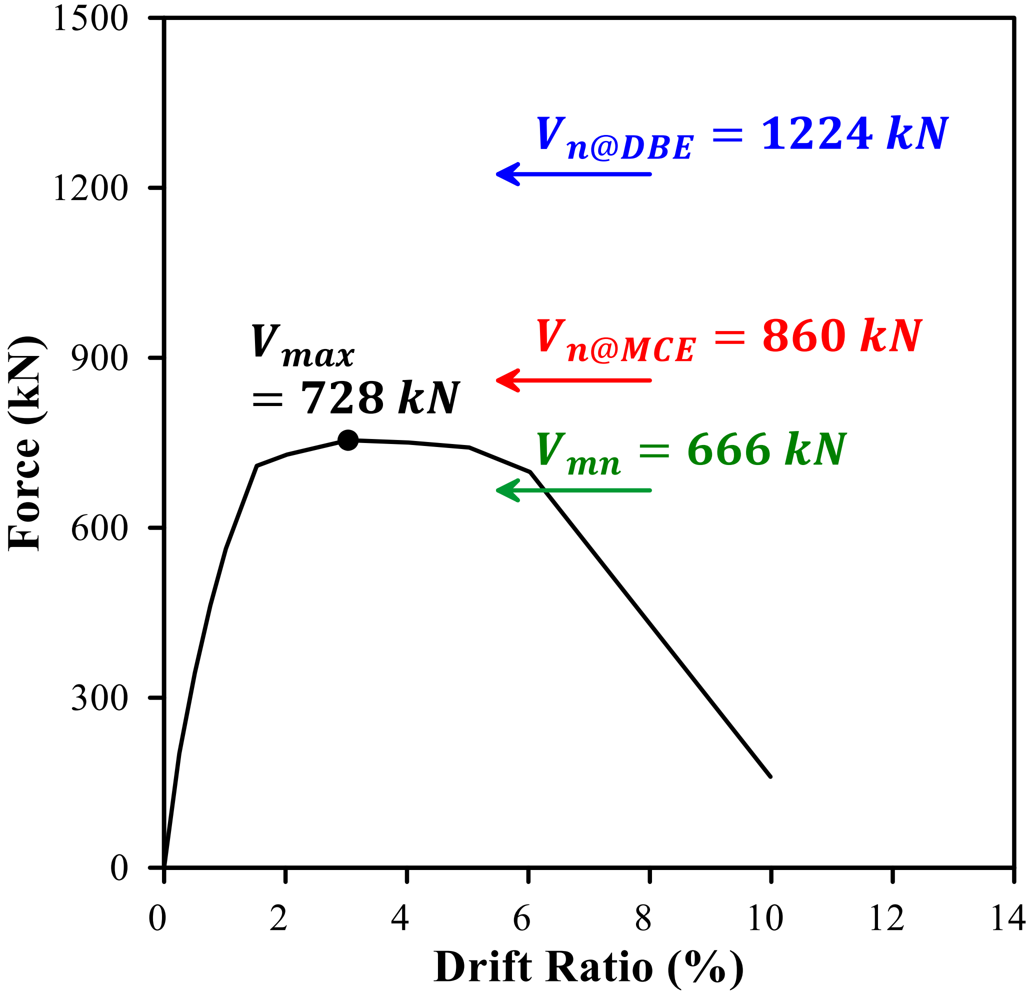
測試行為評估

$\ell_n / h = 4.0$



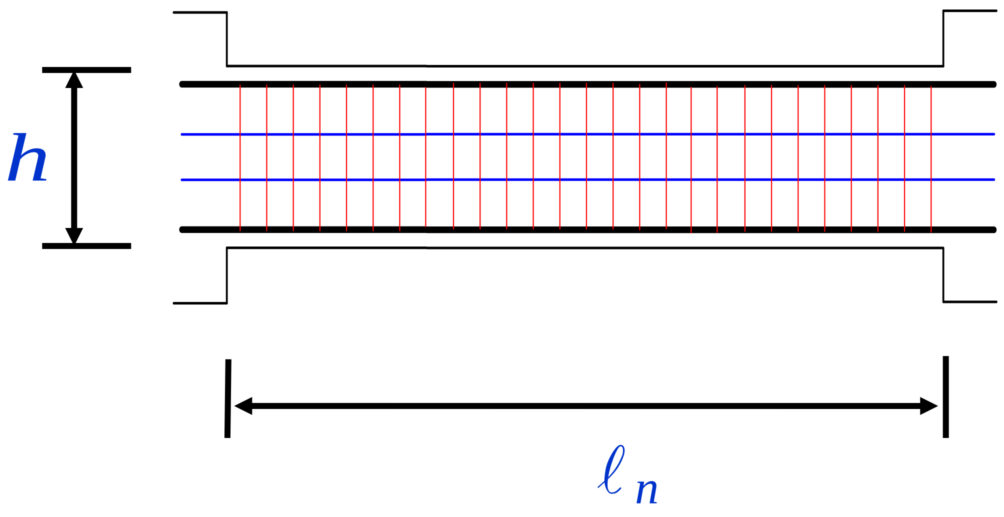


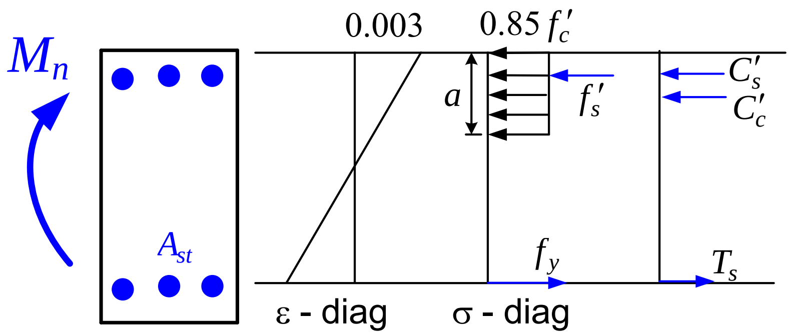




中短連接梁之設計建議

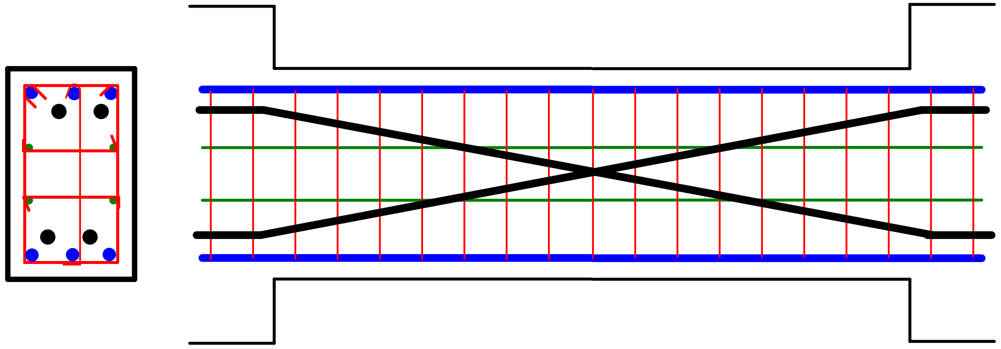
$\ell_n / h > 4$ 傳統配筋連接梁之設計





設計地震 Design Based Earthquake **DBE**	$M_n \geq \frac{M_u}{\phi}$	選定破壞機制 並予以控制
最大可能地震 Max. Credible Earthquake **MCE**	$V_n = 0 + A_v \times f_y \times \frac{d}{s} \geq \frac{V_p}{\phi}$	剪力容量設計

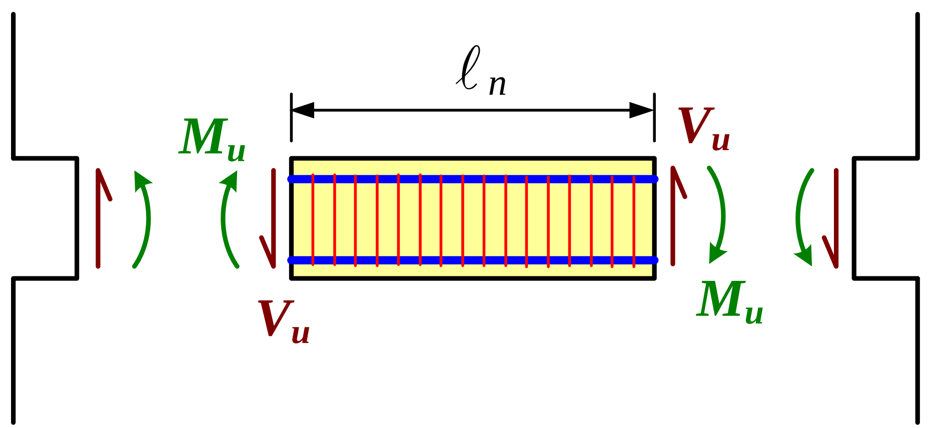
$2 \le \ell_n / h \le 4$ 混合型(Hybrid)鋼筋連接梁之設計



- A_ℓ 撓曲鋼筋
- A_{vd} 對角鋼筋

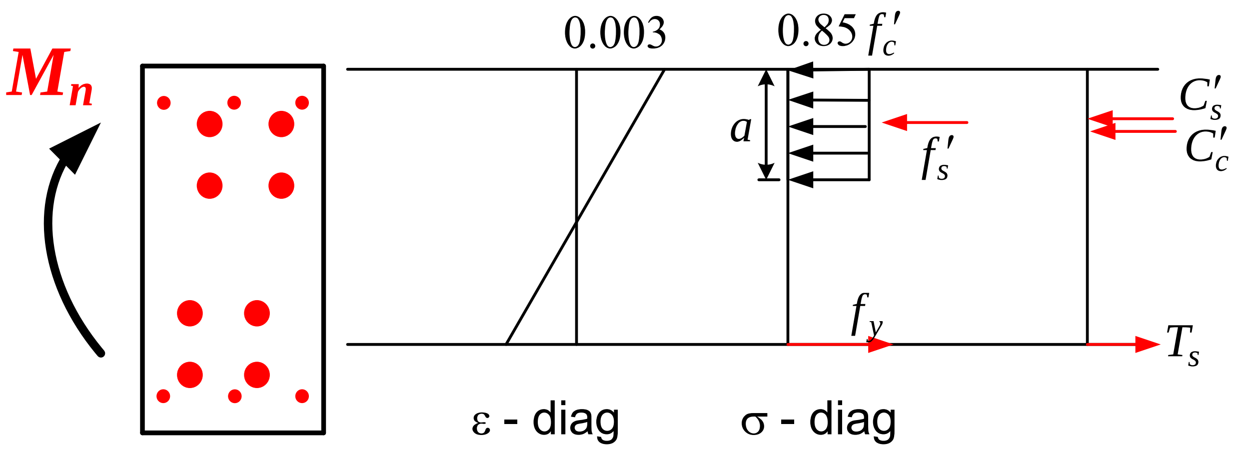
設計地震 Design Based Earthquake **DBE**	$M_n \ge M_u / \phi$ $A_{st} \times f_y \times jd \ge M_u / 0.9$	計算撓曲鋼筋總量 A_{st}
最大可能地震 Max. Credible Earthquake **MCE**	$C_d \sin 45° + 2.5 A_{vd} f_y \sin\alpha \ge V_p / \phi$ $A_\ell = A_{st} - A_{vd} \cos\alpha$	剪力容量設計 計算對角鋼筋 A_{vd} 計算撓曲鋼筋 A_ℓ

計算撓曲鋼筋總量 − A_{st}



Demand (DBE): M_u

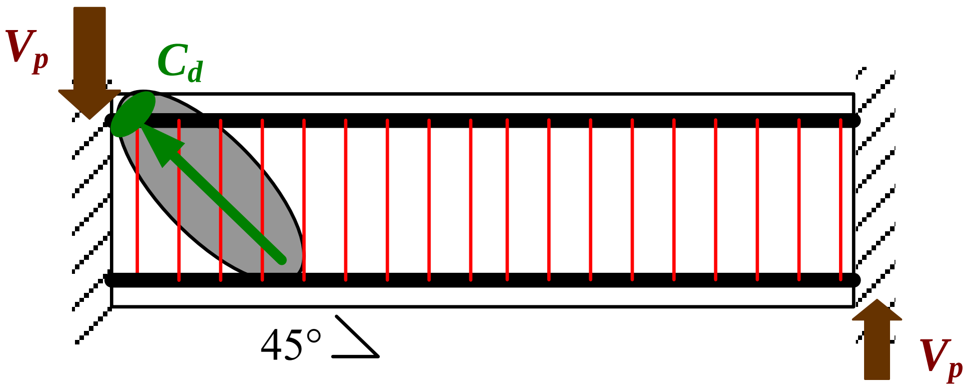
Flexural Strength :



$$\phi M_n \geq M_u$$

Solve for: A_{st}

壓桿抗剪強度是否足夠

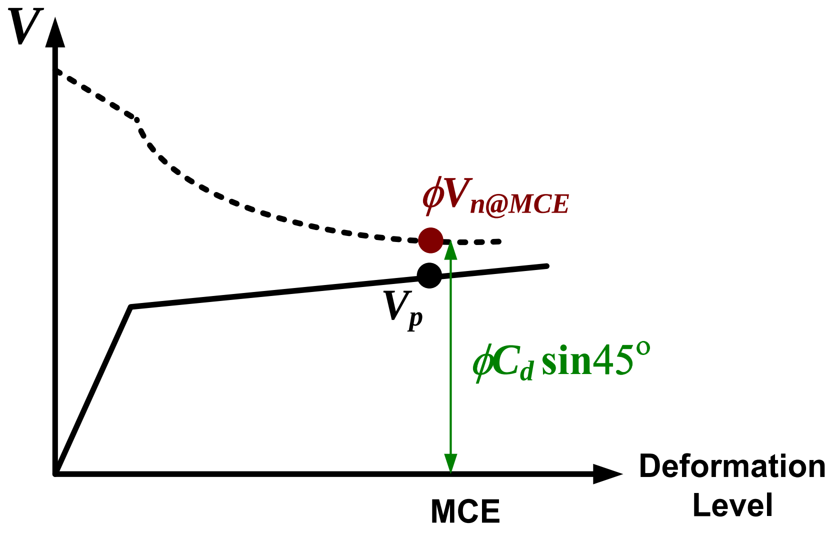


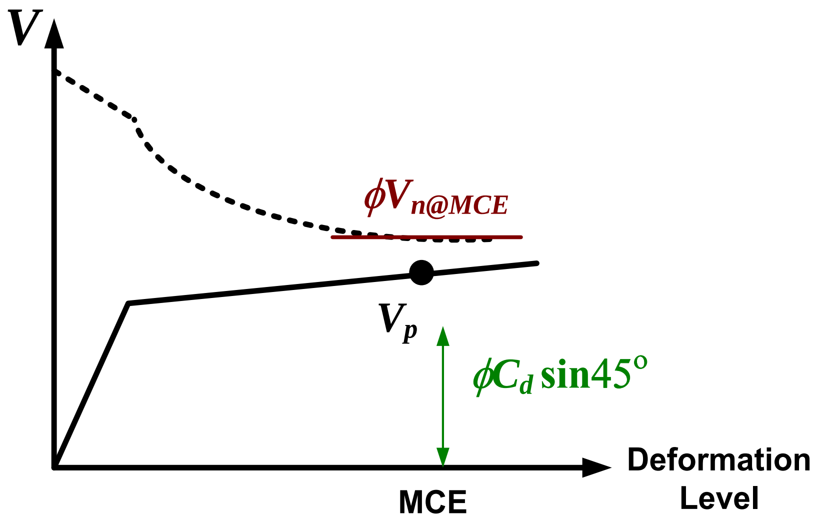
$$C_d \sin 45^o = K\zeta f_c' A_{str} \sin 45^o$$

$$Case\ 1:\ \phi \times C_d \sin 45° \geq V_p$$

或

$$Case\ 2:\ \phi \times C_d \sin 45° < V_p$$





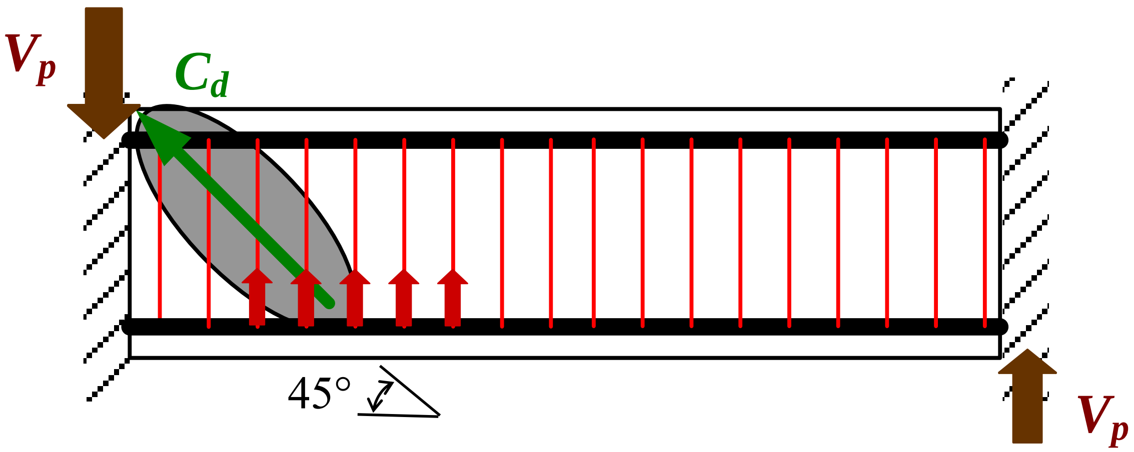
無需對角鋼筋，ϕ=0.75

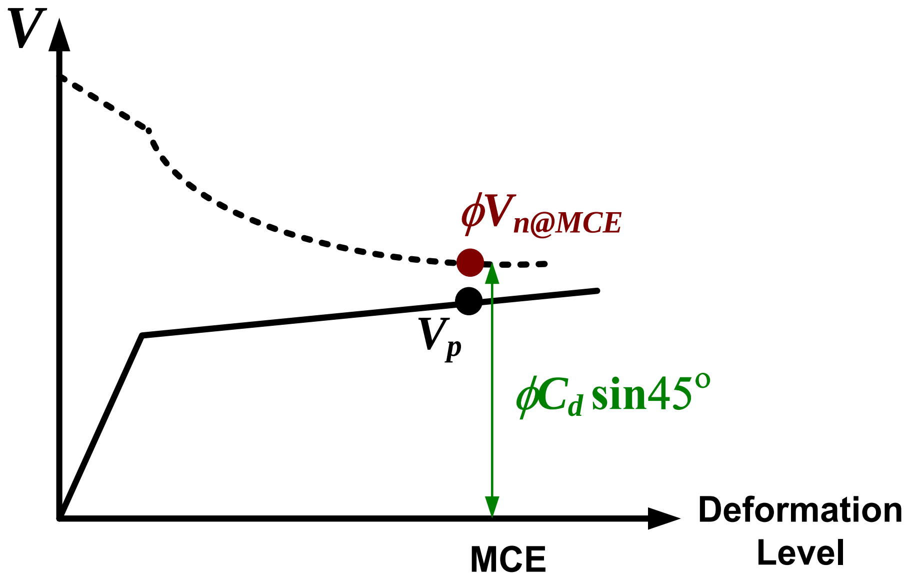
配置對角鋼筋，ϕ=0.85

Case 1: 傳統配筋

$$C_d \sin 45^o > \frac{V_p}{\phi}$$

（壓桿強度足夠）





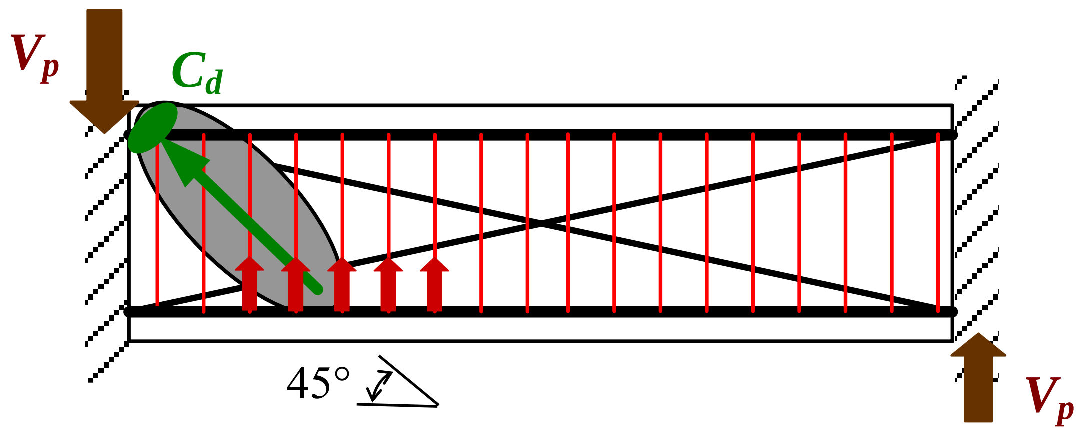
內部抗拉支撐： $V_s = A_{vt} f_{yt} \dfrac{d}{s} \geq \dfrac{V_p}{\phi}$

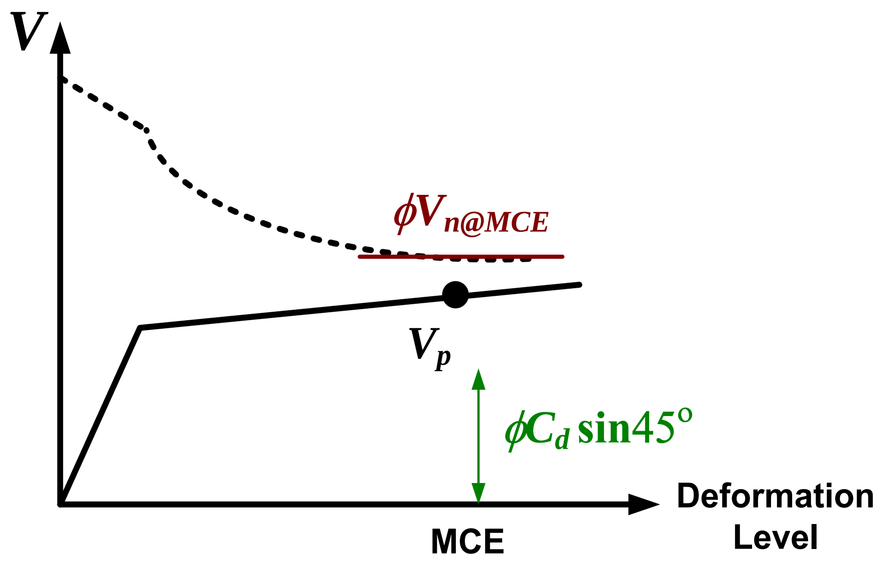
$\phi = 0.75$

Case 2: 對角鋼筋配筋

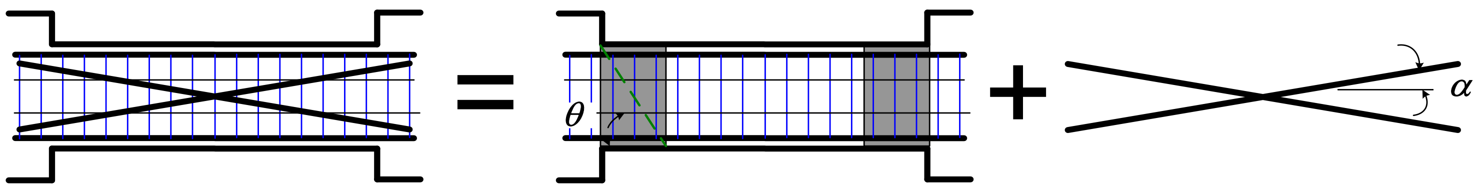
$$C_d \sin 45^o < \frac{V_p}{\phi}$$

（壓桿強度不夠）



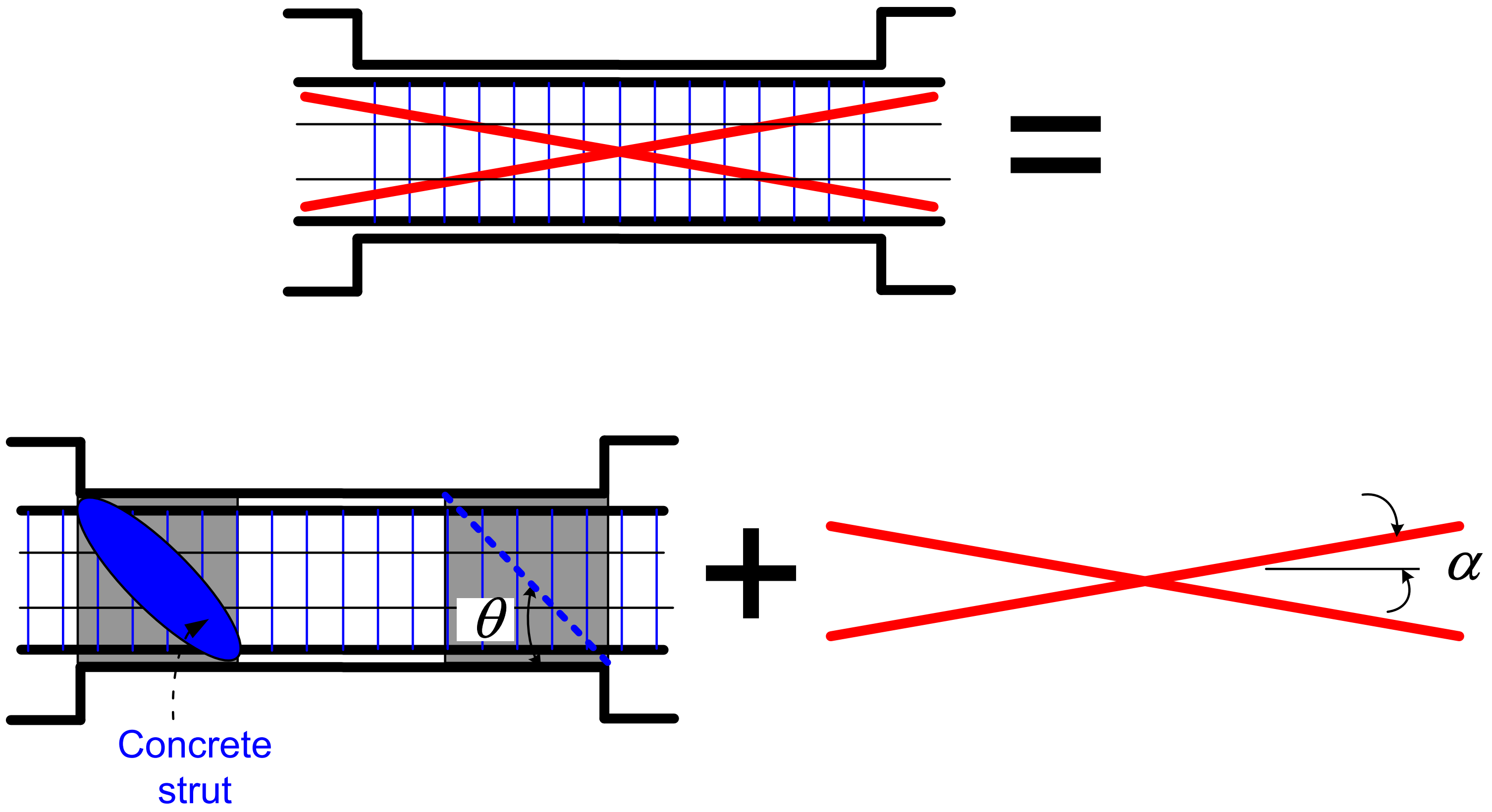


內部抗拉強度要求： $V_s = A_{vt} f_{yt} \dfrac{d}{s} \geq C_d \sin 45^o$



混合型對角鋼筋配置，ϕ=0.85

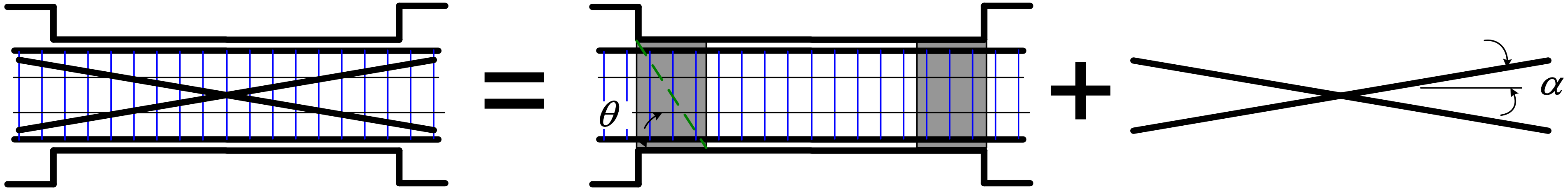
混凝土壓桿與對角鋼筋聯合抗剪 @ MCE



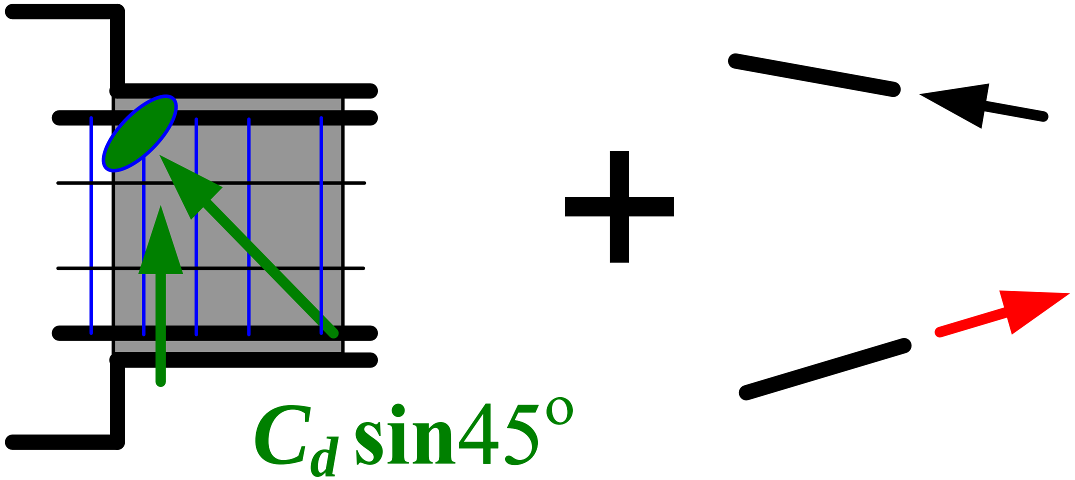
混凝土壓桿 + **對角鋼筋**

(橫向鋼筋維持彈性；θ=45°)　　(拉力鋼筋與壓力鋼筋均應變硬化)

混凝土壓桿與對角鋼筋聯合抗剪 @ MCE



$\theta = 45^{o}$



$C_{dcs} = 1.25 \times f_y \times A_{vd}$

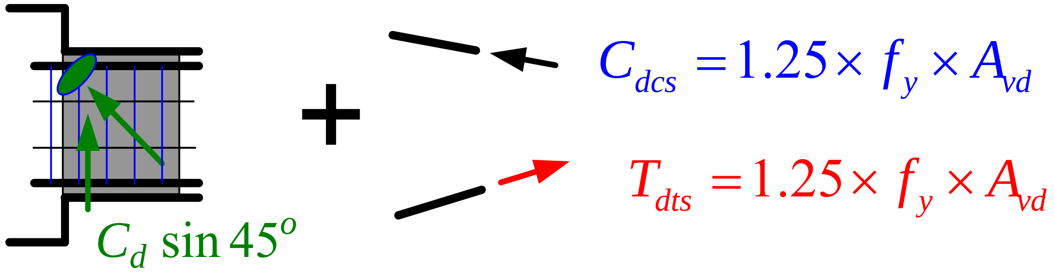
$T_{dts} = 1.25 \times f_y \times A_{vd}$

對角鋼筋應變硬化

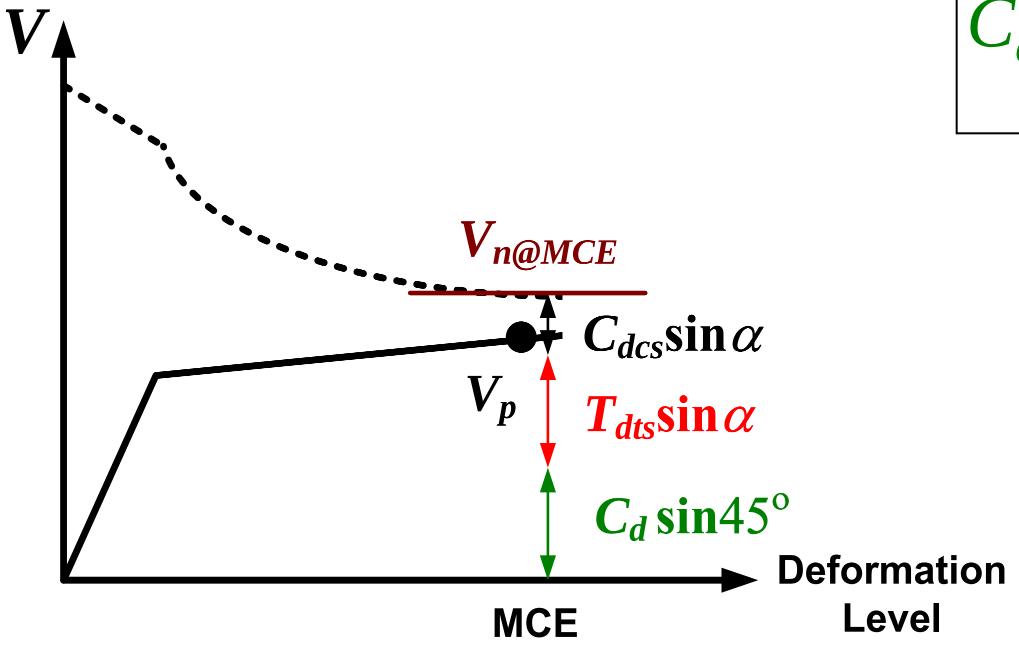
$C_d = K\zeta f_c' A_{str}$

對角鋼筋 A_{vd} 之配置

$$C_d \sin 45^o < \frac{V_p}{\phi}$$



(需配置對角鋼筋)

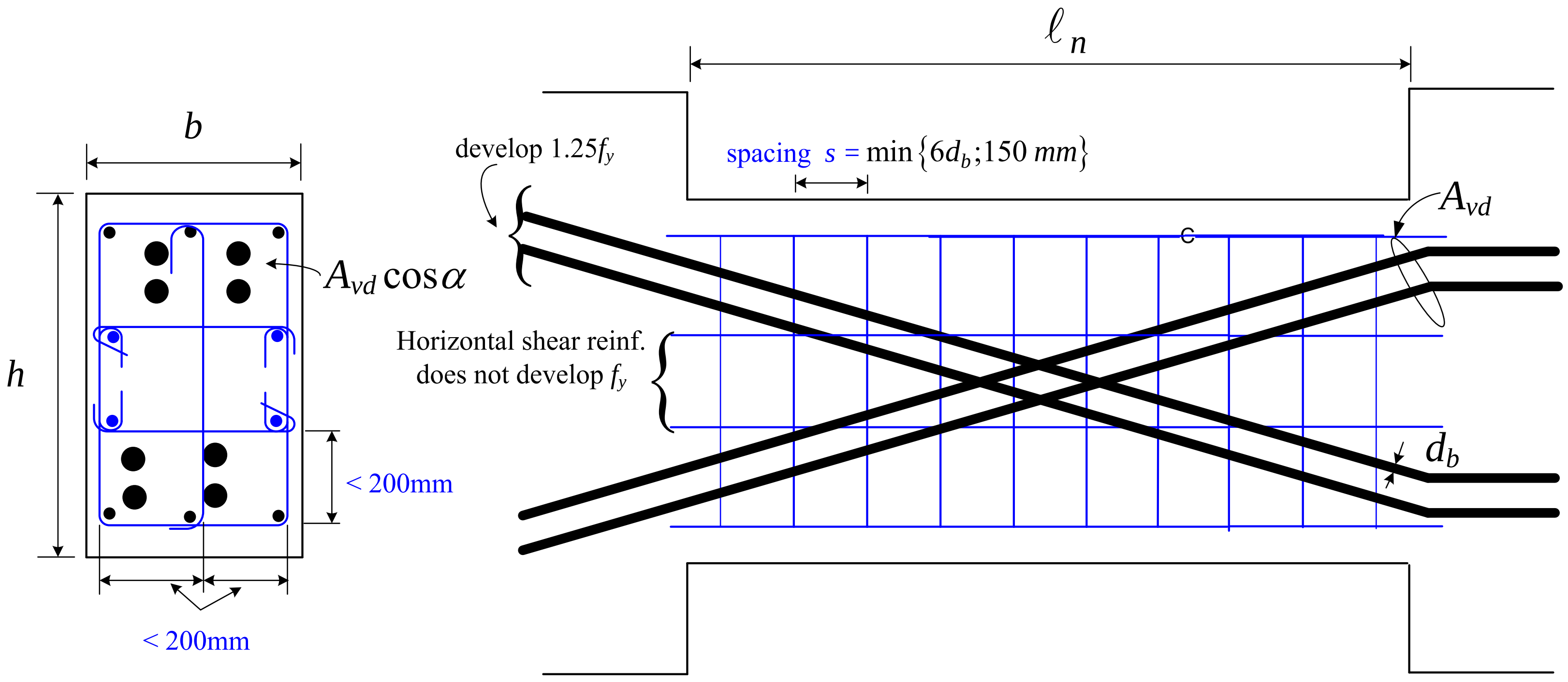


$$C_d \sin 45^o + (1.25 + 1.25) f_y A_{vd} \sin\alpha = \frac{V_p}{\phi}$$

$$\phi = 0.85$$

Solve for: A_{vd}

連接梁之束制鋼筋配置

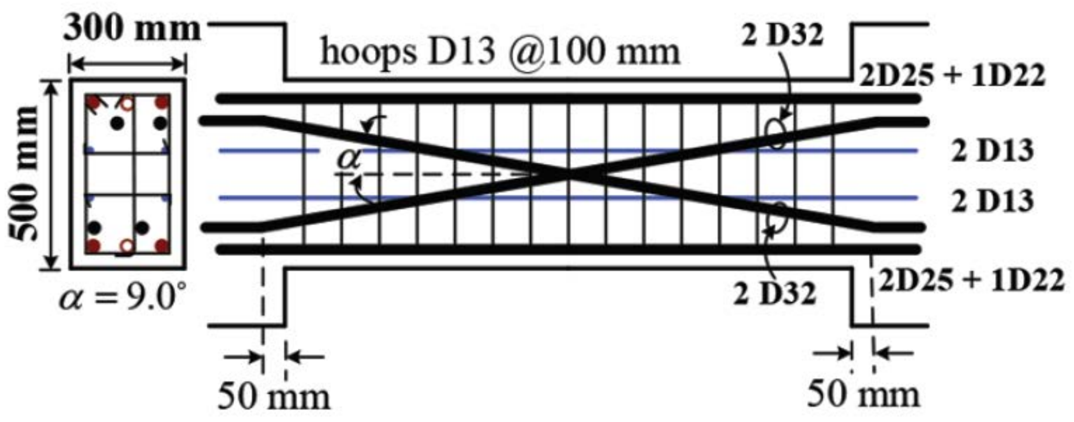


垂直梁寬之橫向鋼筋
暨
平行梁寬之橫向鋼筋

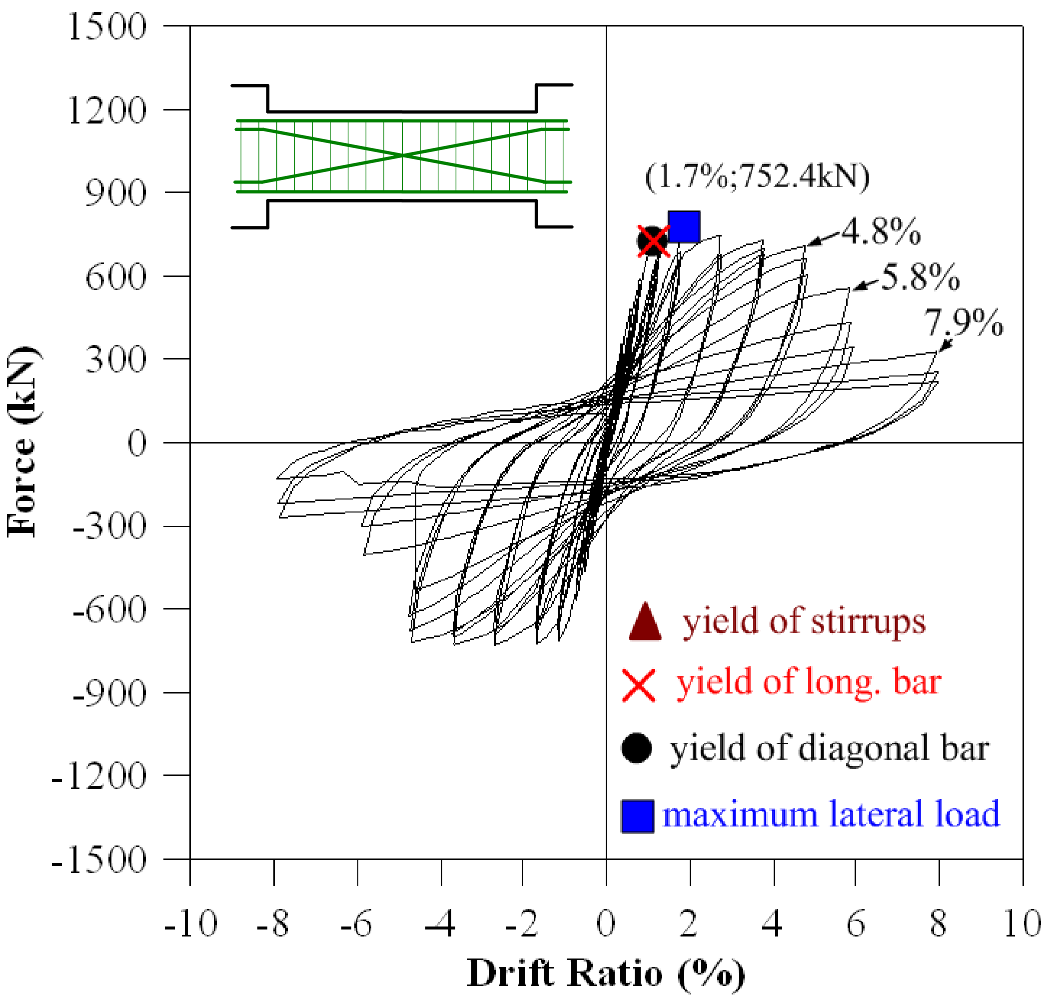
$$A_{sh} = \max\left\{0.3\frac{sb_c f_c'}{f_{yt}}\left(\frac{A_g}{A_{ch}}-1\right);0.09\frac{sb_c f_c'}{f_{yt}}\right\}$$

確保 1：連接梁混凝土之完整性 ； 2：混凝土壓桿強度充分發展

$\ell_n / h = 3$ 混合型(Hybrid)鋼筋連接梁之設計驗證



UDR=5.5%



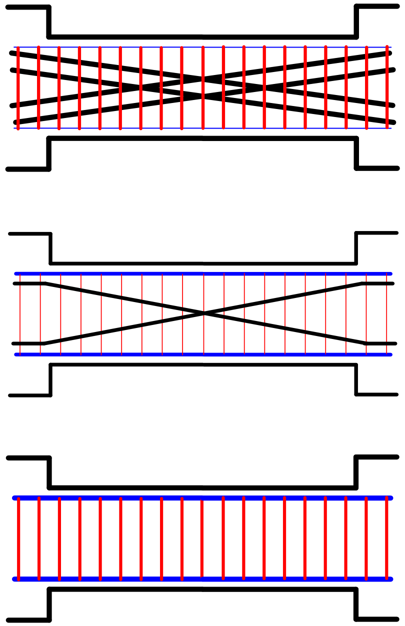
Lim, E., Hwang, S. J., Cheng, C. H., and Lin, P. Y. (2016). "Cyclic Tests of Reinforced Concrete Coupling Beam with Intermediate Span-to-Depth Ratio," ACI Structural Journal, V. 113, No. 3, May-June, pp. 515-524.

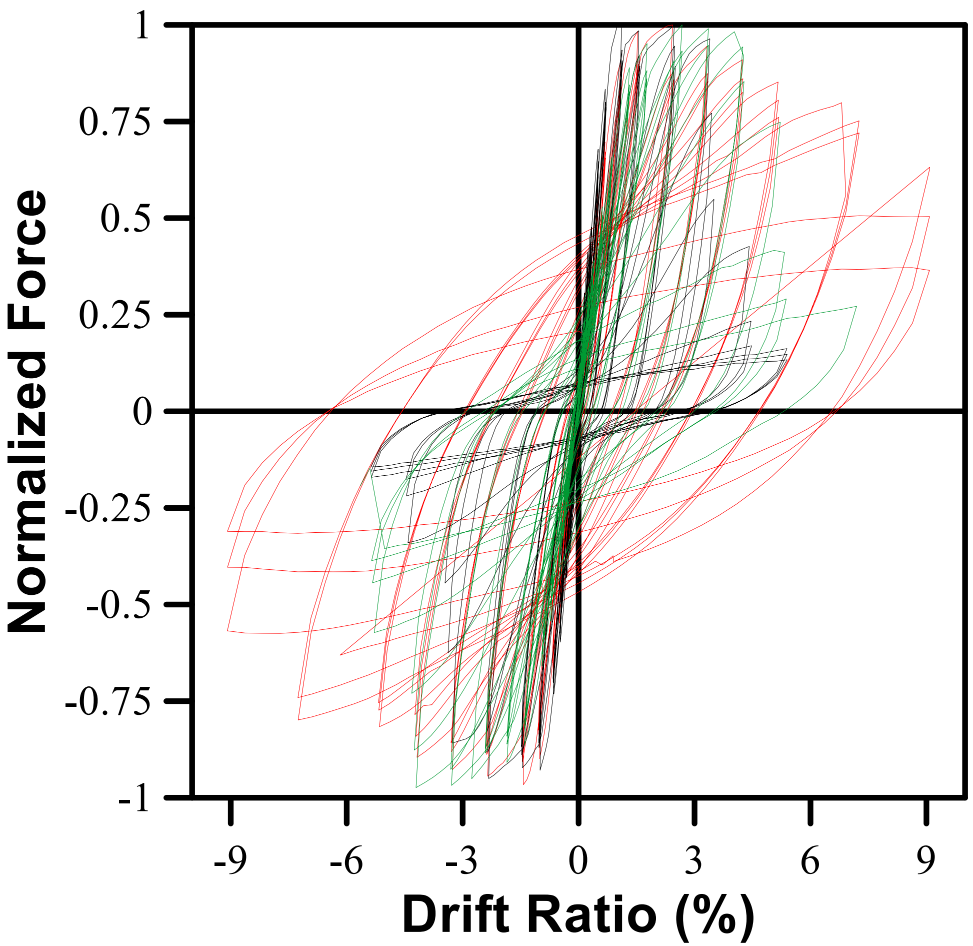
$\ell_n / h = 3$ 混合型(Hybrid)鋼筋連接梁之設計驗證

CB30-DB

CB30-H

CB30-C

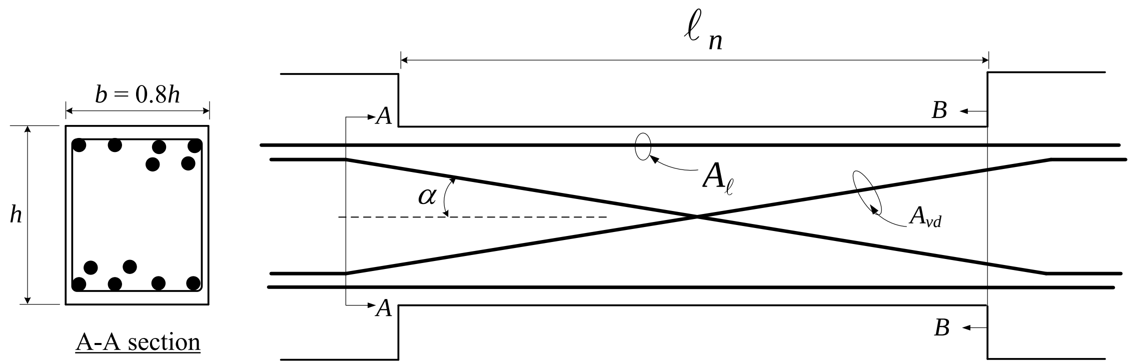




	F.M	Ult. DR	Qual. DR	$\upsilon_{max}\,[MPa]$
CB30-DB	Flexure	7.4%	7.9%	$1.04\sqrt{f_c'}$
CB30-H	Flexure	5.5%	5.5%	$0.92\sqrt{f_c'}$
CB30-C	Flexural Shear	4.1%	2.8%	$0.75\sqrt{f_c'}$

Lim, E., Hwang, S. J., Cheng, C. H., and Lin, P. Y. (2016). “Cyclic Tests of Reinforced Concrete Coupling Beam with Intermediate Span-to-Depth Ratio,” ACI Structural Journal, V. 113, No. 3, May-June, pp. 515-524.

參數研究

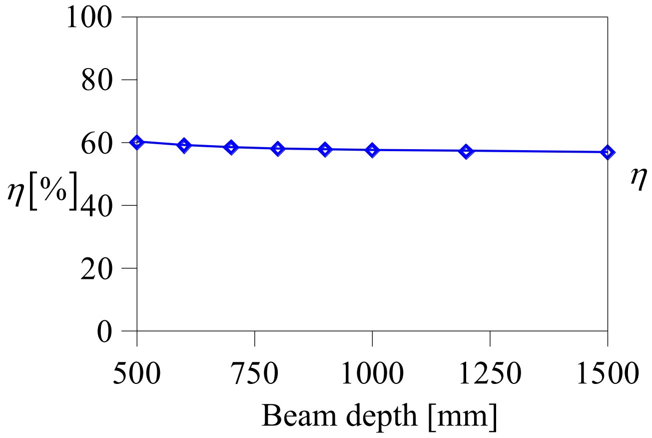


$$f'_c = 28MPa$$

$$f_y = 420MPa$$

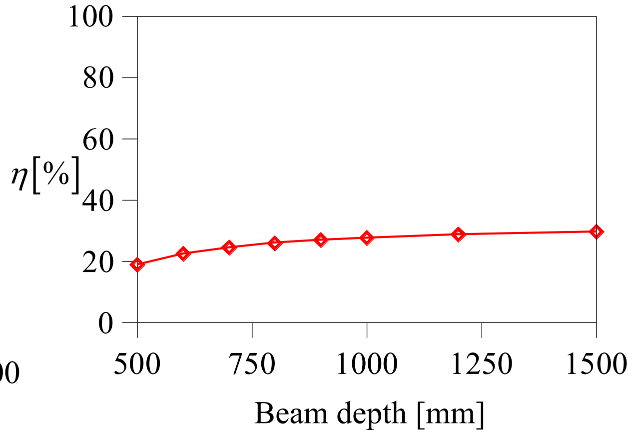
$$\rho_{st} = \frac{A_{st}}{bd} = 2\%$$

$$\frac{\ell_n}{h} = 2$$



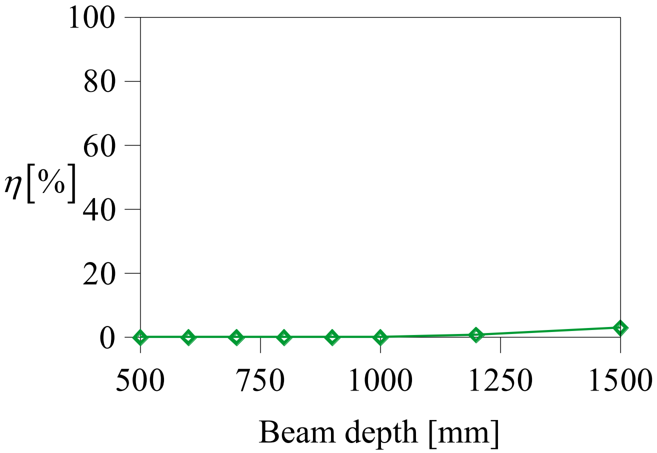
全對角鋼筋

$$\frac{\ell_n}{h} = 3$$



混合型配筋

$$\frac{\ell_n}{h} = 4$$



傳統配筋

測試資料庫之探討

中短連接梁之測試資料庫(節錄版)

No.	Author	Spec. ID	Beam Segment		Diagonal Reinforcement			Longitudinal Reinforcement		Transverse Reinforcement		V_{test} (kN)	UDR (%)
			$\frac{\ell_n}{h}$	f_c' (MPa)	α (°)	f_{yd} (MPa)	Detailing	f_{yl} (MPa)	ρ_ℓ (%)	f_{yt} (MPa)	$\rho_{t,y}$ (%)		
1	Naish *et al.*	CB24F	2.4	47.3	15.7	483	12-#7	483	0.61	483	0.92	761	10.1
2		CB24D	2.4	47.3	15.7	483	12-#7	483	0.25	483	0.29	708	8.6
3		CB33F	3.3	47.3	12.3	483	12-#7	483	0.61	483	0.92	552	8.1
4		CB33D	3.3	47.3	12.3	483	12-#7	483	0.27	483	0.33	528	7.0
5	Weber-Kamin *et al.*	D80-2.5	2.5	57.9	14.2	573	18-#6	613	0.50	613.2	1.00	980	6.8
6		D100-2.5	2.5	55.1	14.2	744	14-#6	613	0.50	613.2	1.00	980	5.2
7*		D120-2.5	2.5	53.7	14.2	799	12-#6	916	0.50	916.4	1.00	1275	6.4
8		D80-3.5	3.5	53.7	10.0	579	18-#7	613	0.50	613.2	1.00	978	8.4
9		D100-3.5	3.5	54.4	10.3	744	18-#6	613	0.50	613.2	1.00	865	6.7
10		D120-3.5	3.5	56.5	10.3	799	16-#6	613	0.50	613.2	1.00	962	5.8
11*	Lim *et al.*	CB30-DA	3.0	39.7	8.8	441	8-#10	441	0.34	441	0.42	671	6.9
12*		CB30-DB	3.0	38.4	8.8	475	8-#10	475	0.48	475	0.71	728	7.5
13*		CB30-2.5-54-N	3.0	58.0	9.0	455	4-#10	457	2.21	486	1.27	752	5.7

註：*縱向錨定鋼筋端部具有充分錨定之試體

Tsai, R. J., Henry, R, S., Elwood, K. J., Hwang, S. J., and Cheng, M. Y. (2026). "Dataset of diagonally reinforced coupling beams: Axial restraint and strength design," Engineering Structures 349, 121759.

初始文獻

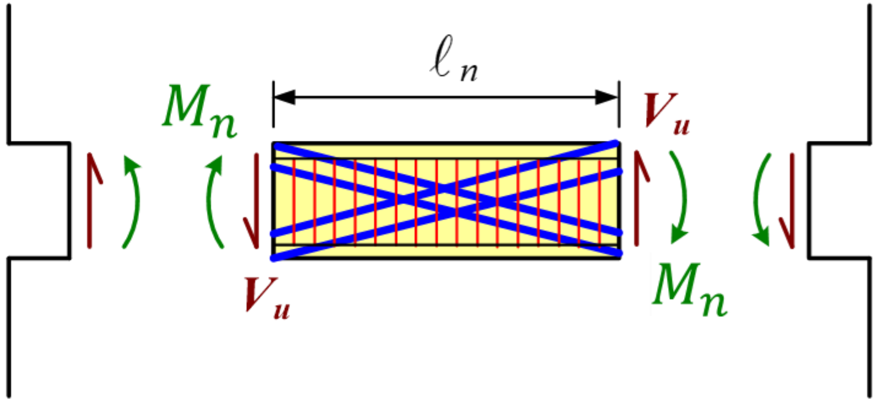
- Naish D, Fry A, Klemencic R, Wallace. J. Reinforced concrete coupling beams-Part I: Testing. ACI Struct J 2013;110(6):1057–66.
- Weber-Kamin A, Lepage A, Lequesne R. Reinforced concrete coupling beams with high-strength steel bars. PRJ-2876, DesignSafe-CI 2020. https://doi.org/ 10.17603/ds2-k0rq-s232.
- Lim E, Hwang S-J, Cheng C-H, Lin P-Y. Cyclic tests of reinforced concrete coupling beam with intermediate span-depth ratio. ACI Struct J 2016;113(3):515–24. https://doi.org/10.14359/51688473.

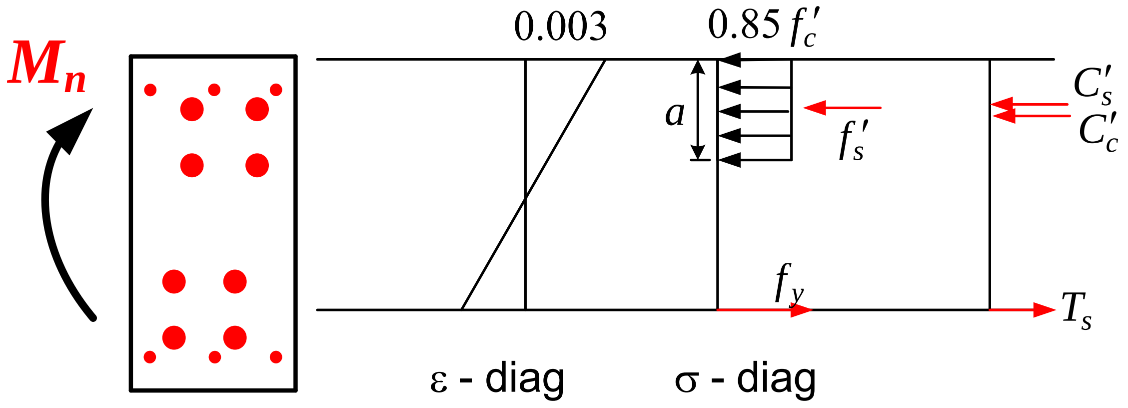
測試試體之耐震行為評估

撓曲強度之計算流程

撓曲強度:

由 XTRACT 取得斷面標稱彎矩 $M_n \Rightarrow V_{mn} = \dfrac{2M_n}{\ell_n}$

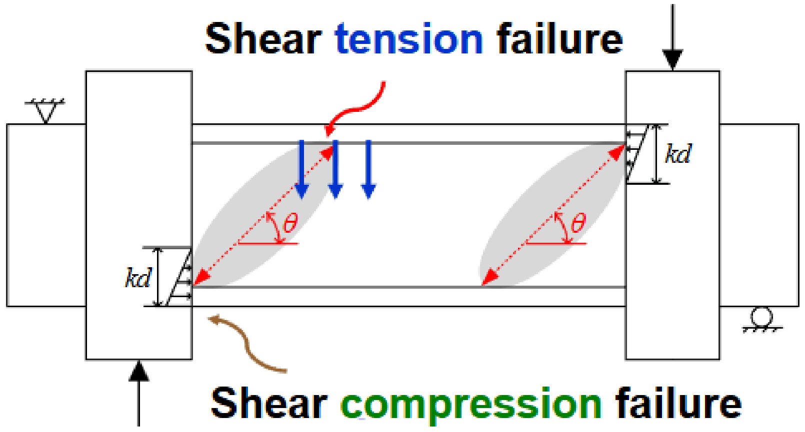




- 斷面分析 - XTRACT
- 平面維持平面

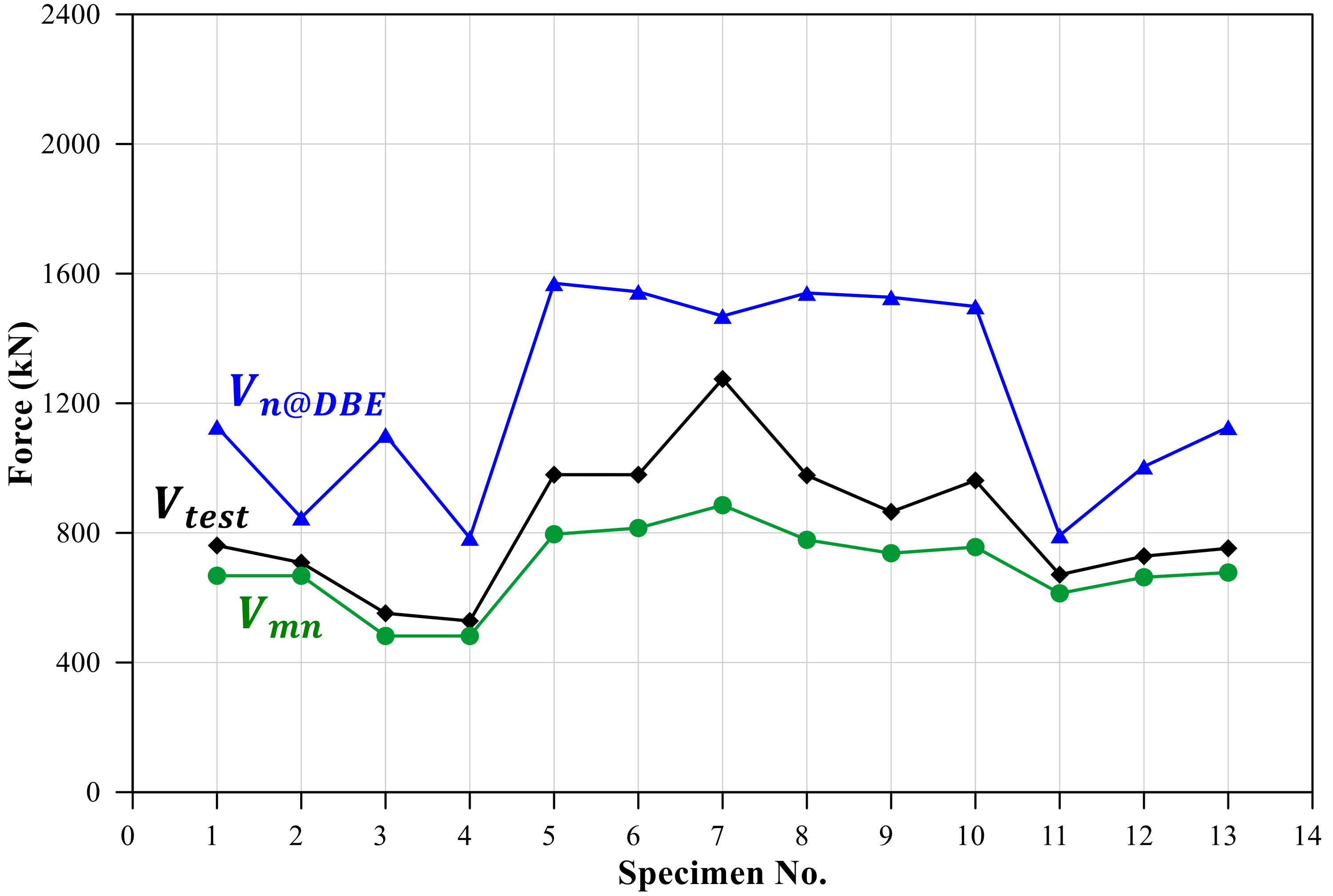
DBE剪力強度之計算流程

Strength (DBE):



1. 彈性壓力區深度 $kd = \left(\sqrt{(\rho_f n)^2 + 2\rho_f n} - \rho_f n\right) d$
2. 壓拉桿指標 $K = \tan^A \theta + \cot^A \theta - 1 + 0.14B \le 1.64$
3. 開裂鋼筋混凝土軟化係數 $\zeta = \frac{3.35}{\sqrt{f_c'}} \le 0.52$
4. 計算剪壓強度 $V_{n,c} = K\zeta f_c' b_w kd \sin\theta$
5. 計算剪拉強度 $V_{n,t} = V_c + V_s = 0.17\sqrt{f_c'(MPa)} b_w d + \frac{A_{vt} f_{yt} d}{s}$; $V_c \ne 0$
6. 計算DBE需求下之剪力強度 $V_{n@DBE} = \min(V_{n,c}, V_{n,t}) + (1.0 + 1.0) A_{vd} f_{yd} \sin\alpha$

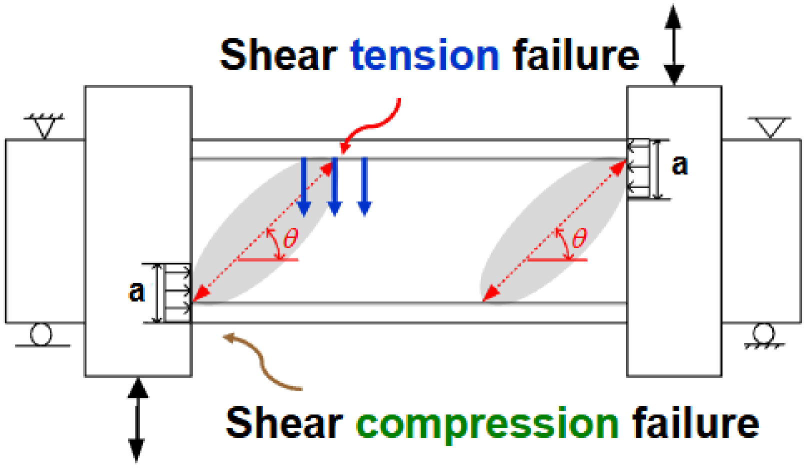
DBE強度行為評估



1. $V_{mn} < V_{n@DBE}$
2. 撓曲強度主控行為
3. $V_{test} > V_{mn}$
4. 撓曲塑鉸變形可以發展

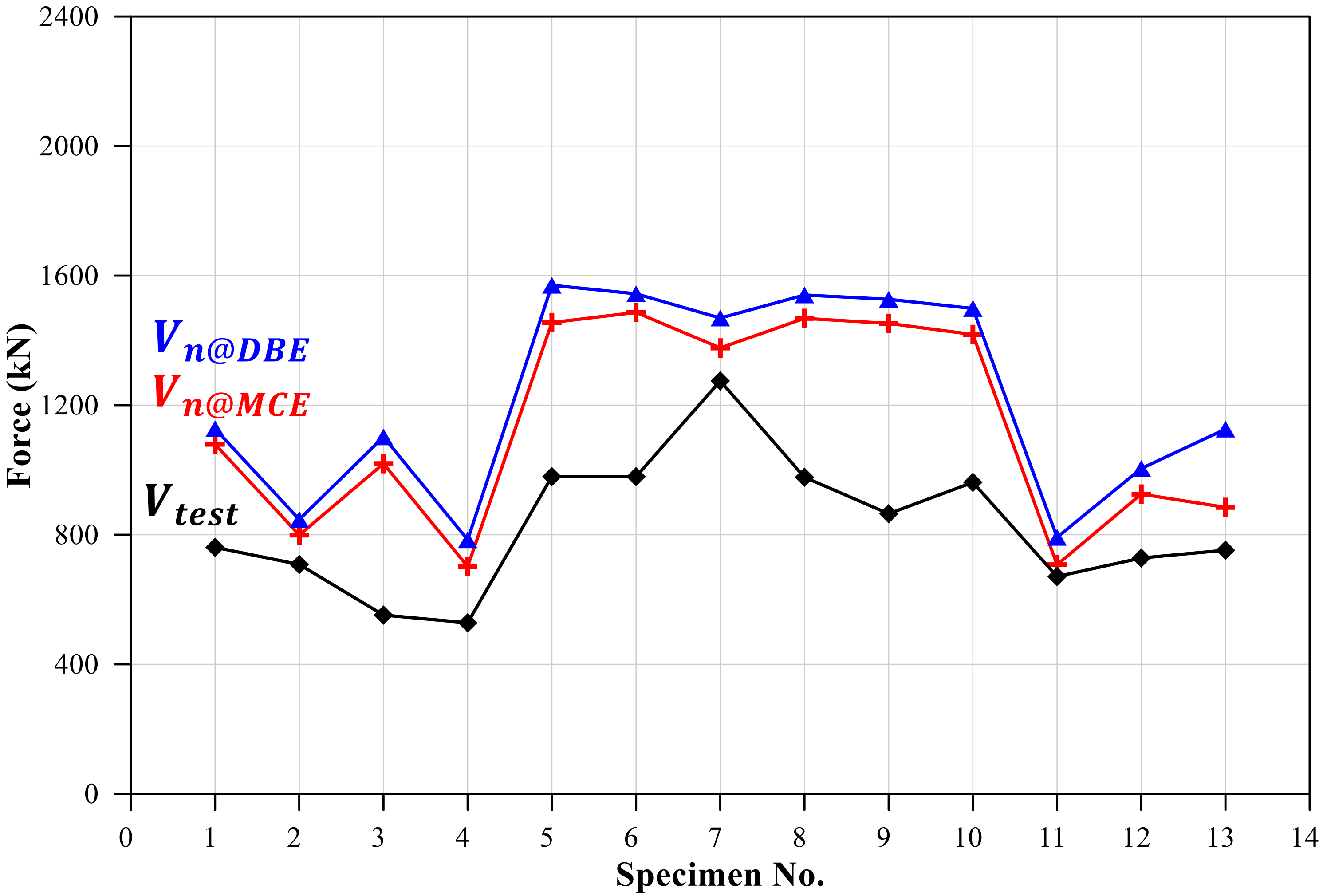
MCE剪力強度之計算流程

Strength (MCE):



1. 壓力區深度 $a = \frac{1.25 f_y \times A_{st}}{0.85 f_c' \times b_w}$
2. 壓拉桿指標 $K = \tan^A\theta + \cot^A\theta - 1 + 0.14B \le 1.64$
3. 開裂鋼筋混凝土軟化係數 $\zeta = \frac{3.35}{\sqrt{f_c'}} \le 0.52$
4. 計算剪壓強度 $V_{n,c} = K\zeta f_c' b_w a \sin\theta$
5. 計算剪拉強度 $V_{n,t} = V_s = \frac{A_{vt} f_{yt} d}{s}$ ；$V_c = 0$
6. 計算MCE需求下之剪力強度 $V_{n@MCE} = \min(V_{n,c}, V_{n,t}) + (1.25 + 1.25)A_{vd} f_{yd} \sin\alpha$

MCE強度行為評估



1. $V_{n@MCE} > V_{test}$
2. 撓曲塑鉸變形充分發展
3. $V_{n@MCE} < V_{n@DBE}$
4. 剪力強度有劣化現象

測試資料庫行為數據

No.	Author	Spec. ID	V_{test} (kN)	UDR (%)	$\frac{V_{test}}{V_{mn}}$	$\frac{V_{test}}{V_{n@DBE}}$	$\frac{V_{test}}{V_{n@MCE}}$
1	Naish *et al.*	CB24F	761	10.1	1.14	0.68	0.71
2		CB24D	708	8.6	1.06	0.84	0.89
3		CB33F	552	8.1	1.15	0.50	0.54
4		CB33D	528	7.0	1.10	0.67	0.75
5	Weber-Kamin *et al.*	D80-2.5	980	6.8	1.23	0.62	0.67
6		D100-2.5	980	5.2	1.20	0.63	0.66
7*		D120-2.5	1275	6.4	1.44	0.87	0.93
8		D80-3.5	978	8.4	1.26	0.63	0.67
9		D100-3.5	865	6.7	1.17	0.57	0.60
10		D120-3.5	962	5.8	1.27	0.64	0.68
11*	Lim *et al.*	CB30-DA	671	6.9	1.09	0.85	0.95
12*		CB30-DB	728	7.5	1.10	0.72	0.79
13*		CB30-2.5-54-N	752	5.7	1.11	0.67	0.85
				AVG	**1.21**	**0.68**	**0.74**
				COV	**0.09**	**0.15**	**0.16**

$$\frac{V_{test}}{V_{mn}} > 1$$

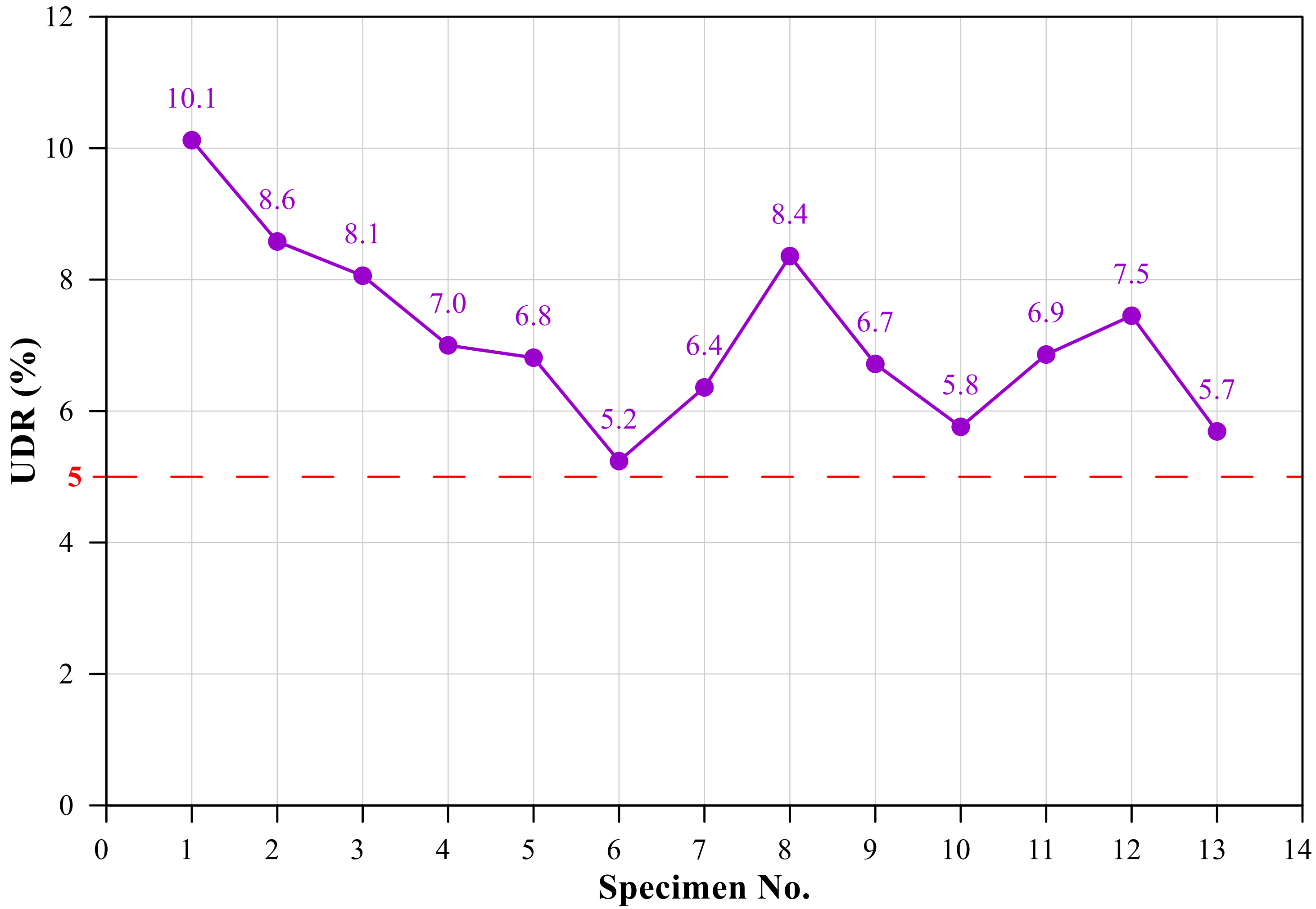
撓曲行為主控

$$\frac{V_{test}}{V_{n@MCE}} < 1$$

剪力強度充分支持撓曲塑鉸發展

註：*縱向錨定鋼筋端部具有充分錨定之試體

撓曲塑鉸變形能力評估

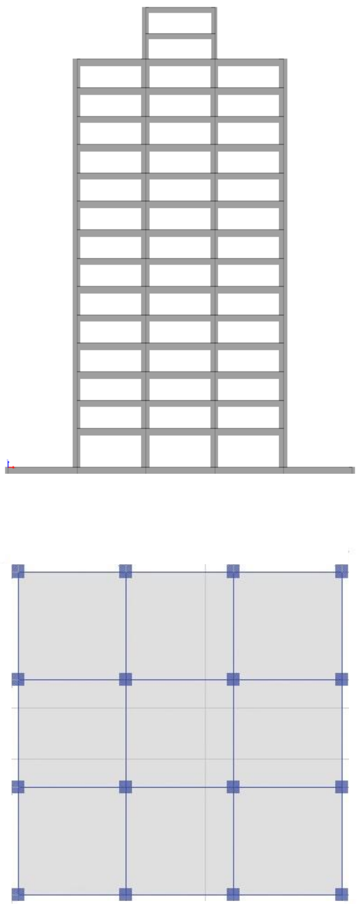


1. 極限變位角(UDR)>5%
2. 撓曲塑鉸變形充分發展

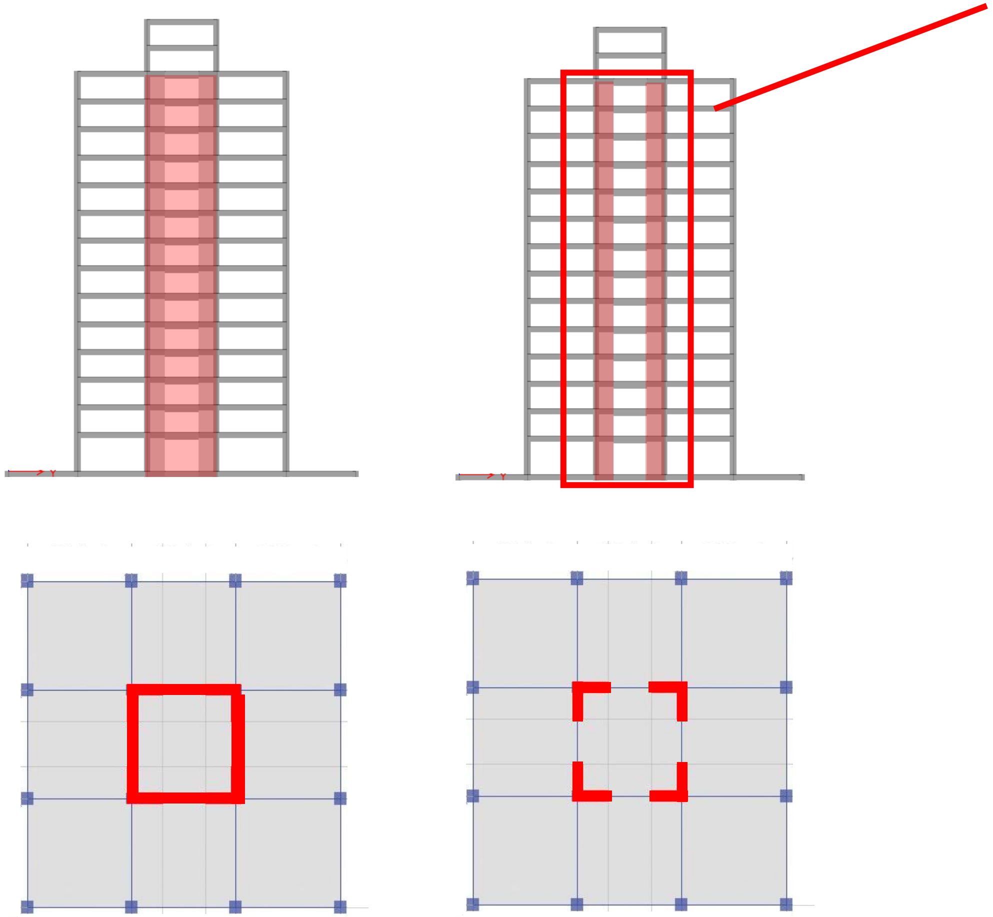
設計案例

結構系統探討

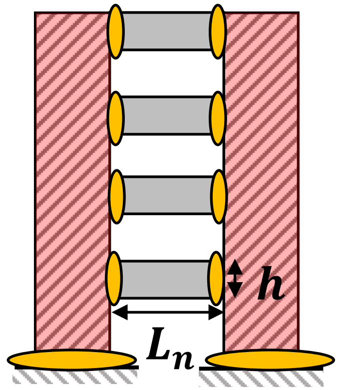
構架系統



二元系統



Span to depth ratio = L_n/h

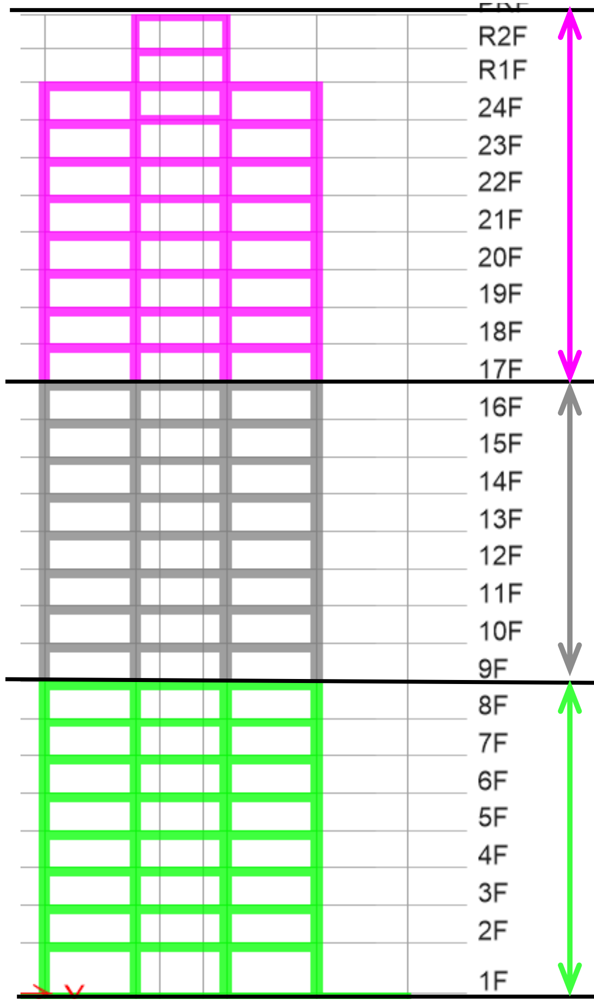


Study Cases:

1. Without opening
2. $L_n/h = 1.5$
3. $L_n/h = 2.4$
4. $L_n/h = 3.3$
5. $L_n/h = 4.2$

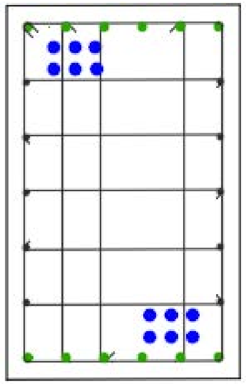
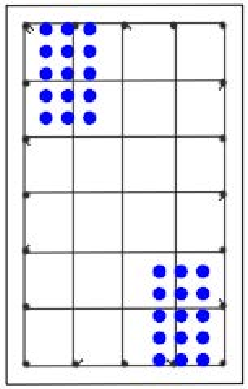
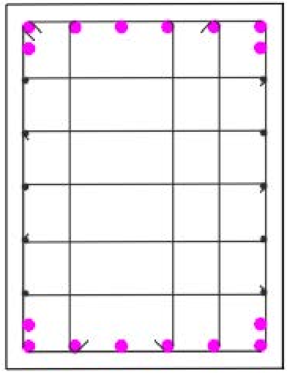
沈家瑋，(2024)「鋼筋混凝土耦合牆系統之設計與應用」，碩士論文，國立台灣大學土木工程學系，臺北，臺灣。

材料強度

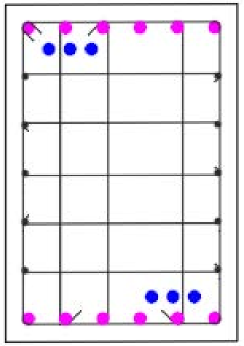
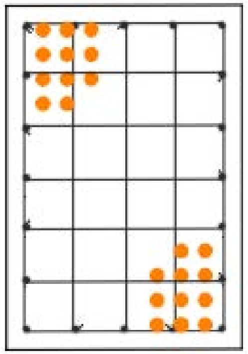
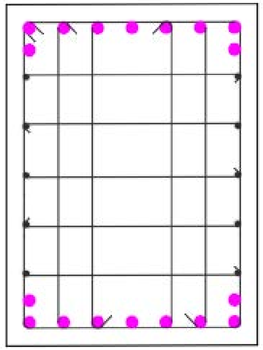


24F structure	Story	f_c'	f_y
	17F～PRFL	350 kgf/cm^2	4200 kgf/cm^2
	9F~17FL	420 kgf/cm^2	5000 kgf/cm^2
	1F~9FL	490 kgf/cm^2	5000 kgf/cm^2

24F連接梁設計(L_n/h=2.4)

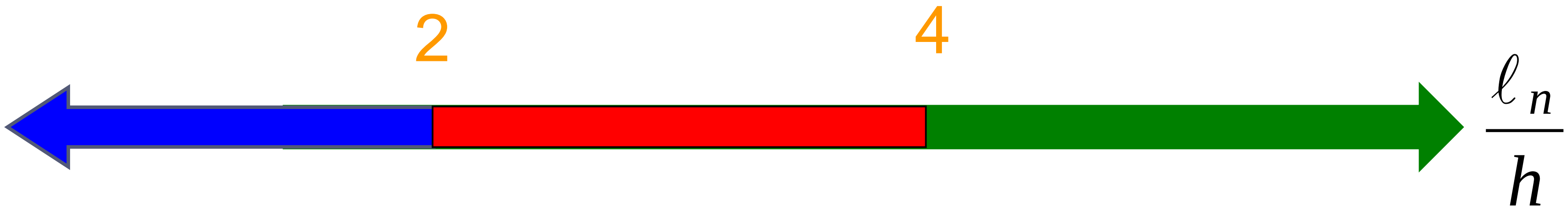
		SST	ACI	
			Full Diagonal	Conventional
Section				
Steel	Diagonal	**6-#10**	**15-#10**	–
	Longitudinal	**6-#7**	–	**8-#10**
備註		–	對角鋼筋α角度小，sinα分量小，以至於鋼筋需求量高	受限於梁最大作用剪力規定，必須擴大混凝土斷面

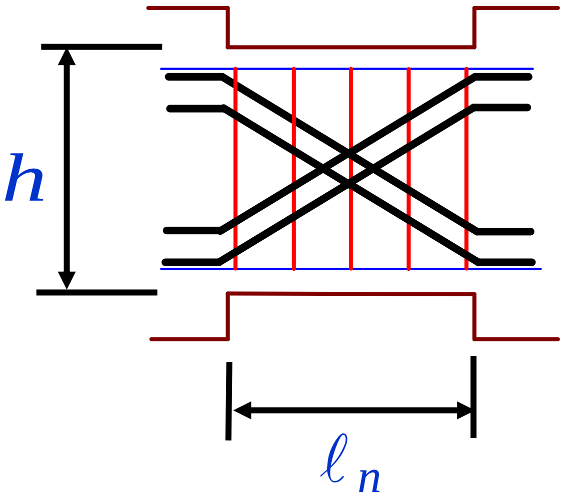
24F連接梁設計($L_n/h=3.3$)

		SST	ACI	
			Full Diagonal	Conventional
Section		70 cm  100 cm	70 cm  100 cm	75 cm  100 cm
Steel	Diagonal	3-#10	11-#12	–
	Longitudinal	6-#10	–	9-#10
備註		詳SST設計案例	對角鋼筋α角度小，sinα分量小，以至於鋼筋需求量高	受限於梁最大作用剪力規定，必須擴大混凝土斷面

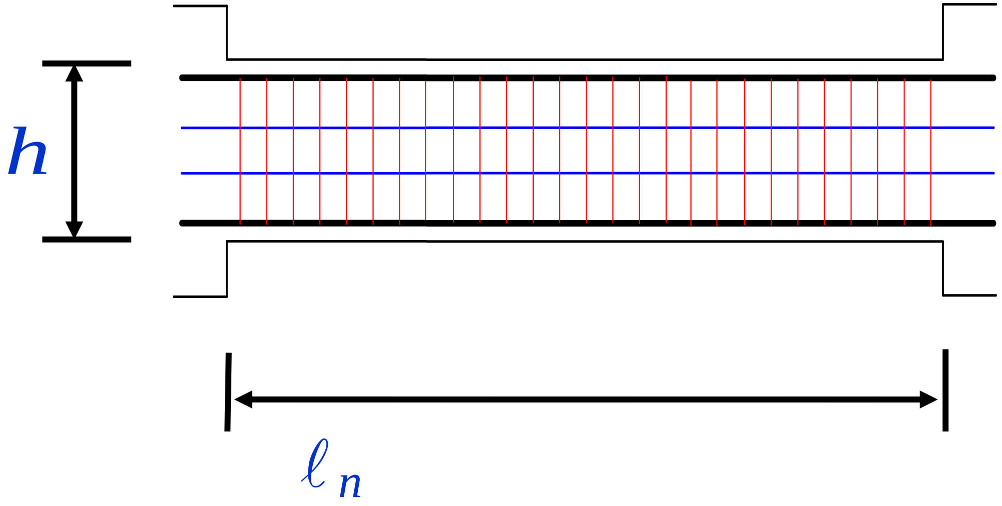
結論

連接梁之設計建議

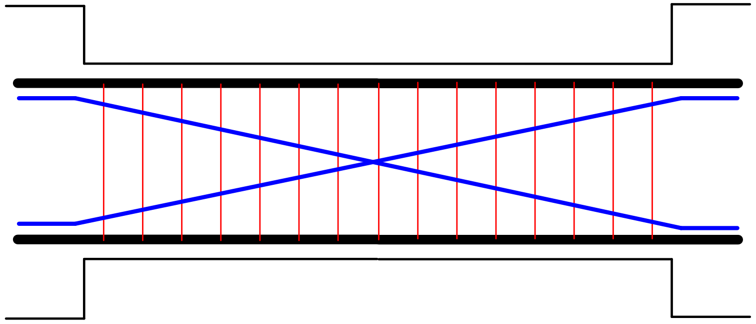




得配置:
(1)對角鋼筋
(2)傳統配筋
(3)混合型配筋

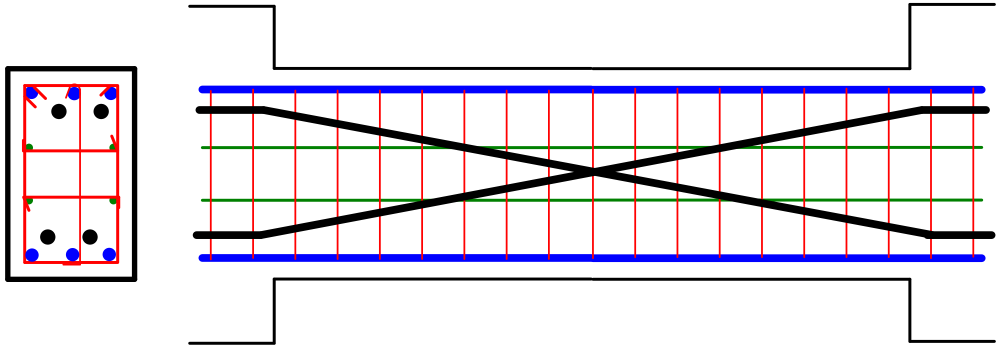


應配置對跨度中點對稱之兩組對角向鋼筋



應作傳統鋼筋設計

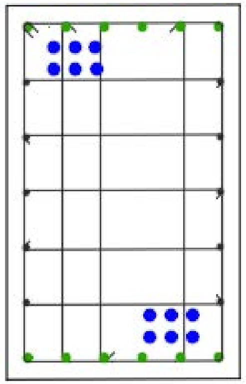
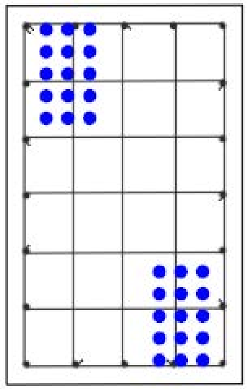
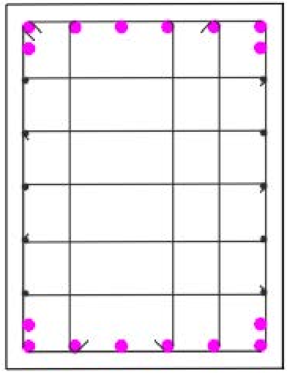
$2 \le \ell_n / h \le 4$ 混合型(Hybrid)鋼筋連接梁之設計



- A_ℓ 撓曲鋼筋
- A_{vd} 對角鋼筋

設計地震 Design Based Earthquake **DBE**	$M_n \ge M_u / \phi$ $A_{st} \times f_y \times jd \ge M_u / 0.9$	計算撓曲鋼筋總量 A_{st}
最大可能地震 Max. Credible Earthquake **MCE**	$C_d \sin 45° + 2.5 A_{vd} f_y \sin\alpha \ge V_p / \phi$ $A_\ell = A_{st} - A_{vd} \cos\alpha$	剪力容量設計 計算對角鋼筋 A_{vd} 計算撓曲鋼筋A_ℓ

24F連接梁設計($L_n/h=2.4$)

		SST	ACI	
			Full Diagonal	Conventional
Section		70 *cm*  110 *cm*	70 *cm*  110 *cm*	85 *cm*  110 *cm*
Steel	Diagonal	**6-#10**	**15-#10**	–
	Longitudinal	**6-#7**	–	**8-#10**
備註		–	對角鋼筋α角度小，sinα分量小，以至於鋼筋需求量高	受限於梁最大作用剪力規定，必須擴大混凝土斷面

敬請指教！

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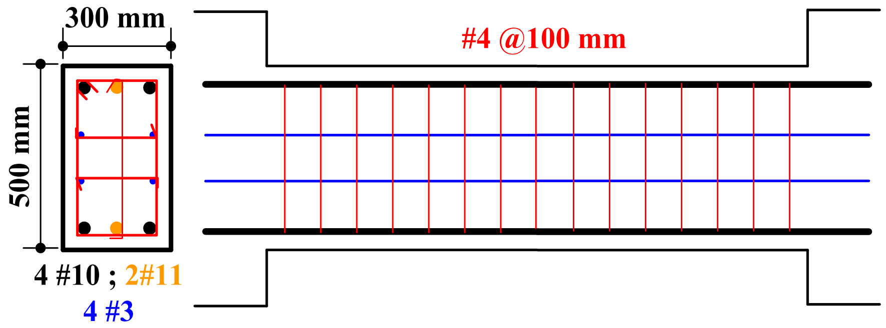
附錄：連接梁強度評估計算

實驗試體

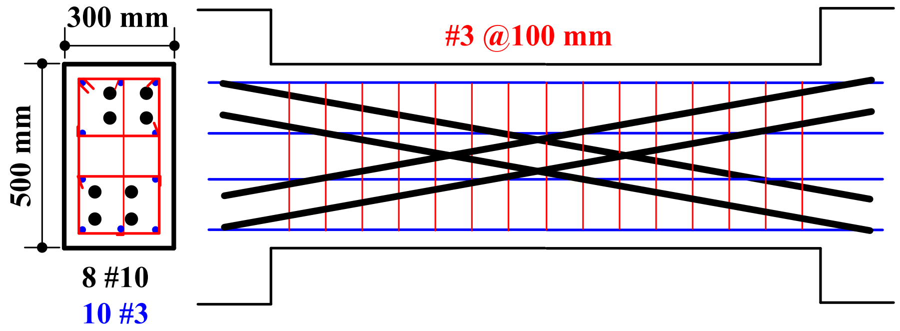
$$\ell_n / h = 3.0$$

CB30-C



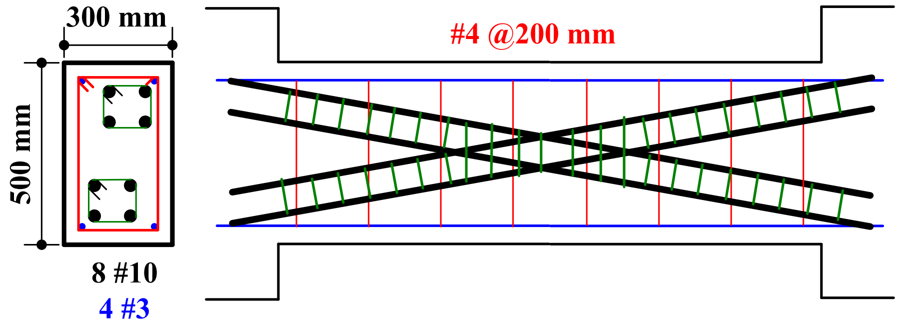
$f_c' = 47.9\ MPa$
$\rho_f = 2.03\%$

CB30-DB



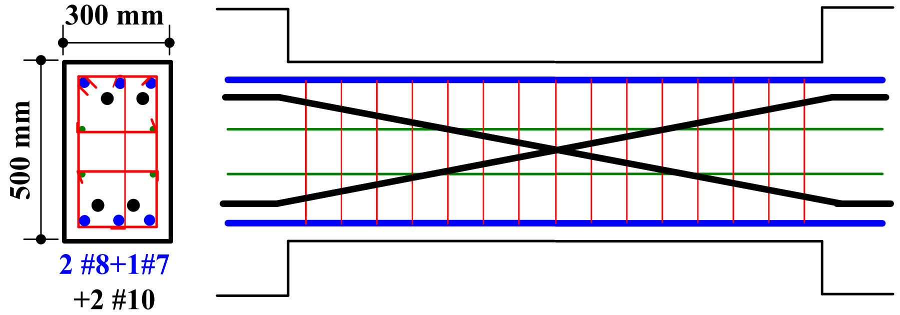
$f_c' = 38.4\ MPa$
$\rho_f = 2.86\%$

CB30-DA



$f_c' = 39.7\ MPa$
$\rho_f = 2.89\%$

CB30-H

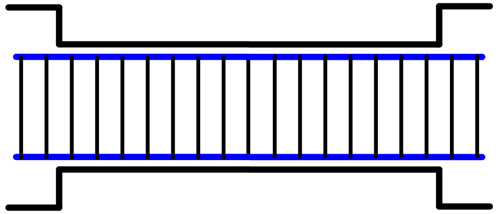


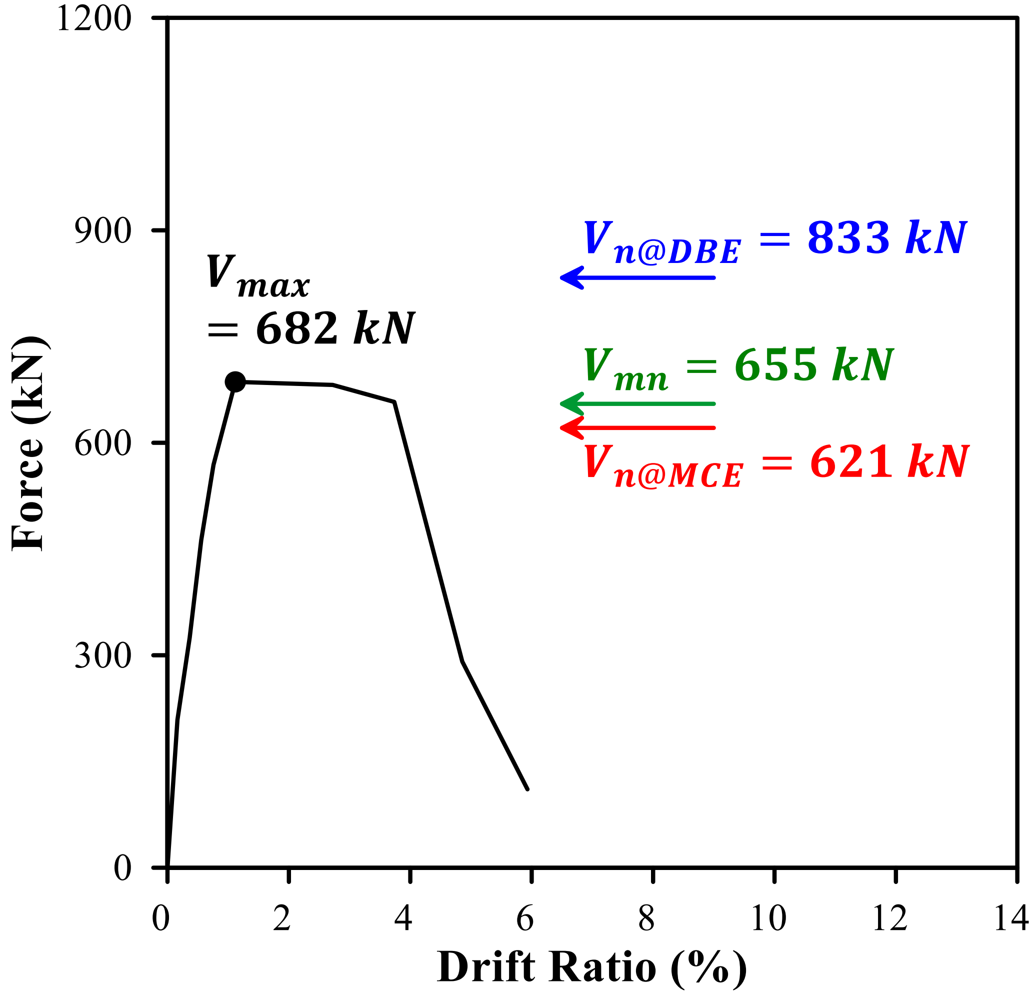
$f_c' = 58.0\ MPa$
$\rho_f = 2.51\%$

撓曲與剪力強度評估

$\ell_n / h = 3.0$

CB30-C





[Flexure Strength]

$V_{mn} = 2M_n/\ell_n = 655\ kN$

[Strength - DBE]

$$kd = \left(\sqrt{(\rho_f n)^2 + 2\rho_f n} - \rho_f n\right) d = 149\ mm$$

$$A_{str} = b_w \times kd = 44575\ mm^2$$

$$\theta = \theta_{max} = 45.0°$$

$$K = \tan^A\theta + \cot^A\theta - 1 + 0.14B \le 1.64 \xrightarrow{take} 1.14$$

$$V_{n,c} = C_d \sin\theta = K A_{str} \zeta f_c' \sin\theta = 833\ kN$$

$$V_{n,t} = V_c + V_s = 0.17\sqrt{f_c'} b_w d + \frac{A_v f_{yt} d}{s} = 878\ kN$$

$$V_{n@DBE} = \min(V_{n,c}, V_{n,t}) + 0 = 833\ kN$$

[Strength - MCE]

$$a = \frac{1.1 f_{y,actual} \times A_{st}}{0.85 f_c' \times b_w} = 111\ mm$$

$$A_{str} = b_w \times a = 33247\ mm^2$$

$$\theta = \theta_{max} = 45.0°$$

$$K = \tan^A\theta + \cot^A\theta - 1 + 0.14B \le 1.64 \xrightarrow{take} 1.14$$

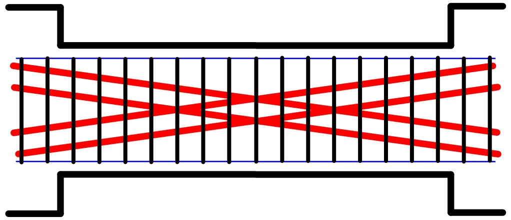
$$V_{n,c} = C_d \sin\theta = K A_{str} \zeta f_c' \sin\theta = 621\ kN$$

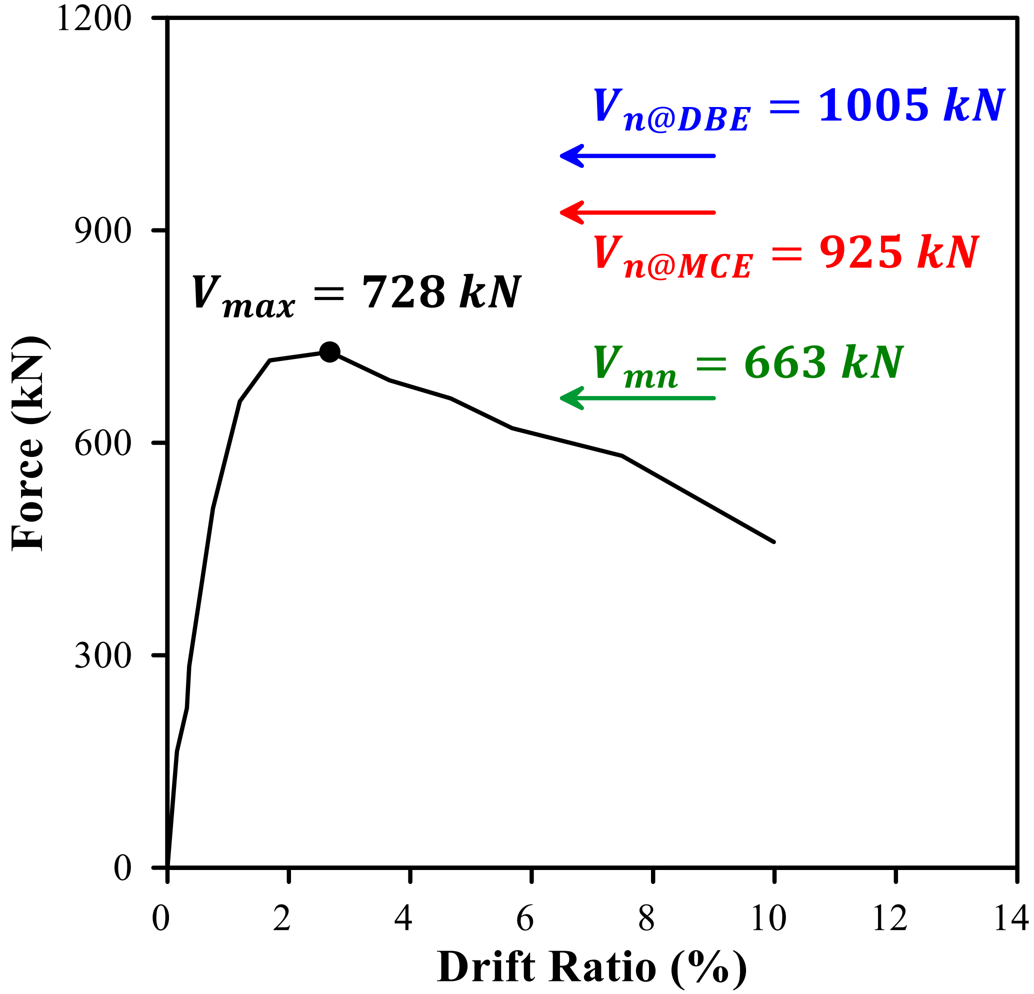
$$V_{n,t} = V_s = \frac{A_v f_{yt} d}{s} = 725\ kN$$

$$V_{n@MCE} = \min(V_{n,c}, V_{n,t}) + 0 = 621\ kN$$

$\ell_n / h = 3.0$

CB30-DB





撓曲與剪力強度評估

[Flexure Strength]

$V_{mn} = 2M_n/\ell_n = 663\ kN$

[Strength - DBE]

$kd = \left(\sqrt{(\rho_f n)^2 + 2\rho_f n} - \rho_f n\right) d = 199\ mm$

$A_{str} = b_w \times kd = 59695\ mm^2$

$\theta = \theta - \Delta\theta \le 26.5° \xrightarrow{take} 26.5°$

$K = \tan^A \theta + \cot^A \theta - 1 + 0.14B \le 1.64 \xrightarrow{take} 1.64$

$V_{n,c} = C_d \sin\theta = K A_{str} \zeta f_c' \sin\theta = 872\ kN$

$V_{n,t} = V_c + V_s = 0.17\sqrt{f_c'} b_w d + \frac{A_v f_{yt} d}{s} = 531\ kN$

$(1.0 + 1.0) A_{vd} f_{yd} \sin\alpha = 473 kN$

$V_{n@DBE} = \min(V_{n,c}, V_{n,t}) + (1.0 + 1.0) A_{vd} f_{yd} \sin\alpha = 1005\ kN$

[Strength - MCE]

$a = \frac{1.1 f_{y,actual} \times A_{st}}{0.85 f_c' \times b_w} = 172\ mm$

$A_{str} = b_w \times a = 51527\ mm^2$

$\theta = \theta - \Delta\theta \le 26.5° \xrightarrow{take} 26.5°$

$K = \tan^A \theta + \cot^A \theta - 1 + 0.14B \le 1.64 \xrightarrow{take} 1.64$

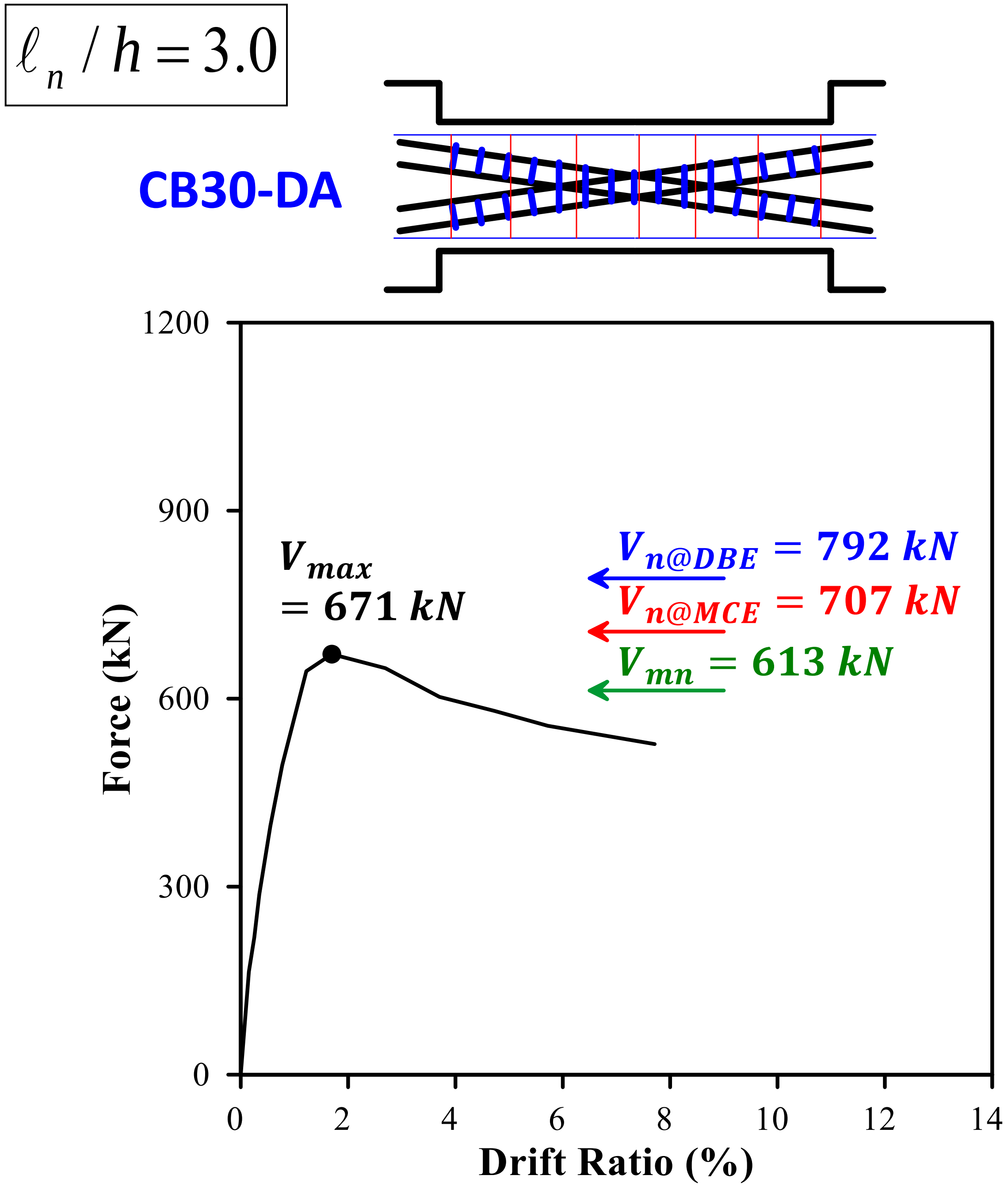
$V_{n,c} = C_d \sin\theta = K A_{str} \zeta f_c' \sin\theta = 753\ kN$

$V_{n,t} = V_s = \frac{A_v f_{yt} d}{s} = 405\ kN$

$(1.1 + 1.1) A_{vd} f_{yd,actual} \sin\alpha = 520\ kN$

$V_{n@MCE} = \min(V_{n,c}, V_{n,t}) + (1.1 + 1.1) A_{vd} f_{yd,actual} \sin\alpha = 925\ kN$

撓曲與剪力強度評估



[Flexure Strength]

$V_{mn} = 2M_n/\ell_n = 613\ kN$

[Strength - DBE]

$kd = \left(\sqrt{(\rho_f n)^2 + 2\rho_f n} - \rho_f n\right) d = 199\ mm$

$A_{str} = b_w \times kd = 59590\ mm^2$

$\theta = \theta - \Delta\theta \leq 26.5° \xrightarrow{take} 26.5°$

$K = \tan^A \theta + \cot^A \theta - 1 + 0.14B \leq 1.64 \xrightarrow{take} 1.30$

$V_{n,c} = C_d \sin\theta = K A_{str} \zeta f_c' \sin\theta = 712\ kN$

$V_{n,t} = V_c + V_s = 0.17\sqrt{f_c'} b_w d + \frac{A_v f_{yt} d}{s} = 353\ kN$

$(1.0 + 1.0) A_{vd} f_{yd} \sin\alpha = 440\ kN$

$V_{n@DBE} = \min(V_{n,c}, V_{n,t}) + (1.0 + 1.0) A_{vd} f_{yd} \sin\alpha = 792\ kN$

[Strength - MCE]

$a = \frac{1.1 f_{y,actual} \times A_{st}}{0.85 f_c' \times b_w} = 154\ mm$

$A_{str} = b_w \times a = 46273\ mm^2$

$\theta = \theta - \Delta\theta \leq 26.5° \xrightarrow{take} 26.5°$

$K = \tan^A \theta + \cot^A \theta - 1 + 0.14B \leq 1.64 \xrightarrow{take} 1.30$

$V_{n,c} = C_d \sin\theta = K A_{str} \zeta f_c' \sin\theta = 552\ kN$

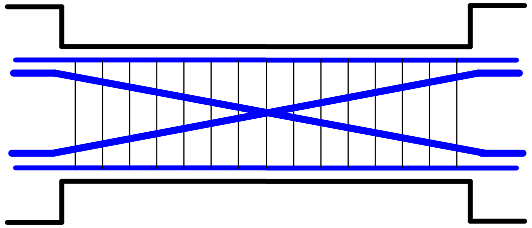
$V_{n,t} = V_s = \frac{A_v f_{yt} d}{s} = 224\ kN$

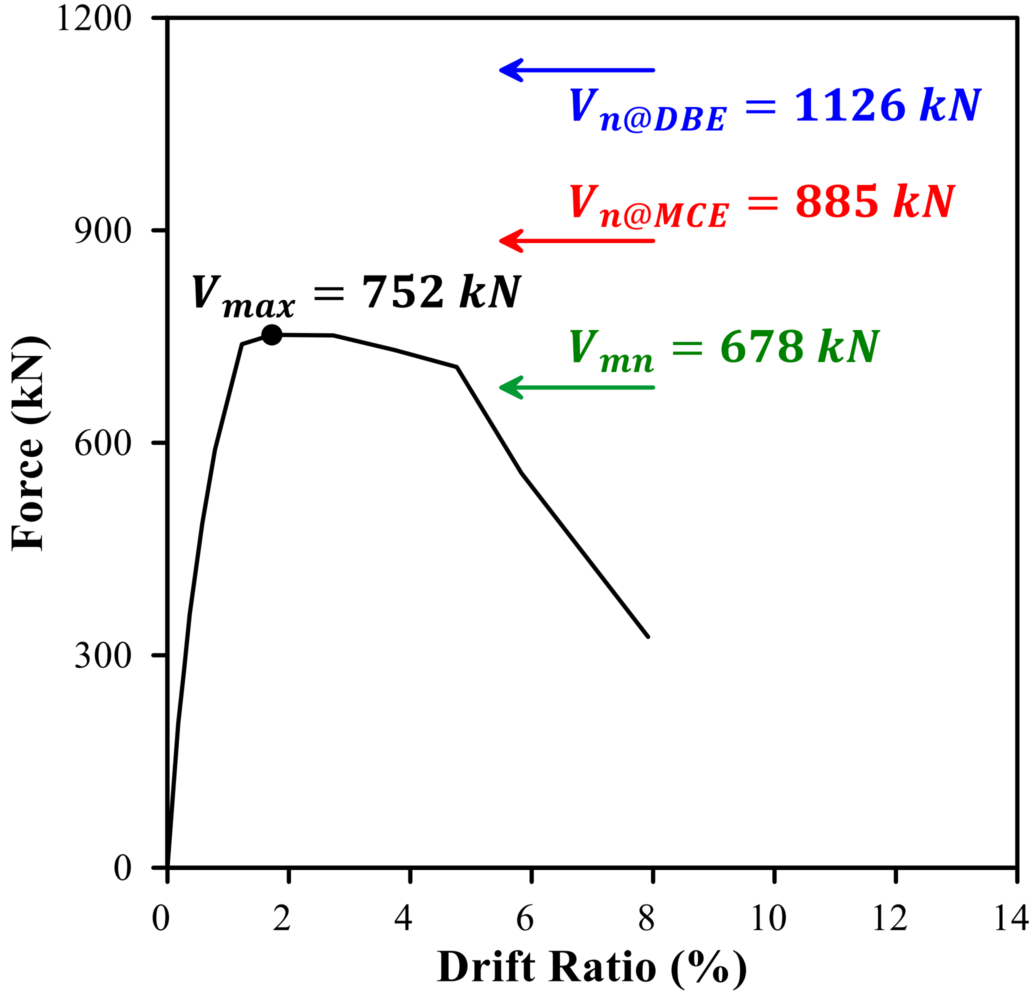
$(1.1 + 1.1) A_{vd} f_{yd,actual} \sin\alpha = 483\ kN$

$V_{n@MCE} = \min(V_{n,c}, V_{n,t}) + (1.1 + 1.1) A_{vd} f_{yd,actual} \sin\alpha = 707\ kN$

$\ell_n / h = 3.0$

CB30-H





撓曲與剪力強度評估

[Flexure Strength]

$V_{mn} = 2M_n/\ell_n = 678\ kN$

[Strength - DBE]

$kd = \left(\sqrt{(\rho_f n)^2 + 2\rho_f n} - \rho_f n\right) d = 177\ mm$

$A_{str} = b_w \times kd = 53099\ mm^2$

$\theta = \theta - \Delta\theta \le 26.5° \xrightarrow{take} 26.5°$

$K = \tan^A \theta + \cot^A \theta - 1 + 0.14B \le 1.64 \xrightarrow{take} 1.64$

$V_{n,c} = C_d \sin\theta = K A_{str} \zeta f_c' \sin\theta = 991\ kN$

$V_{n,t} = V_c + V_s = 0.17\sqrt{f_c'} b_w d + \frac{A_v f_{yt} d}{s} = 894\ kN$

$(1.0 + 1.0) A_{vd} f_{yd} \sin\alpha = 232\ kN$

$V_{n@DBE} = \min(V_{n,c}, V_{n,t}) + (1.0 + 1.0) A_{vd} f_{yd} \sin\alpha = 1126\ kN$

[Strength - MCE]

$a = \frac{1.1 f_{y,actual} \times A_{st}}{0.85 f_c' \times b_w} = 102\ mm$

$A_{str} = b_w \times a = 30611\ mm^2$

$\theta = \theta_{max} = 45°$

$K = \tan^A \theta + \cot^A \theta - 1 + 0.14B \le 1.64 \xrightarrow{take} 1.14$

$V_{n,c} = C_d \sin\theta = K A_{str} \zeta f_c' \sin\theta = 630\ kN$

$V_{n,t} = V_s = \frac{A_v f_{yt} d}{s} = 739\ kN$

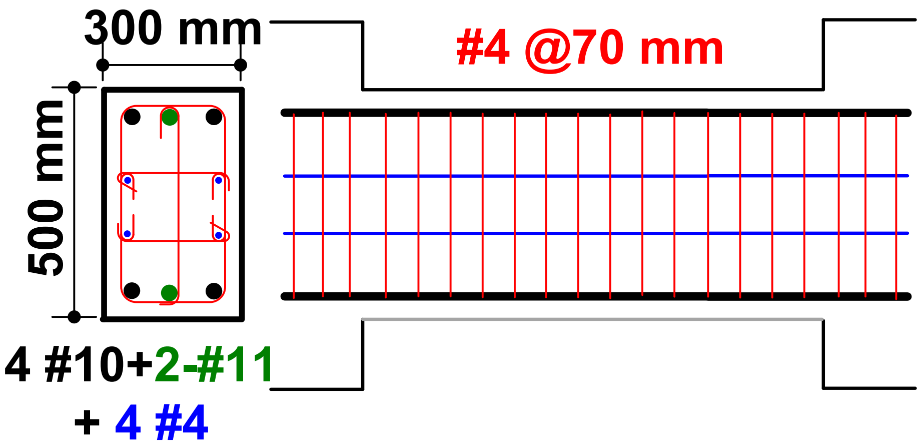
$(1.1 + 1.1) A_{vd} f_{yd,actual} \sin\alpha = 255\ kN$

$V_{n@MCE} = \min(V_{n,c}, V_{n,t}) + (1.1 + 1.1) A_{vd} f_{yd,actual} \sin\alpha = 885\ kN$

$\ell_n / h = 2.0$

實驗試體

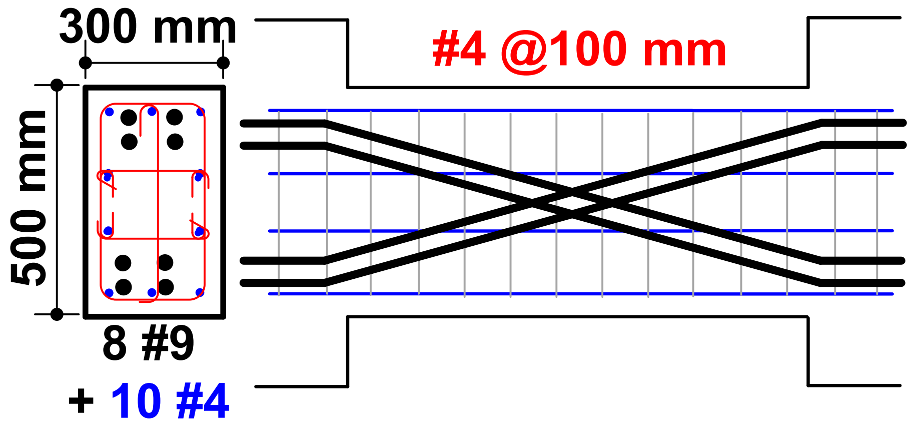
CB20-2



$f'_c = 54.2 MPa$

$\rho_f = 2.05\%$

CB20-1



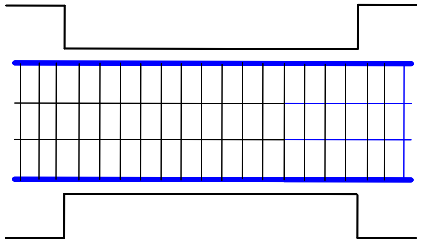
$f'_c = 52.1\ MPa$

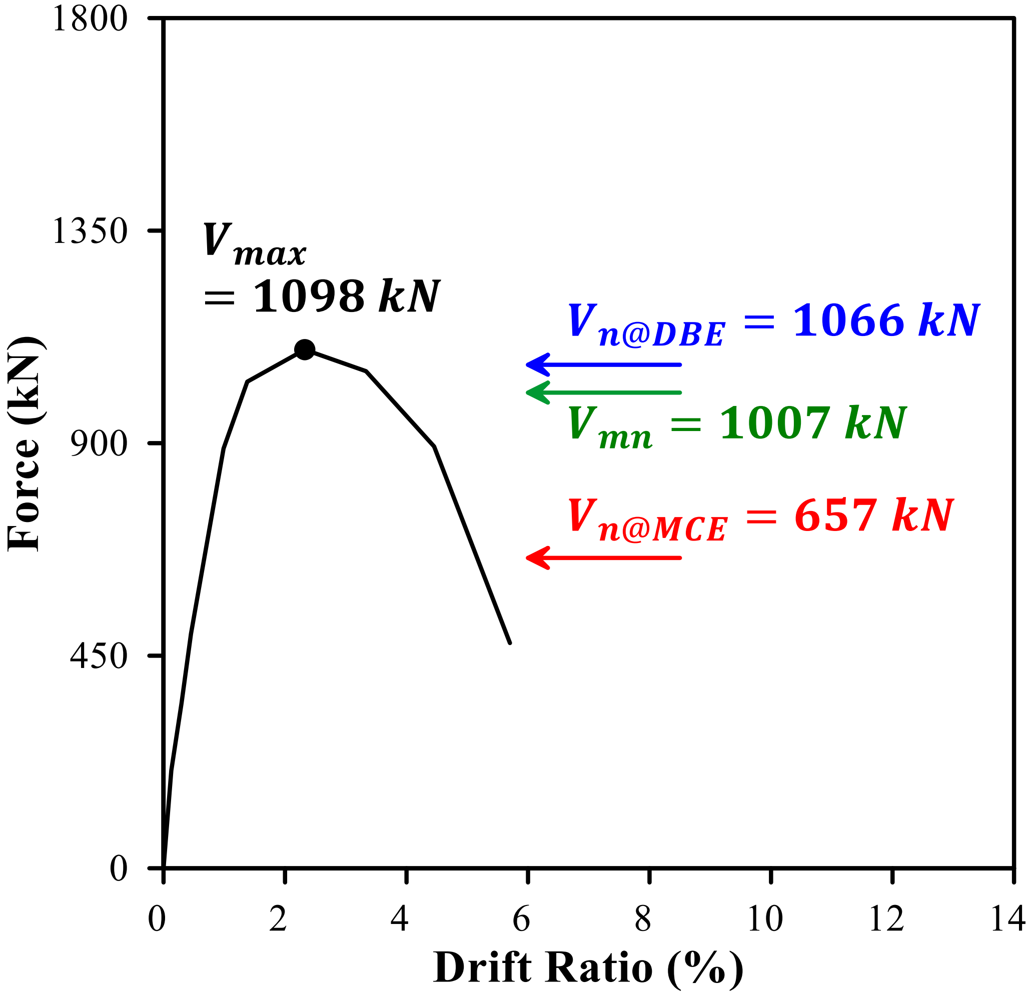
$\rho_f = 2.39\%$

$\ell_n / h = 2.0$

撓曲與剪力強度評估

CB20-2





[Flexure Strength]

$V_{mn} = 2M_n/\ell_n = 1007\ kN$

[Strength - DBE]

$kd = \left(\sqrt{(\rho_f n)^2 + 2\rho_f n} - \rho_f n\right) d = 179\ mm$

$A_{str} = b_w \times kd = 53203\ mm^2$

$\theta = \theta_{max} = 45.0°$

$K = \tan^A \theta + \cot^A \theta - 1 + 0.14B \leq 1.64 \xrightarrow{take} 1.14$

$V_{n,c} = C_d \sin\theta = K A_{str} \zeta f_c' \sin\theta = 1066\ kN$

$V_{n,t} = V_c + V_s = 0.17\sqrt{f_c'} b_w d + \frac{A_v f_{yt} d}{s} = 1334\ kN$

$V_{n@DBE} = \min(V_{n,c}, V_{n,t}) + 0 = 1066\ kN$

[Strength - MCE]

$a = \frac{1.1 f_{y,actual} \times A_{st}}{0.85 f_c' \times b_w} = 110\ mm$

$A_{str} = b_w \times a = 33034\ mm^2$

$\theta = \theta_{max} = 45.0°$

$K = \tan^A \theta + \cot^A \theta - 1 + 0.14B \leq 1.64 \xrightarrow{take} 1.14$

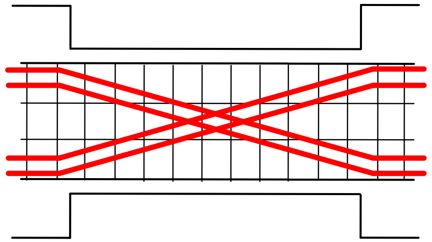
$V_{n,c} = C_d \sin\theta = K A_{str} \zeta f_c' \sin\theta = 657\ kN$

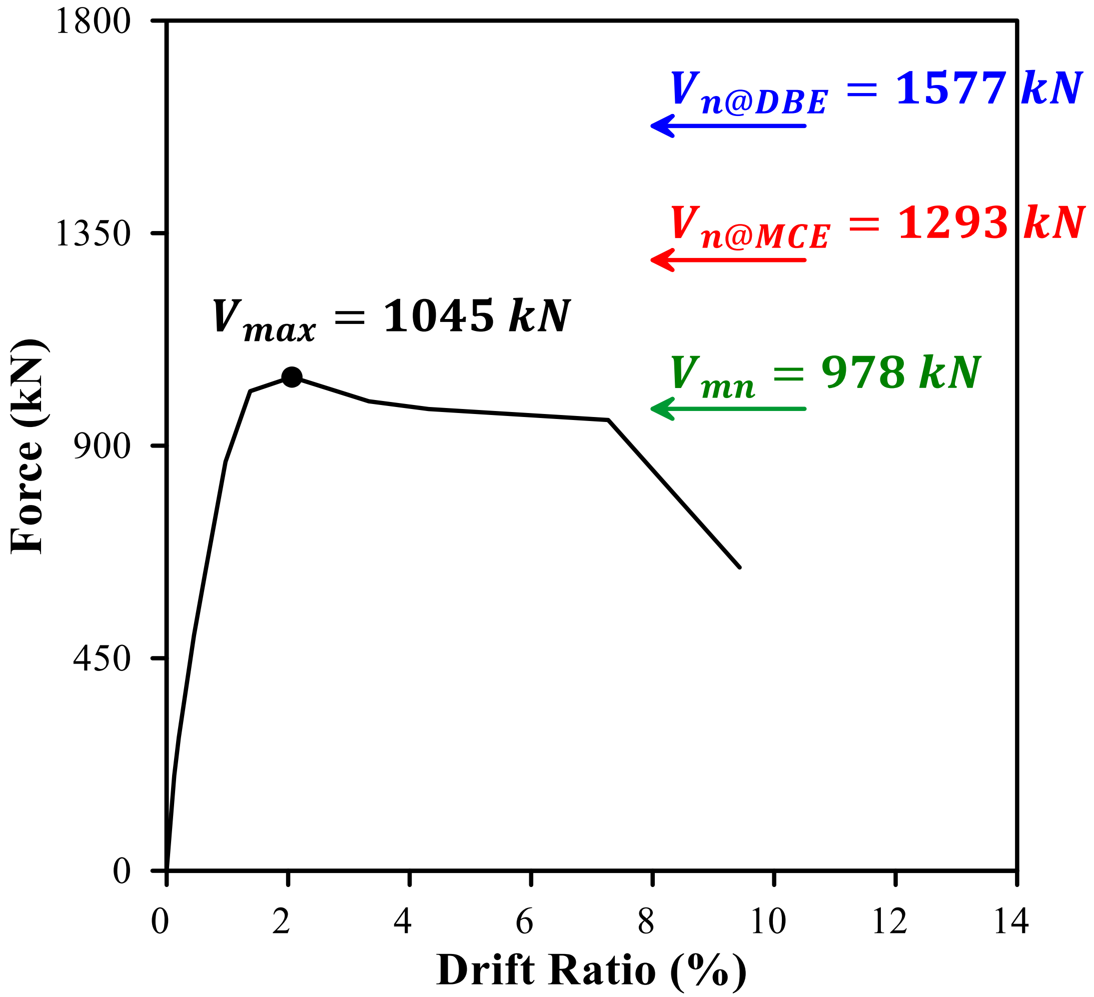
$V_{n,t} = V_s = \frac{A_v f_{yt} d}{s} = 1173\ kN$

$V_{n@MCE} = \min(V_{n,c}, V_{n,t}) + 0 = 657\ kN$

$\ell_n / h = 2.0$

CB20-1





撓曲與剪力強度評估

[Flexure Strength]

$V_{mn} = 2M_n/\ell_n = 978\ kN$

[Strength - DBE]

$kd = \left(\sqrt{(\rho_f n)^2 + 2\rho_f n} - \rho_f n\right) d = 177\ mm$

$A_{str} = b_w \times kd = 53203\ mm^2$

$\theta = \theta - \Delta\theta \le 26.5° \xrightarrow{take} 26.5°$

$K = \tan^A \theta + \cot^A \theta - 1 + 0.14B \le 1.64 \xrightarrow{take} 1.64$

$V_{n,c} = C_d \sin\theta = K A_{str} \zeta f_c' \sin\theta = 941\ kN$

$V_{n,t} = V_c + V_s = 0.17\sqrt{f_c'} b_w d + \frac{A_v f_{yt} d}{s} = 912\ kN$

$(1.0 + 1.0) A_{vd} f_{yd} \sin\alpha = 665\ kN$

$V_{n@DBE} = \min(V_{n,c}, V_{n,t}) + (1.0 + 1.0) A_{vd} f_{yd} \sin\alpha = 1577\ kN$

[Strength - MCE]

$a = \frac{1.1 f_{y,actual} \times A_{st}}{0.85 f_c' \times b_w} = 96\ mm$

$A_{str} = b_w \times a = 28810\ mm^2$

$\theta = \theta_{max} = 45.0°$

$K = \tan^A \theta + \cot^A \theta - 1 + 0.14B \le 1.64 \xrightarrow{take} 1.14$

$V_{n,c} = C_d \sin\theta = K A_{str} \zeta f_c' \sin\theta = 561\ kN$

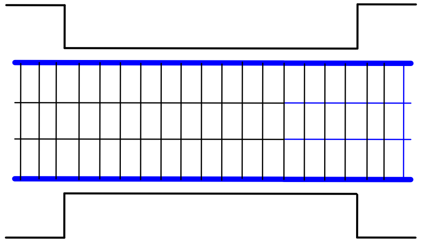
$V_{n,t} = V_s = \frac{A_v f_{yt} d}{s} = 1173\ kN$

$(1.1 + 1.1) A_{vd} f_{yd,actual} \sin\alpha = 732\ kN$

$V_{n@MCE} = \min(V_{n,c}, V_{n,t}) + (1.1 + 1.1) A_{vd} f_{yd,actual} \sin\alpha = 1293\ kN$

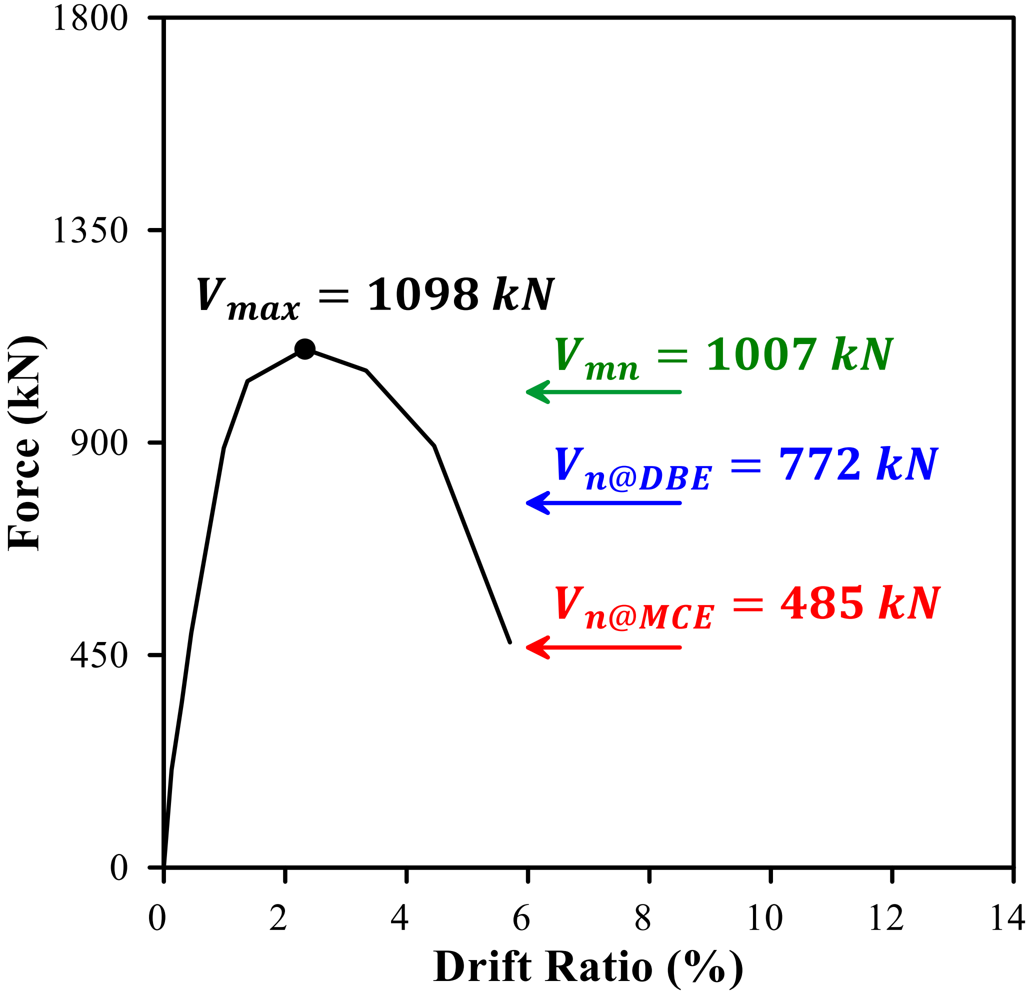
$\ell_n / h = 2.0$

CB20-2

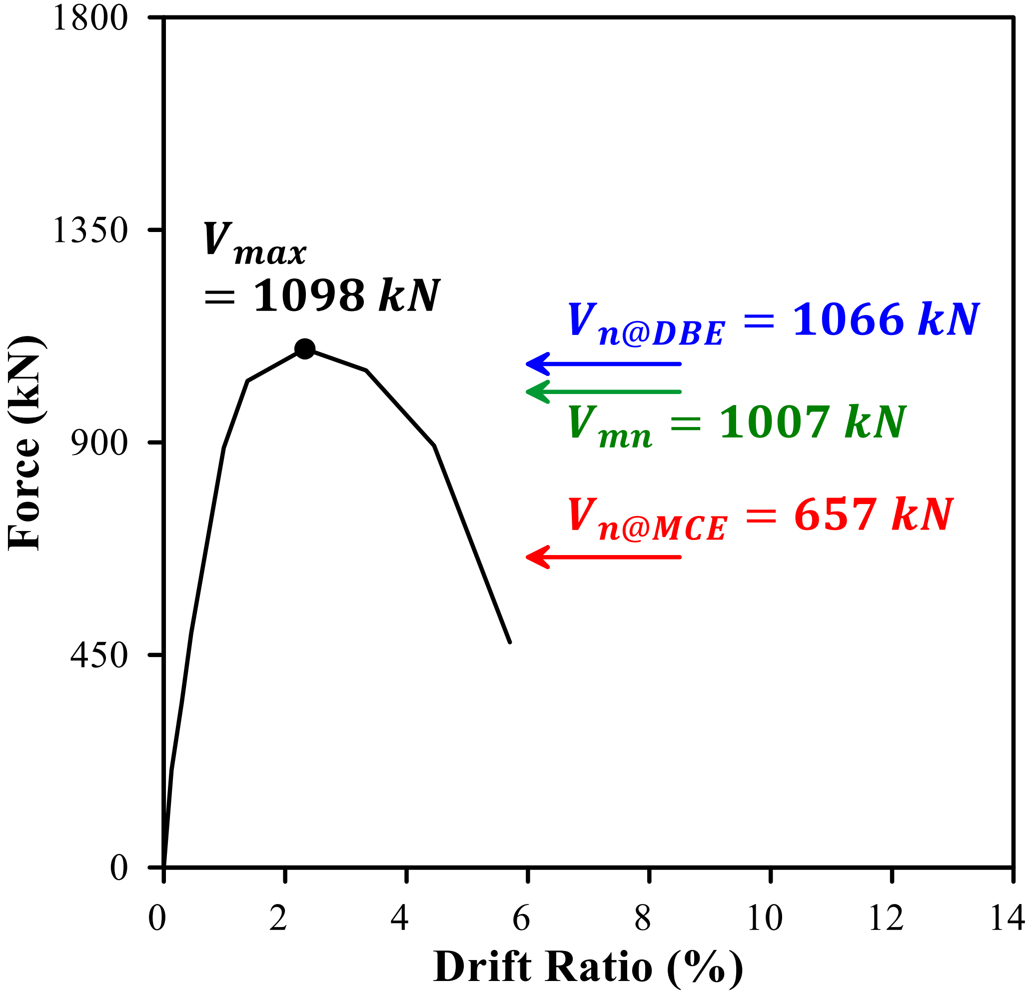


D-區域與DBD-區域評估之比較

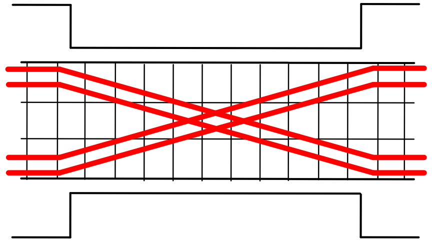
Assume singly-D region



Assume DBD region

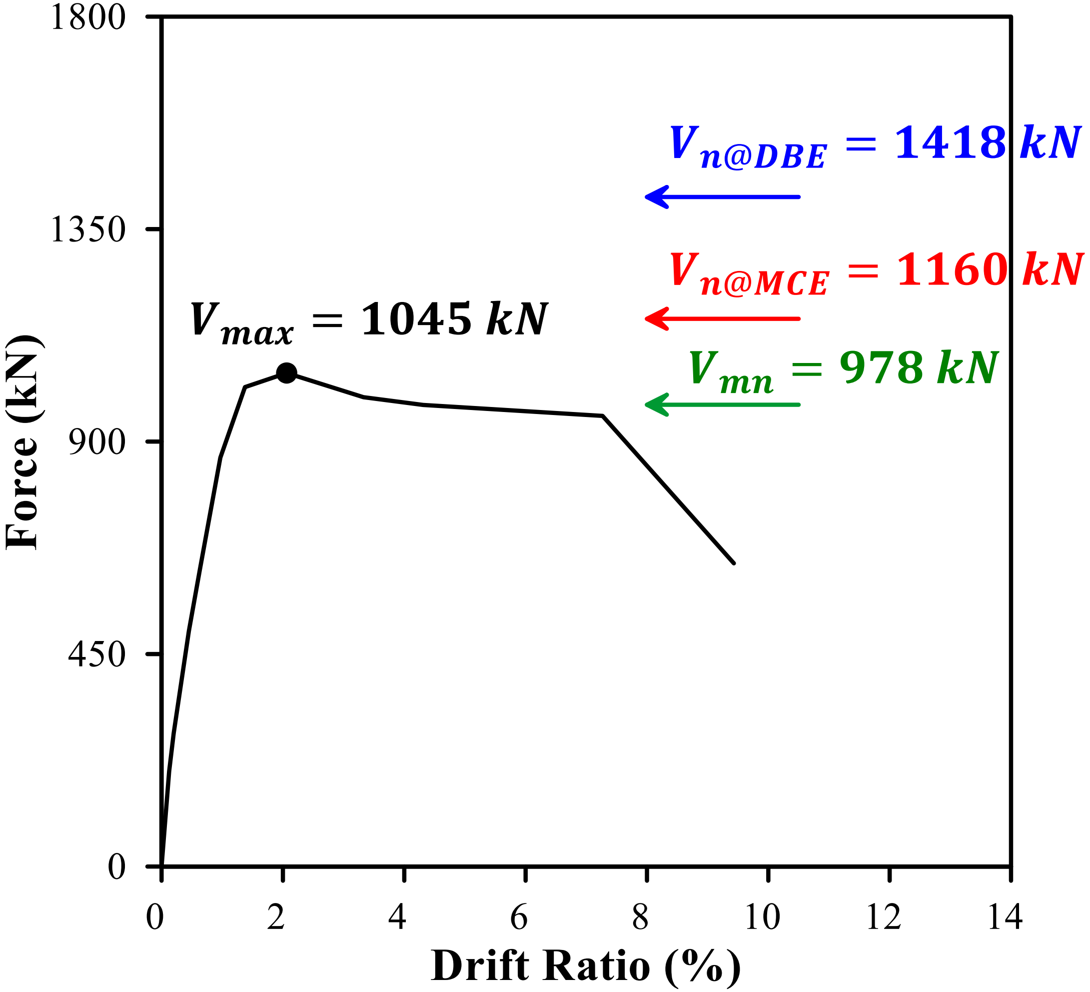


$\ell_n / h = 2.0$ **CB20-1**

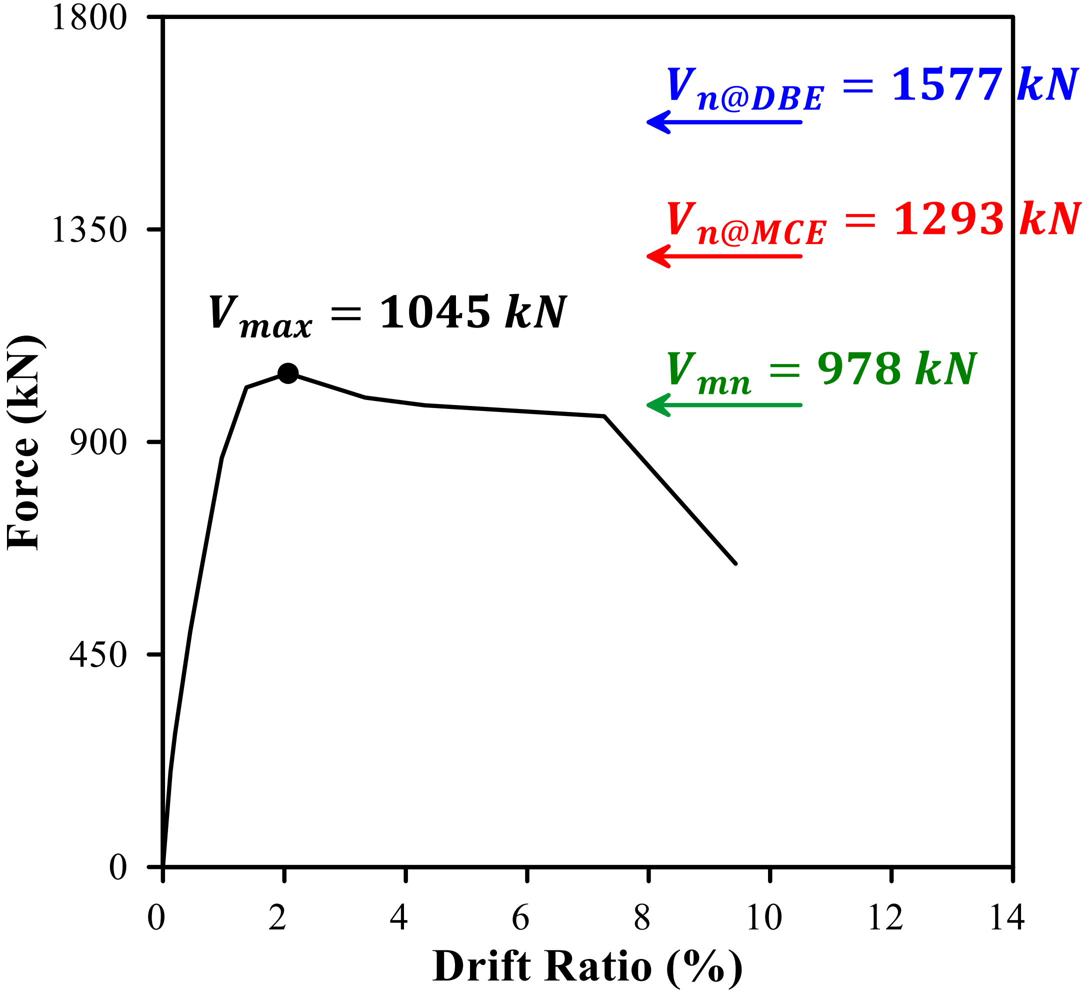


D-區域與DBD-區域評估之比較

Assume singly-D region

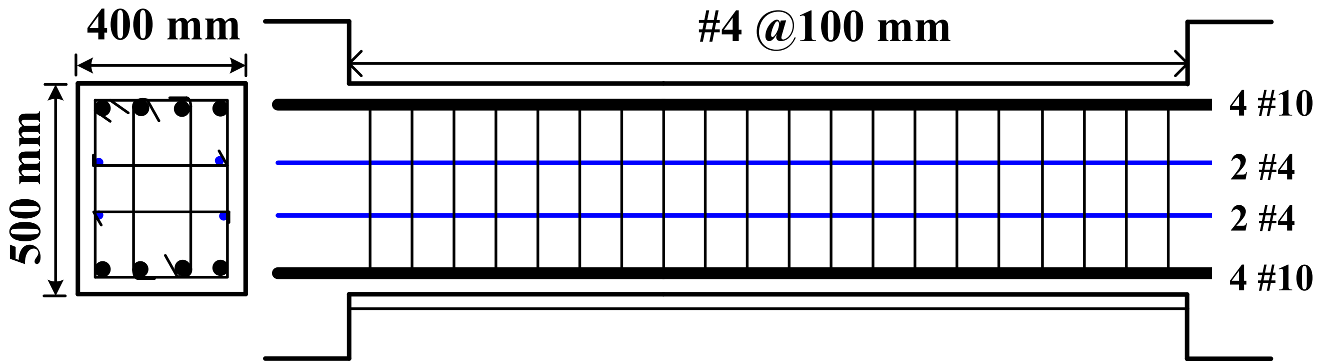


Assume DBD region



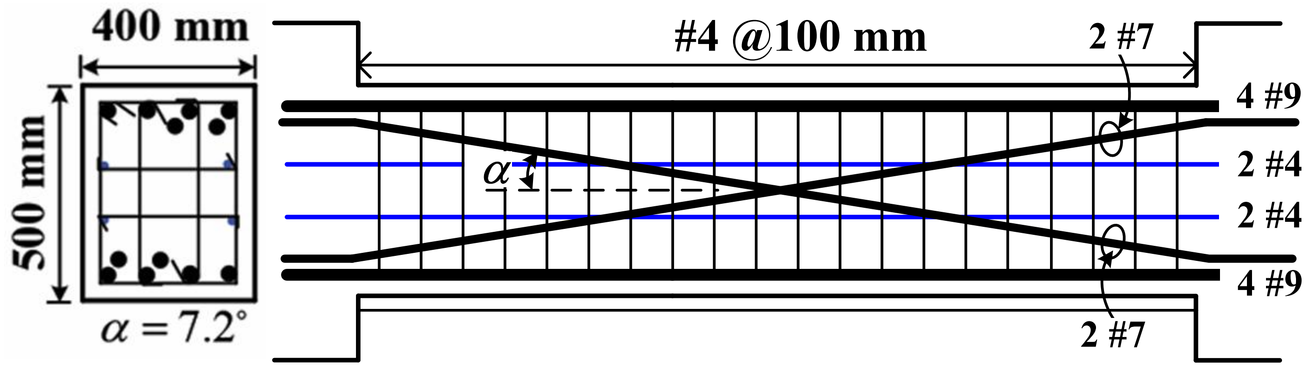
實驗試體 $\ell_n / h = 4.0$

CB40-C



$f_c' = 58.1\ MPa$
$\rho_f = 1.18\%$

CB40-H

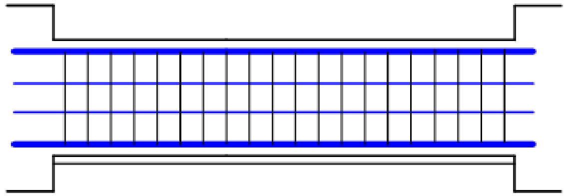


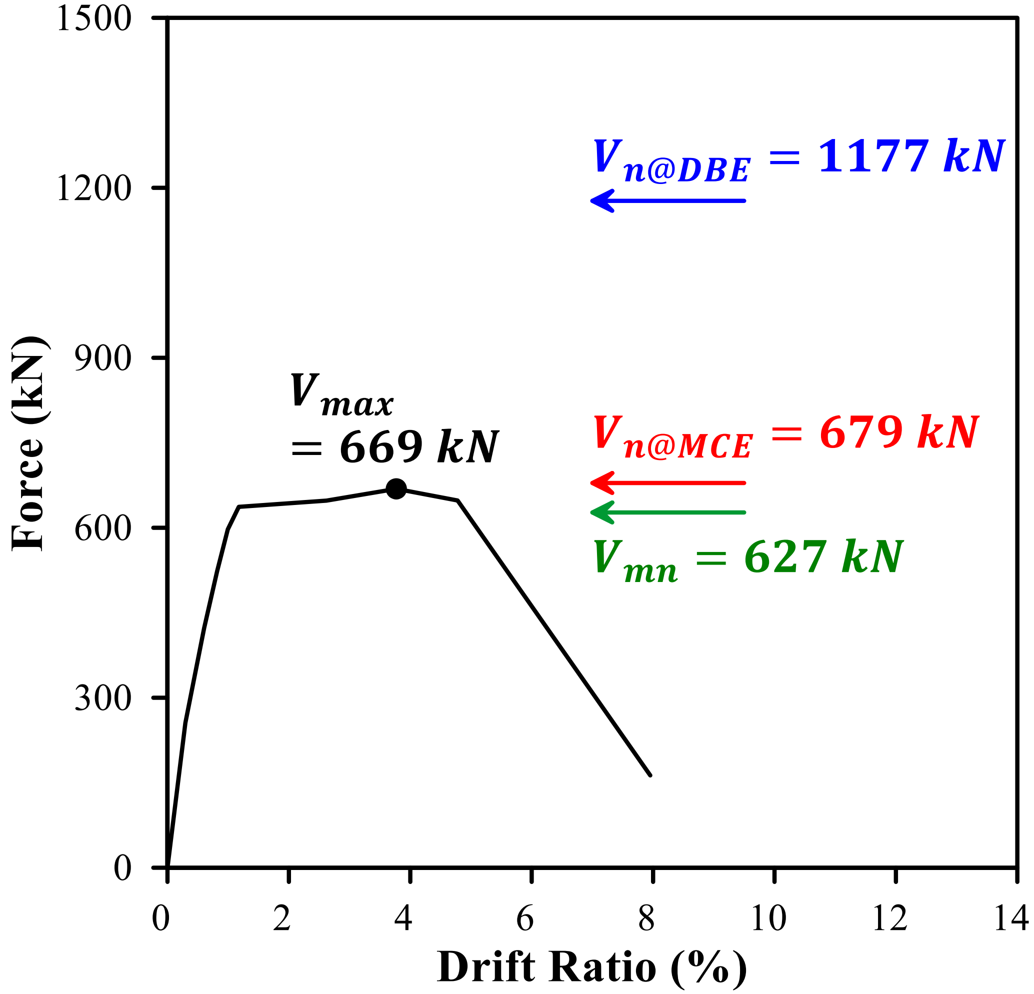
$f_c' = 58.5\ MPa$
$\rho_f = 1.29\%$

撓曲與剪力強度評估

$\ell_n / h = 4.0$

CB40-C





[Flexure Strength]

$V_{mn} = 2M_n/\ell_n = 627\ kN$

[Strength - DBE]

$kd = \left(\sqrt{(\rho_f n)^2 + 2\rho_f n} - \rho_f n\right) d = 143\ mm$

$A_{str} = b_w \times kd = 57185\ mm^2$

$\theta = \theta_{max} = 45.0°$

$K = \tan^A \theta + \cot^A \theta - 1 + 0.14B \leq 1.64 \xrightarrow{take} 1.14$

$V_{n,c} = C_d \sin\theta = K A_{str} \zeta f_c' \sin\theta = 1177\ kN$

$V_{n,t} = V_c + V_s = 0.17\sqrt{f_c'} b_w d + \frac{A_v f_{yt} d}{s} = 1286\ kN$

$V_{n@DBE} = \min(V_{n,c}, V_{n,t}) + 0 = 1177\ kN$

[Strength - MCE]

$a = \frac{1.1 f_{y,actual} \times A_{st}}{0.85 f_c' \times b_w} = 83\ mm$

$A_{str} = b_w \times a = 33011\ mm^2$

$\theta = \theta_{max} = 45.0°$

$K = \tan^A \theta + \cot^A \theta - 1 + 0.14B \leq 1.64 \xrightarrow{take} 1.14$

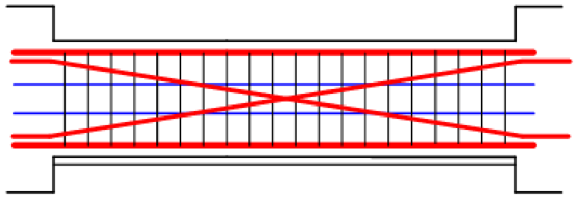
$V_{n,c} = C_d \sin\theta = K A_{str} \zeta f_c' \sin\theta = 679\ kN$

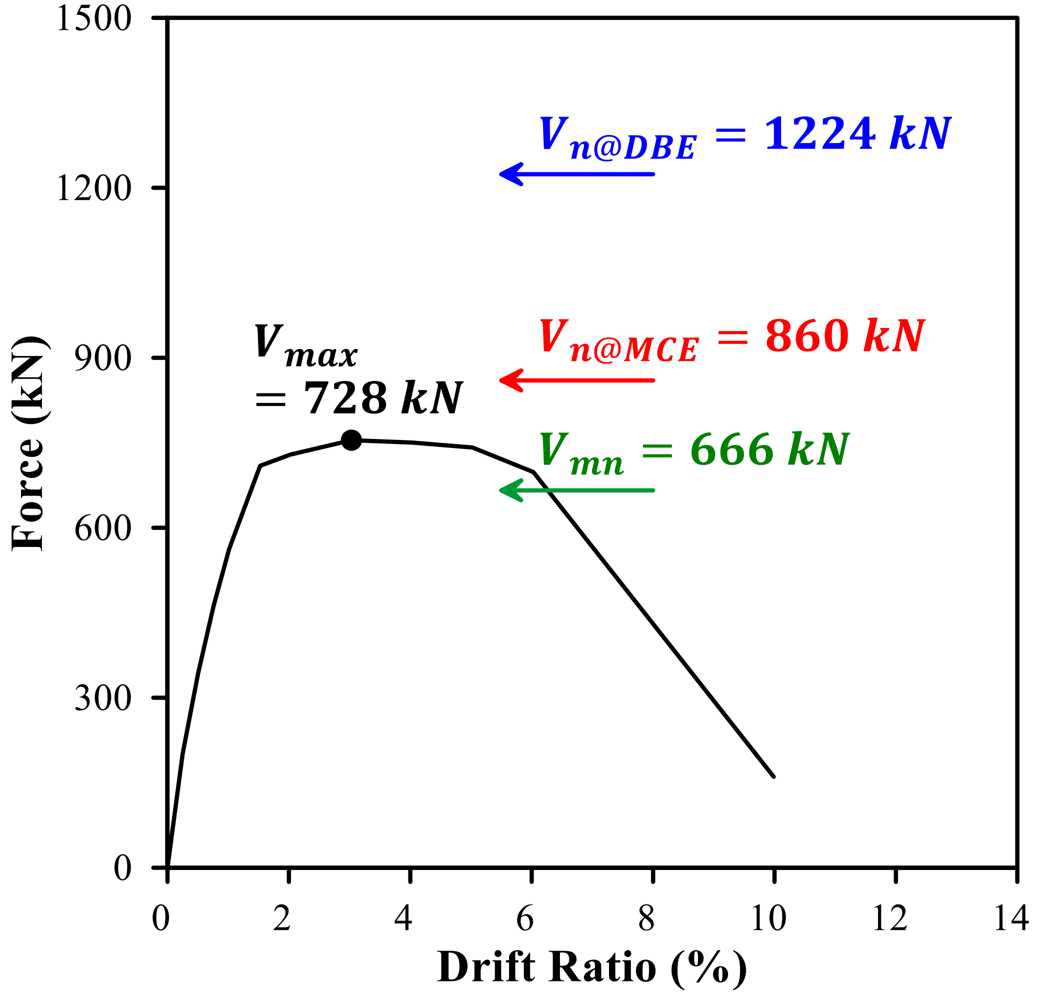
$V_{n,t} = V_s = \frac{A_v f_{yt} d}{s} = 1062\ kN$

$V_{n@MCE} = \min(V_{n,c}, V_{n,t}) + 0 = 679\ kN$

$\ell_n / h = 4.0$

CB40-H





撓曲與剪力強度評估

[Flexure Strength]

$V_{mn} = 2M_n/\ell_n = 666\ kN$

[Strength - DBE]

$kd = \left(\sqrt{(\rho_f n)^2 + 2\rho_f n} - \rho_f n\right) d = 137\ mm$

$A_{str} = b_w \times kd = 54959\ mm^2$

$\theta = \theta_{max} = 45.0°$

$K = \tan^A \theta + \cot^A \theta - 1 + 0.14B \leq 1.64 \xrightarrow{take} 1.14$

$V_{n,c} = C_d \sin\theta = K A_{str} \zeta f_c' \sin\theta = 1135\ kN$

$V_{n,t} = V_c + V_s = 0.17\sqrt{f_c'} b_w d + \frac{A_v f_{yt} d}{s} = 1193\ kN$

$(1.0 + 1.0) A_{vd} f_{yd} \sin\alpha = 89\ kN$

$V_{n@DBE} = \min(V_{n,c}, V_{n,t}) + (1.0 + 1.0) A_{vd} f_{yd} \sin\alpha = 1224\ kN$

[Strength - MCE]

$a = \frac{1.1 f_{y,actual} \times A_{st}}{0.85 f_c' \times b_w} = 92\ mm$

$A_{str} = b_w \times a = 36878\ mm^2$

$\theta = \theta_{max} = 45.0°$

$K = \tan^A \theta + \cot^A \theta - 1 + 0.14B \leq 1.64 \xrightarrow{take} 1.14$

$V_{n,c} = C_d \sin\theta = K A_{str} \zeta f_c' \sin\theta = 762\ kN$

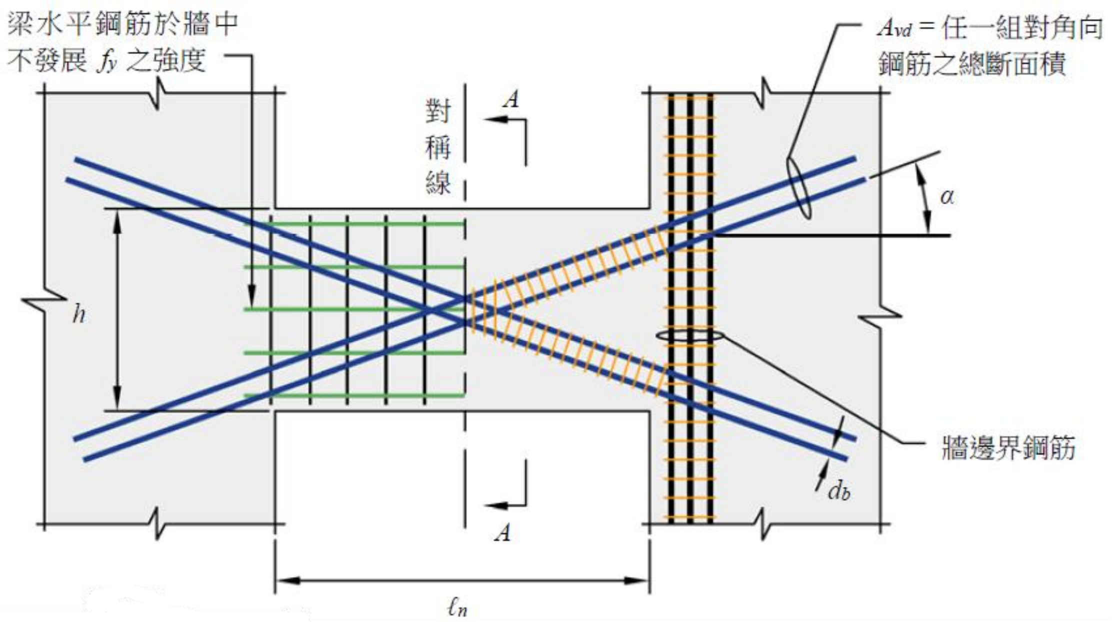
$V_{n,t} = V_s = \frac{A_v f_{yt} d}{s} = 985\ kN$

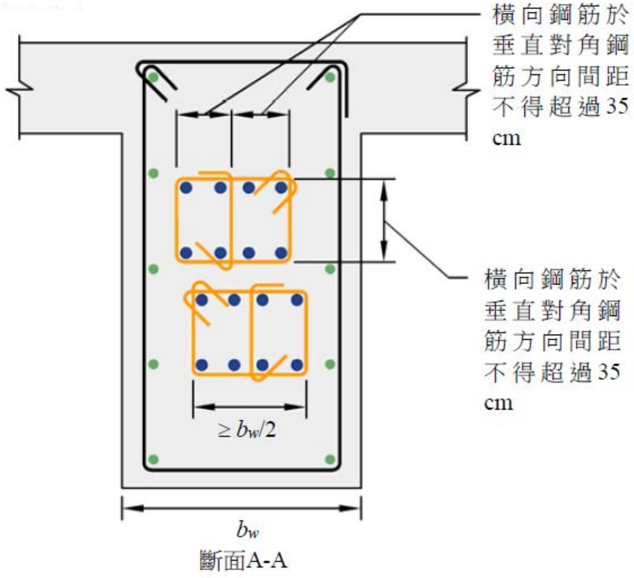
$(1.1 + 1.1) A_{vd} f_{yd,actual} \sin\alpha = 98\ kN$

$V_{n@MCE} = \min(V_{n,c}, V_{n,t}) + (1.1 + 1.1) A_{vd} f_{yd,actual} \sin\alpha = 860\ kN$

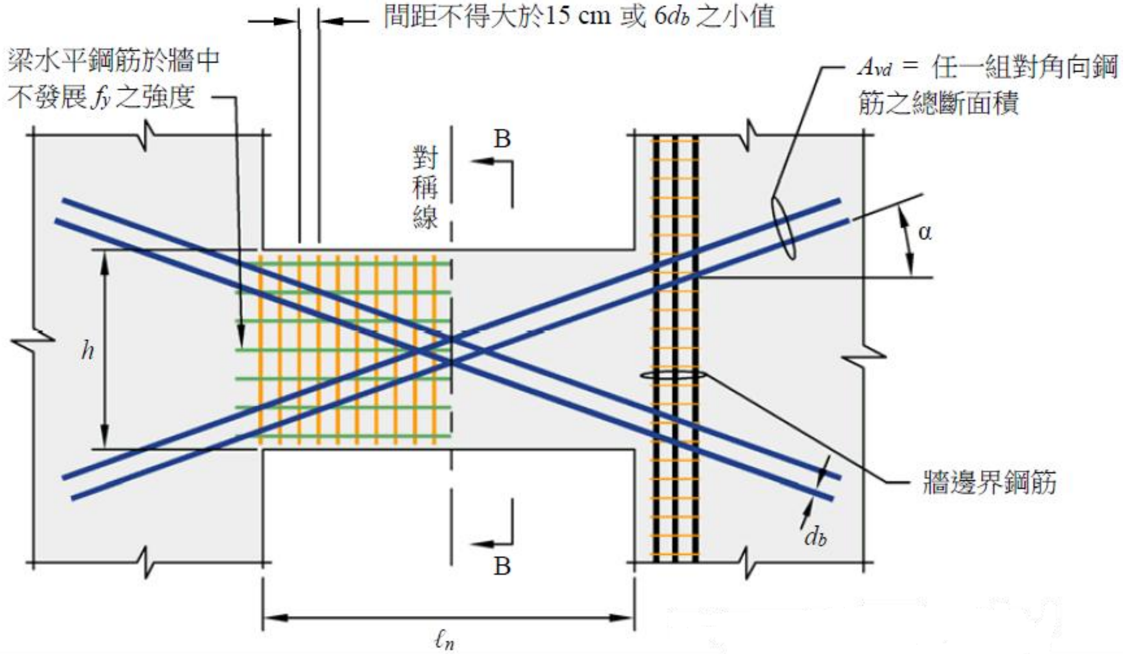
附錄：連接梁圍束效應之探討

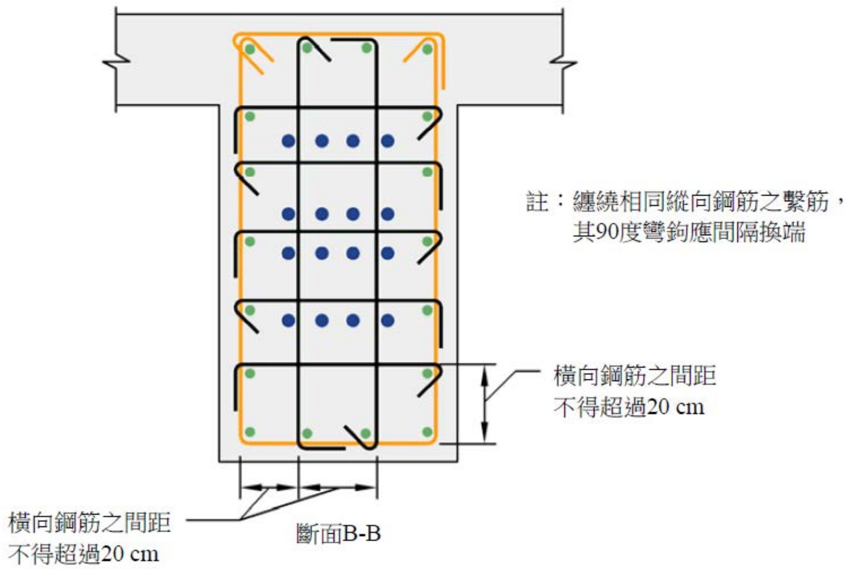
圍束對角鋼筋





連接梁斷面全圍束





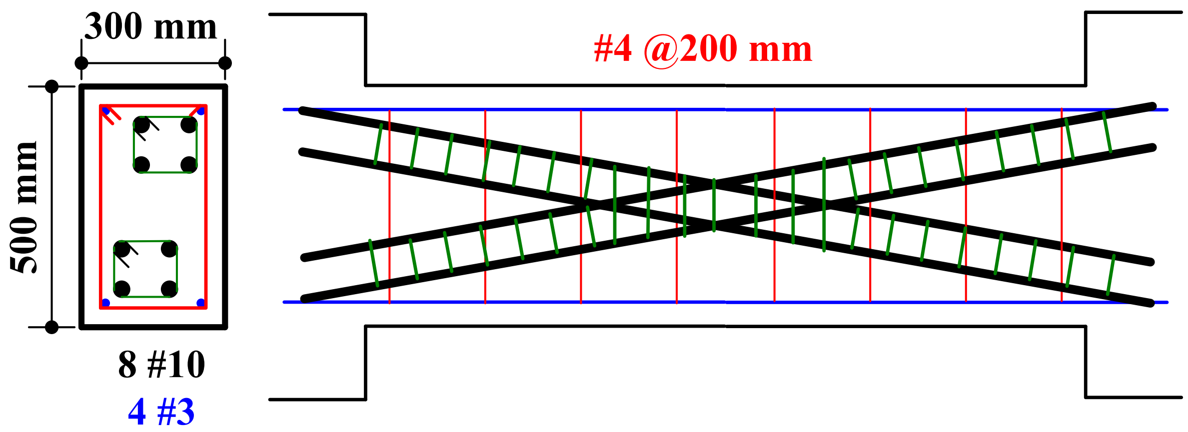
$\ell_n / h = 3.0$

實驗試體

CB30-DA

$f_c' = 39.7\ MPa$

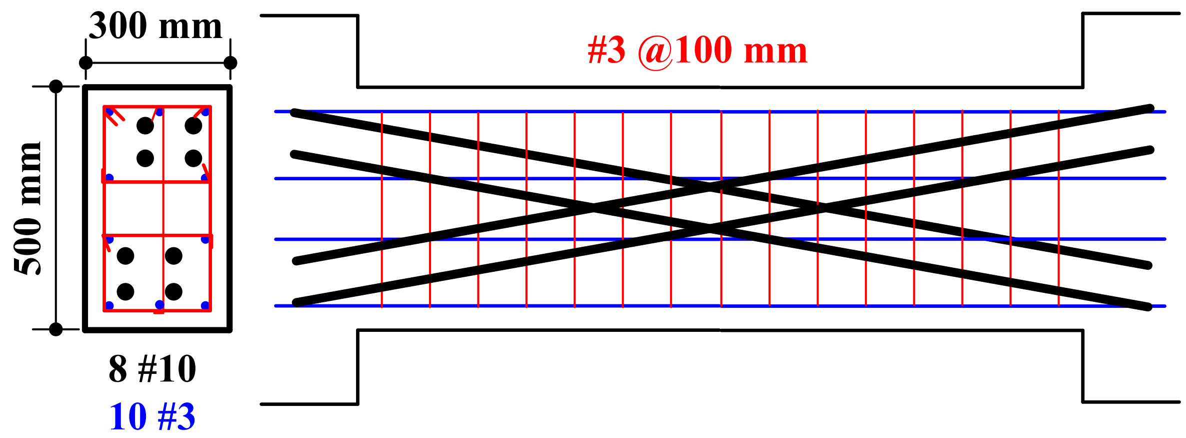
$\rho_f = 2.89\%$



CB30-DB

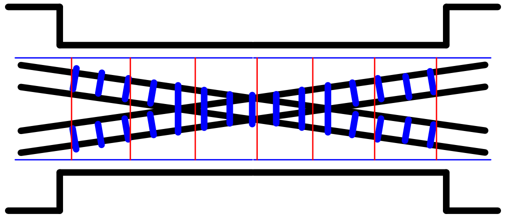
$f_c' = 38.4\ MPa$

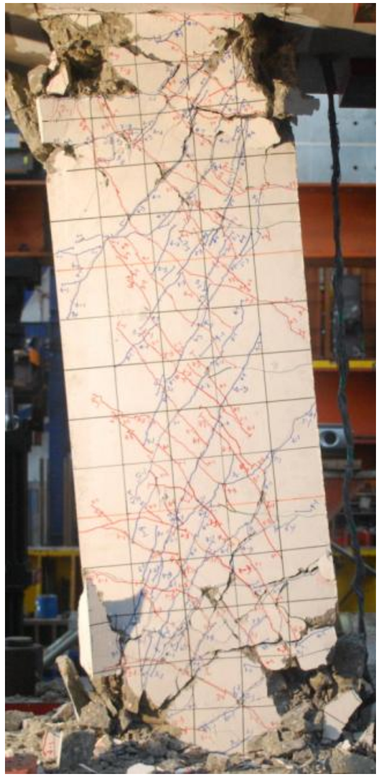
$\rho_f = 2.86\%$



試驗結果

CB30-DA

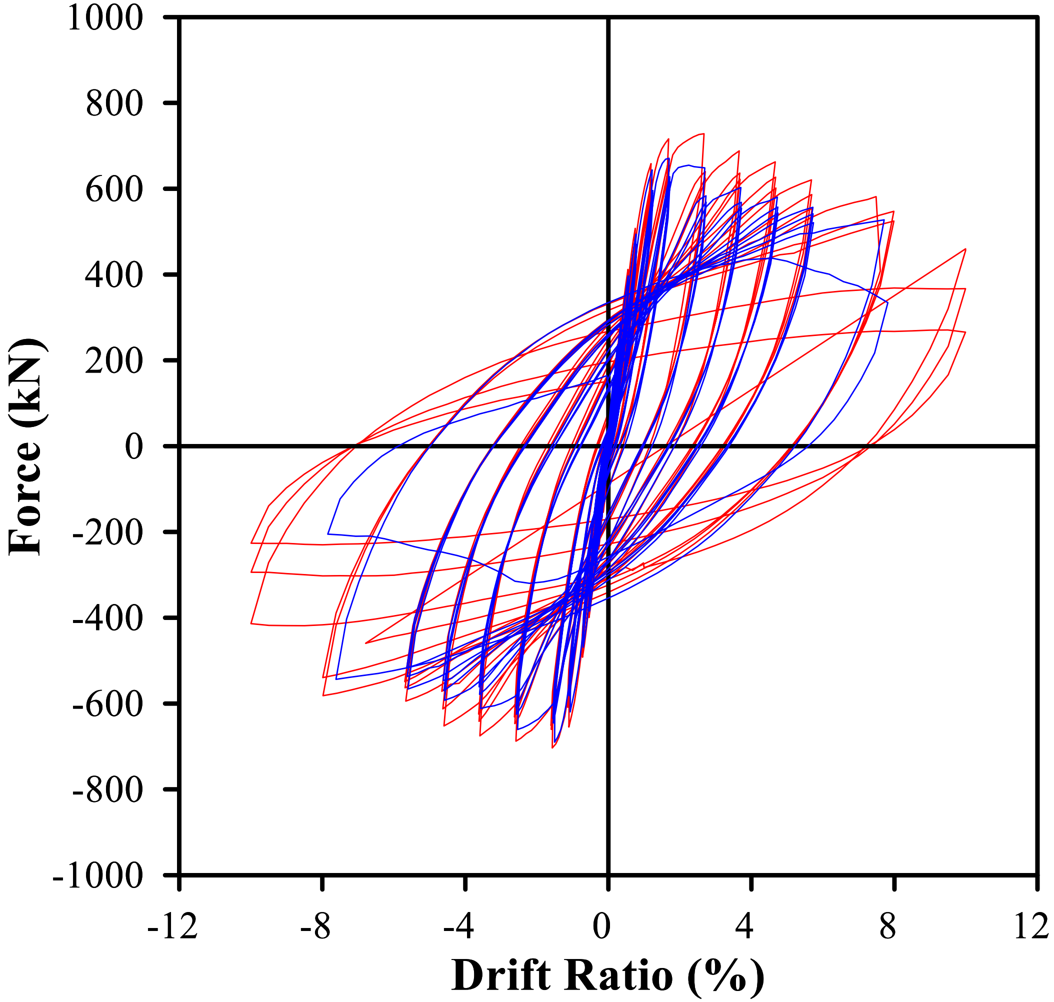




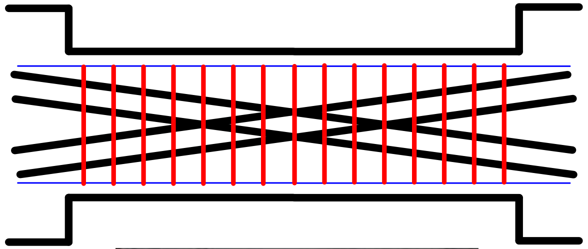
7.7%

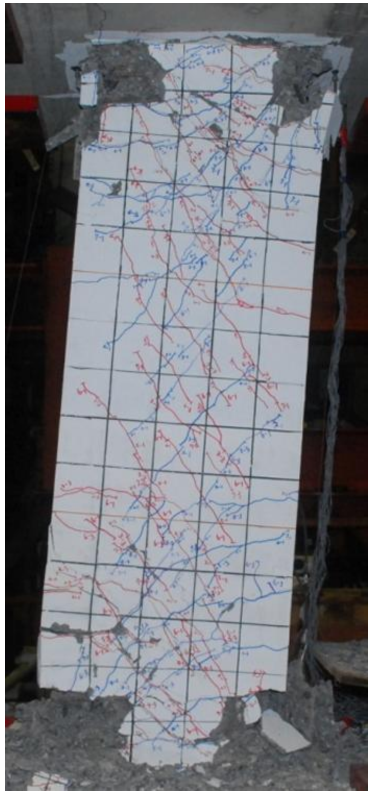
$$\upsilon = 0.98\sqrt{f_c'}$$

UDR=7.0%



CB30-DB





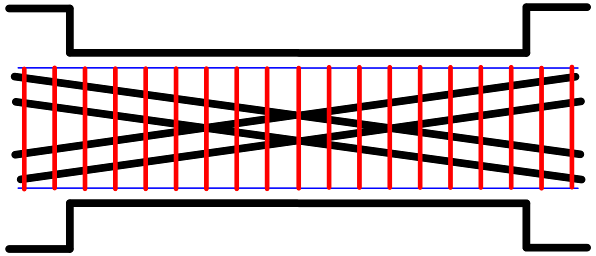
8.0%

$$\upsilon = 1.04\sqrt{f_c'}$$

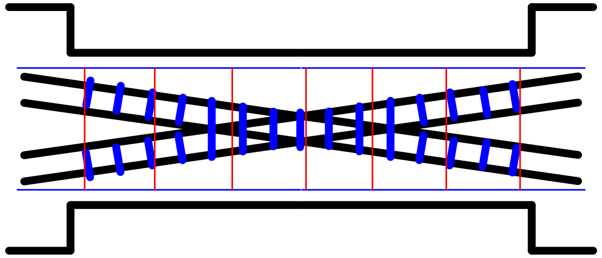
UDR=7.4%

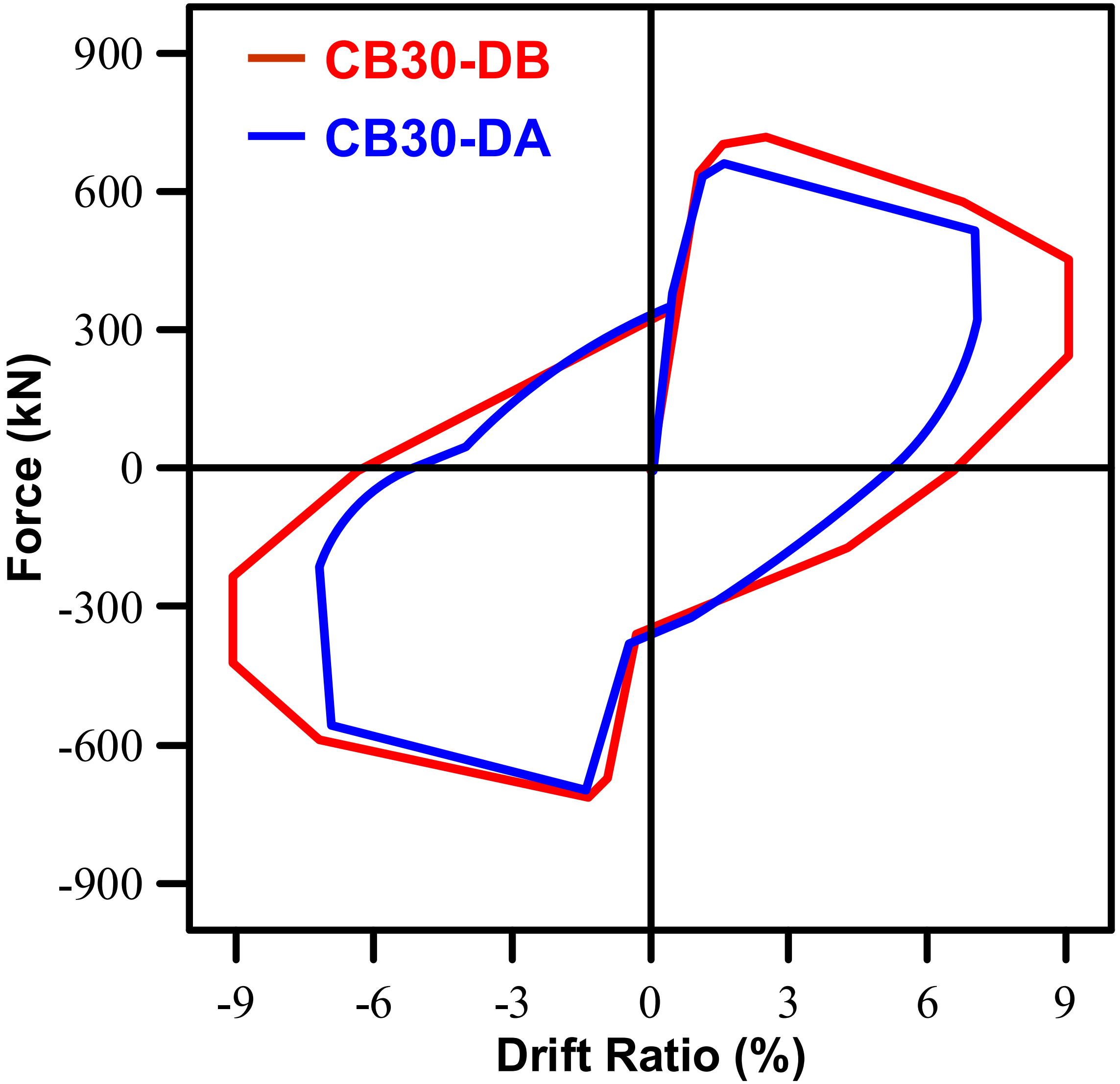
遲滯消能效應

CB30-DB (Type B)



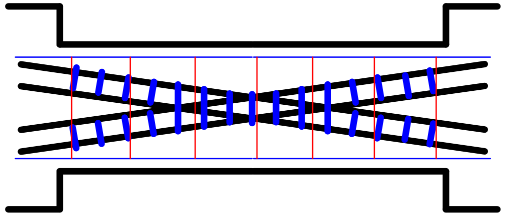
CB30-DA (Type A)

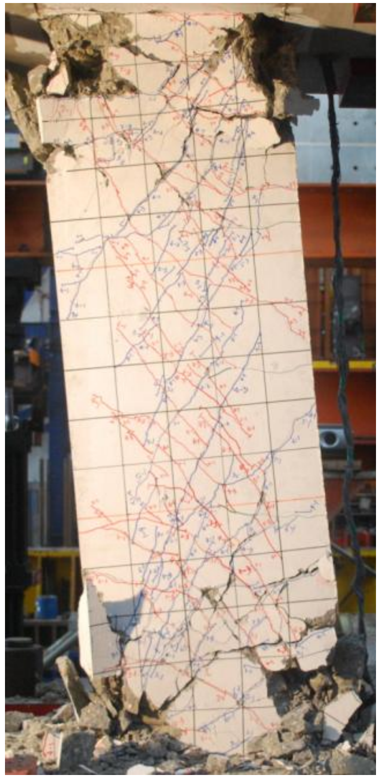


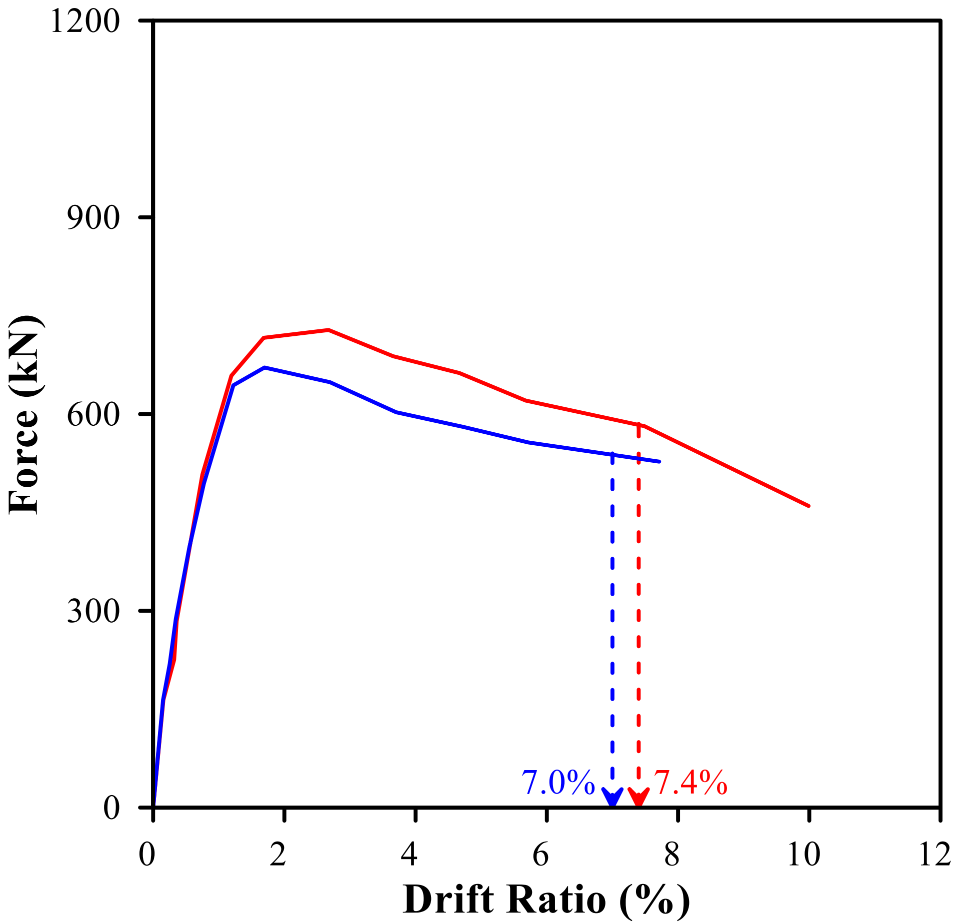


極限層間位移角

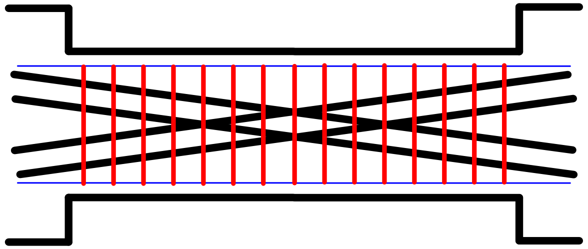
CB30-DA





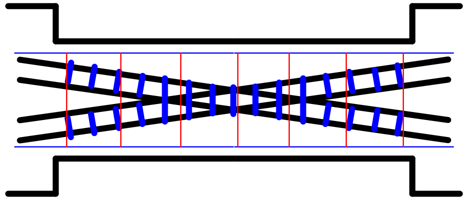


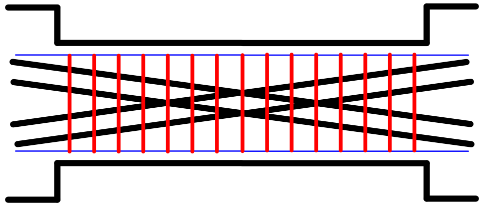
CB30-DB



充份橫向鋼筋束制全梁之效果較佳

CB30-DA

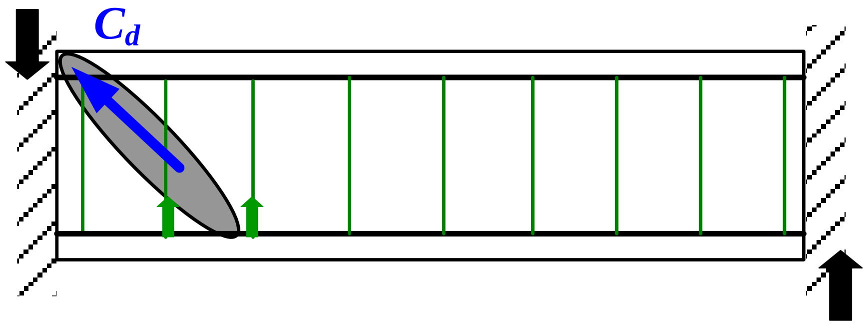


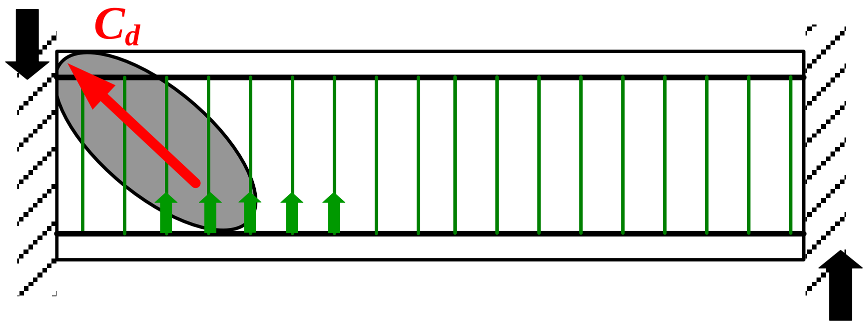


CB30-DB

(對角鋼筋) = (對角鋼筋)

(梁橫向鋼筋) < (梁橫向鋼筋)



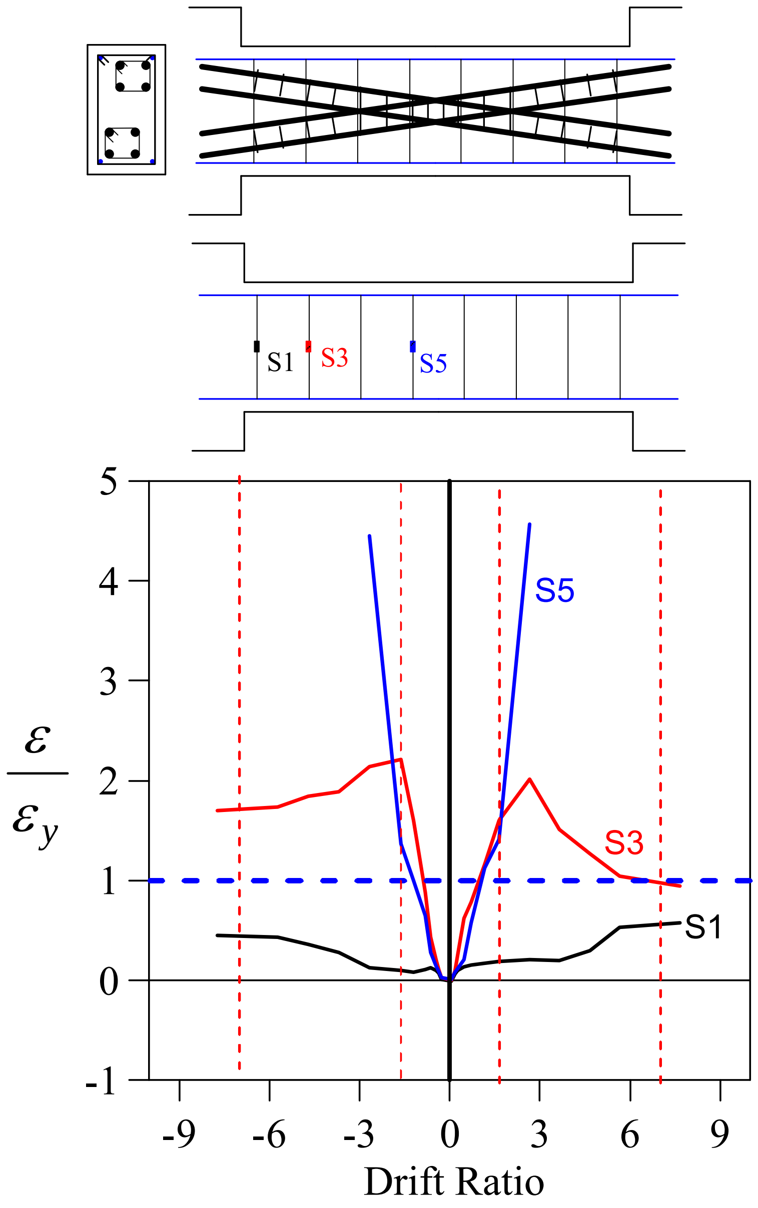


(壓桿強度C_d) < (壓桿強度C_d)

極限位移角=7.0% < 極限位移角=7.4%

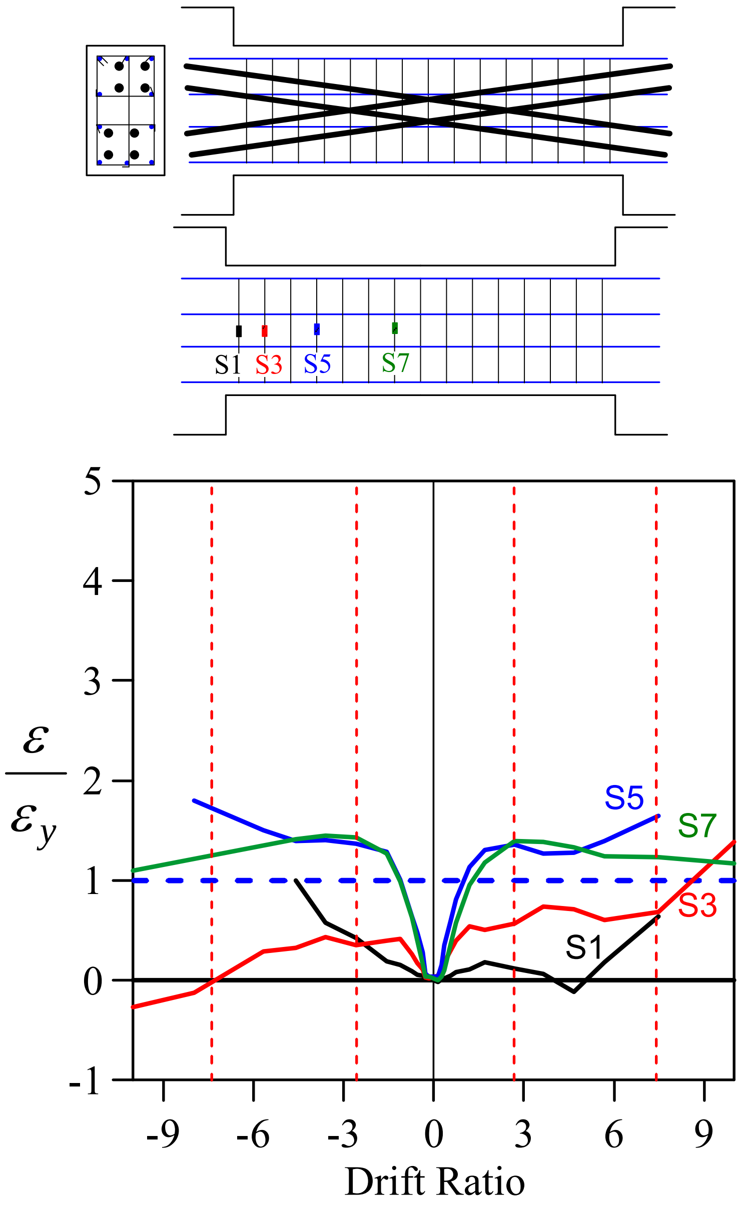
箍筋之應變量測

圍束對角鋼筋



Yield more

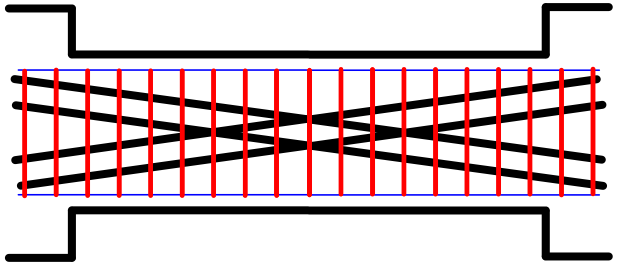
連接梁斷面全圍束



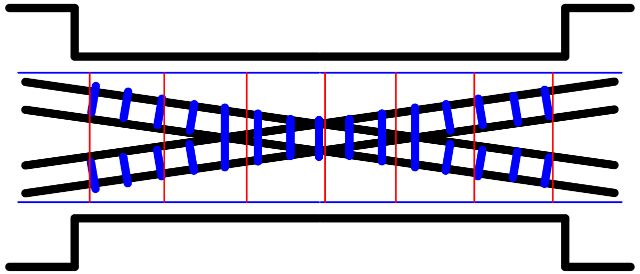
Yield less

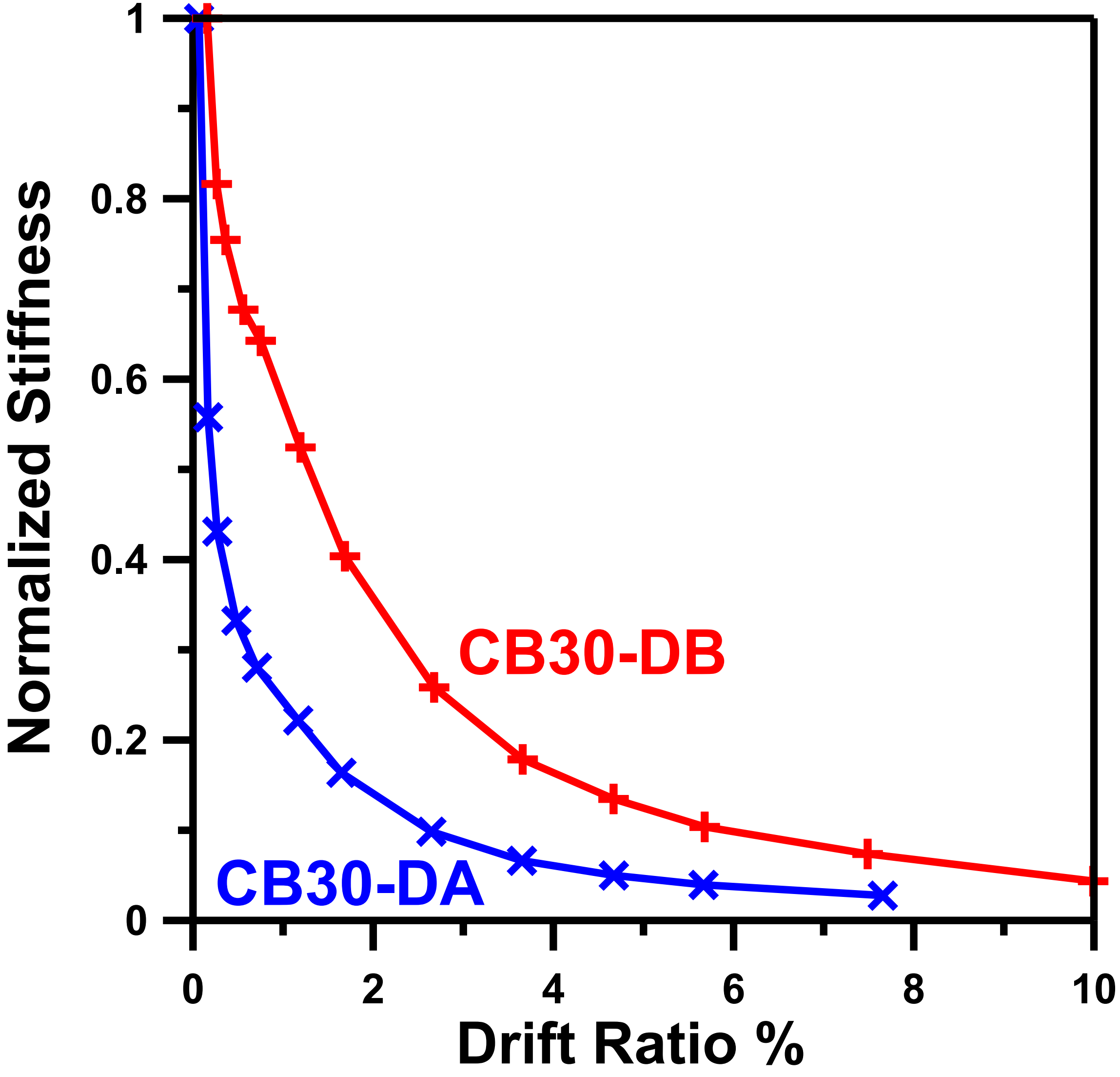
勁度衰減

CB30-DB (Type B)

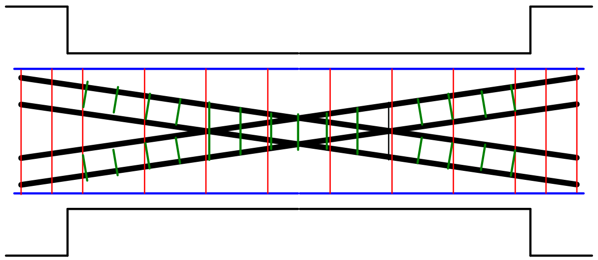


CB30-DA (Type A)

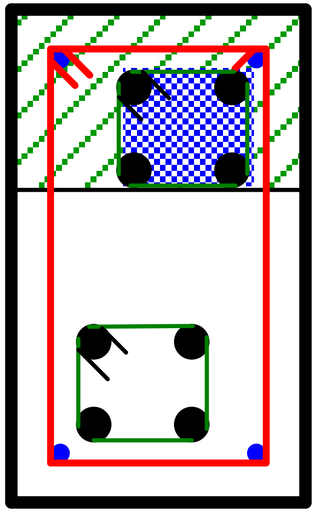


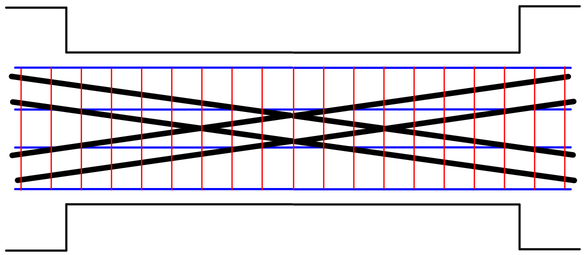


CB30-DA (Type A) vs CB30-DB (Type B)

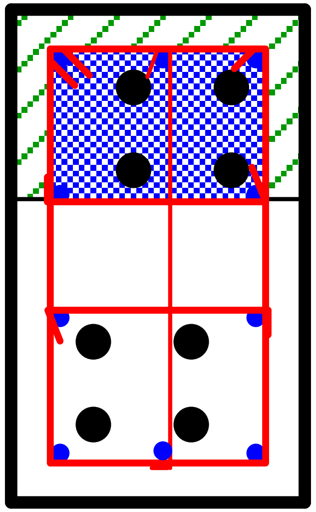


- Confinement only for diagonal bars
- Smaller confined area for compressive strut





- Confinement for whole section
- Larger confined area for compressive strut



鋼筋混凝土開孔牆之模擬

黃世建 教授
吳冠穎 研究生
蔡仁傑 博士後研究員
土木工程學系

國立臺灣大學
National Taiwan University

大綱

- 前言
- 開孔牆實驗觀察
- 分析理論
- 單窗開孔牆之驗證
- 多重開孔牆之模擬與驗證
- 電腦分析模型之建議
- 耐震評估與補強之應用
- 開孔牆之壓拉桿配筋方法
- 結論

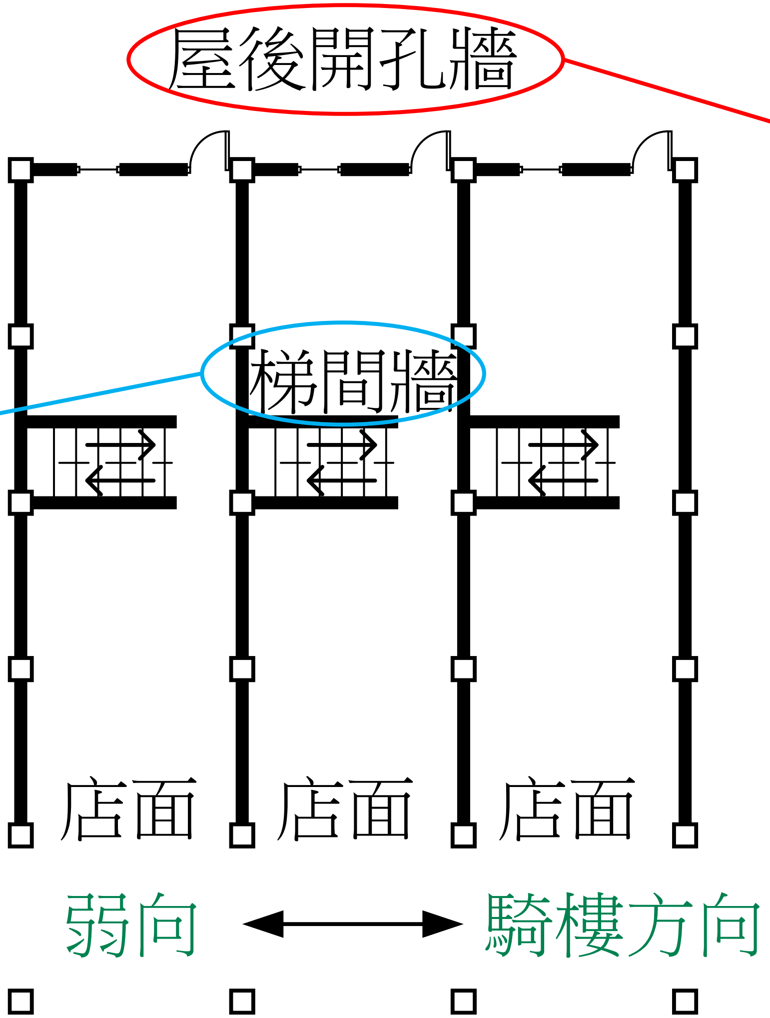
前言

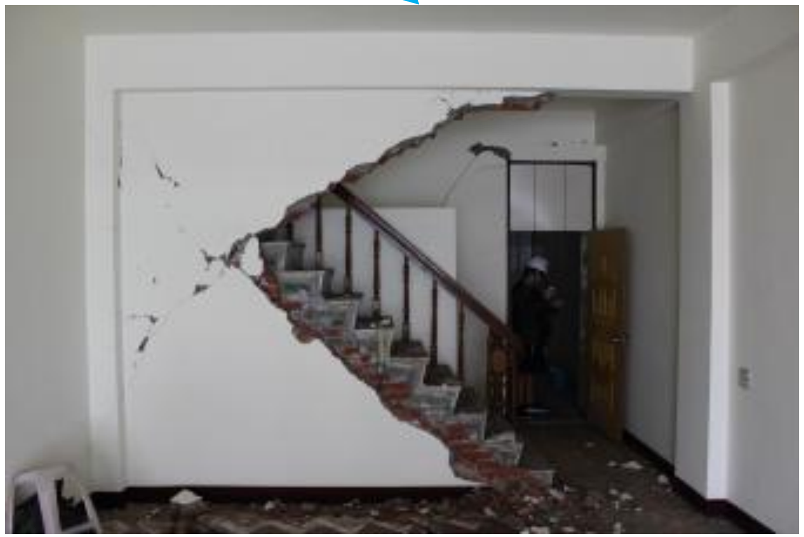
台灣街屋

杜昱石，(2014)「低矮鋼筋混凝土街屋具典型開口外牆之耐震行為研究」，碩士論文，
國立台灣科技大學，營建工程學系，臺北，臺灣。

於耐震評估與補強時應予以考量

- 騎樓街道方向為結構之弱向
- 皆為開孔牆

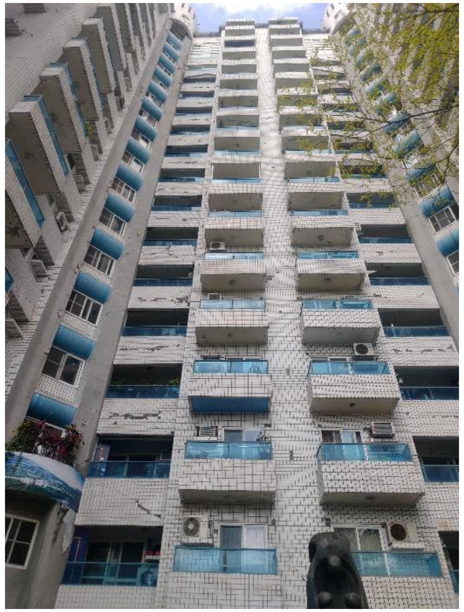


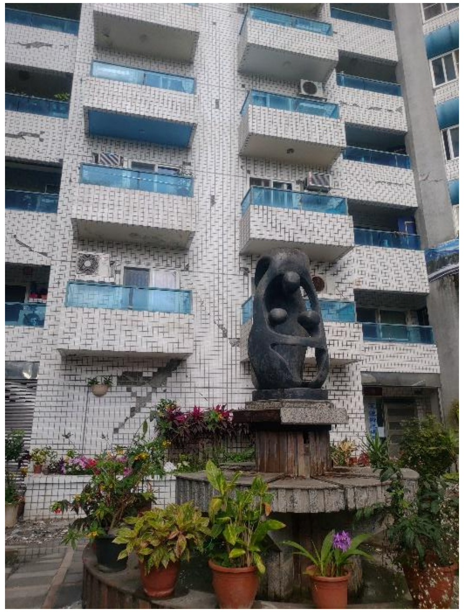


開孔牆之剪力破壞

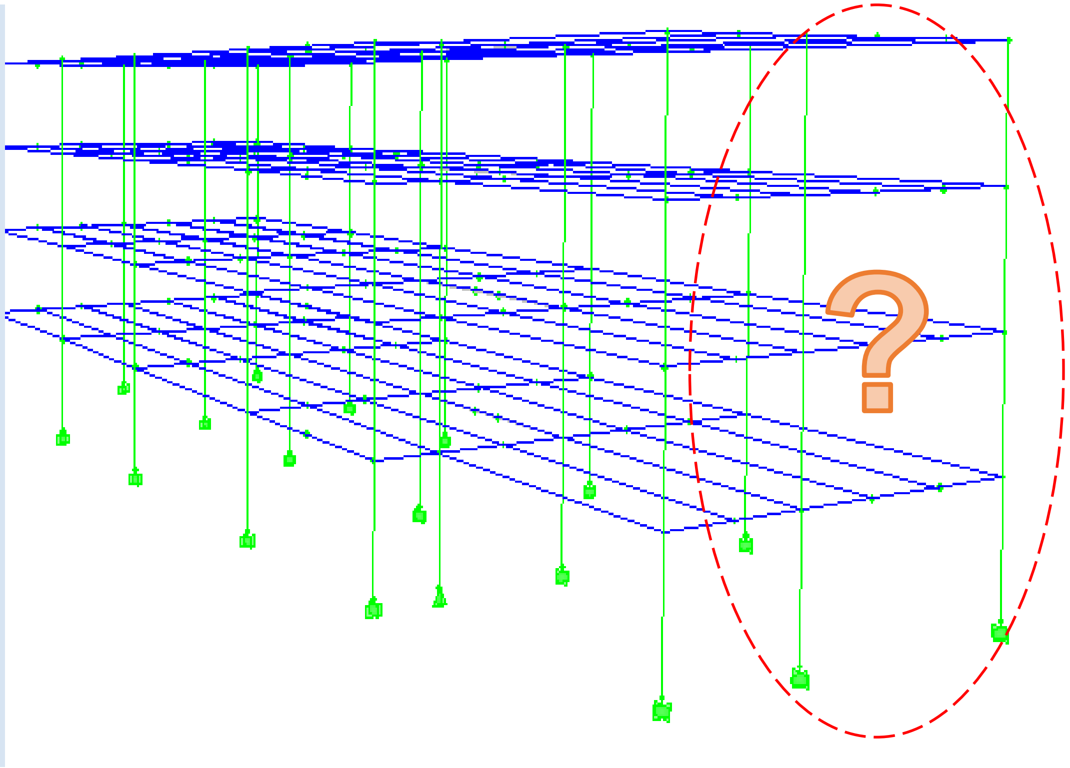
Torre O'Higgins Building
Damaged by 2010 Chile Earthquake

開孔牆之剪力破壞 – 2024 花蓮地震



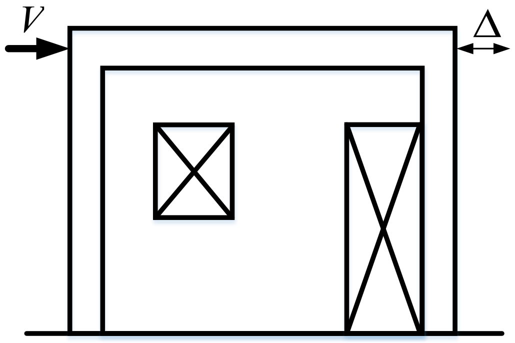


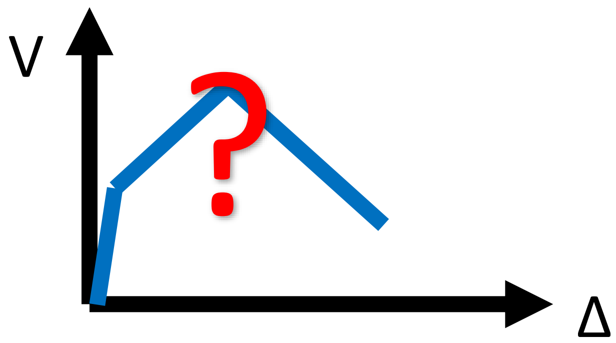
於功能設計時不宜忽略



講習會目的

- 建立開孔牆側力位移曲線分析模型





- 應用範圍：

  耐震評估與補強

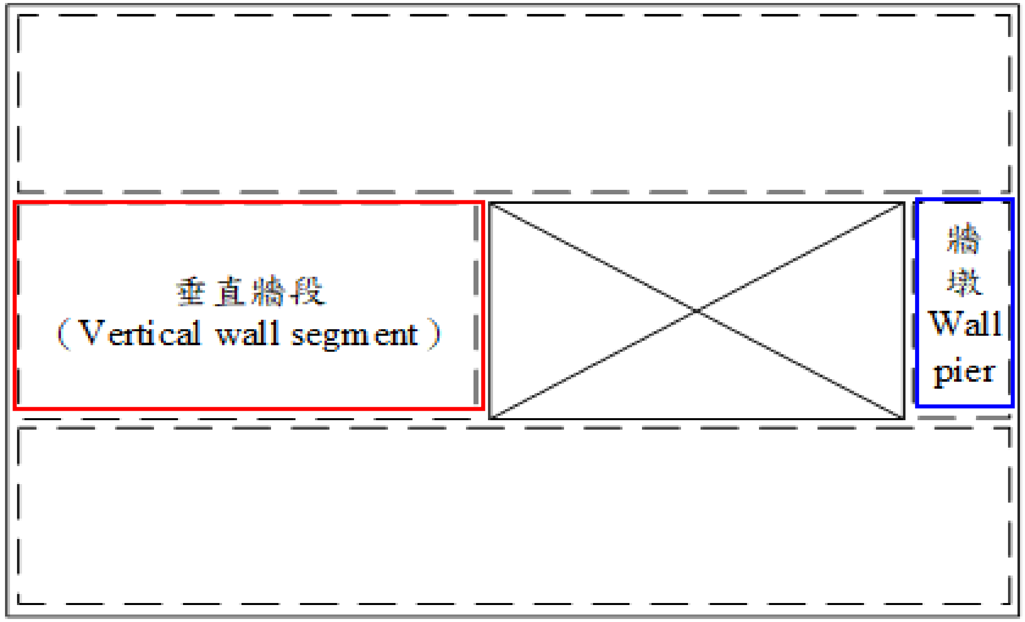
  功能設計

  開孔牆之損傷控制

開孔牆實驗觀察

Yeh, R. L., Tseng, C. C., and Hwang, S. J. (2018). "Shear Strength of Reinforced Concrete Vertical Wall Segments under Seismic Loading," ACI Structural Journal, V. 115, No. 5, September, pp. 1485-1494.

垂直牆段與牆墩之分類



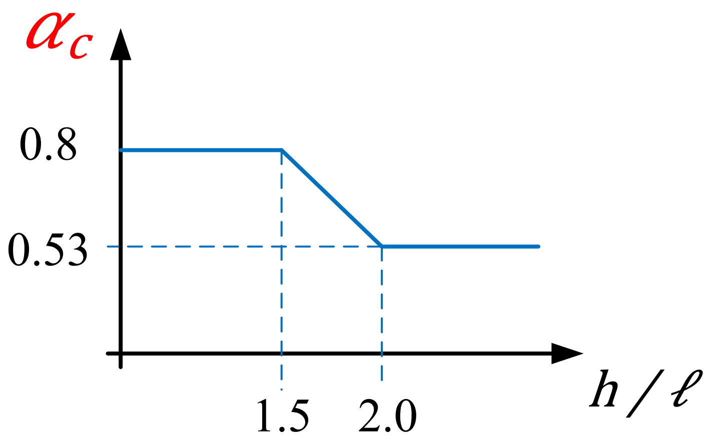
- 開孔造成垂直牆段與牆墩
- 其均由幾何不連續引致之撓曲臨界斷面所定義
- 垂直牆段按剪力牆設計
- 牆墩按柱設計

垂直牆段之淨高/垂直牆段之牆長 (h_w/ℓ_w)	垂直牆段之牆長/牆厚 (ℓ_w/b_w)		
	$(\ell_w/b_w) \le 2.5$	$2.5 < (\ell_w/b_w) \le 6.0$	$(\ell_w/b_w) > 6.0$
$h_w/\ell_w < 2.0$	牆	牆	牆
$h_w/\ell_w \ge 2.0$	牆墩宜符合柱設計之需求；參見第18.7.8.1節	牆墩宜符合柱設計需求或其替代條款；參見第18.7.8.1節	牆

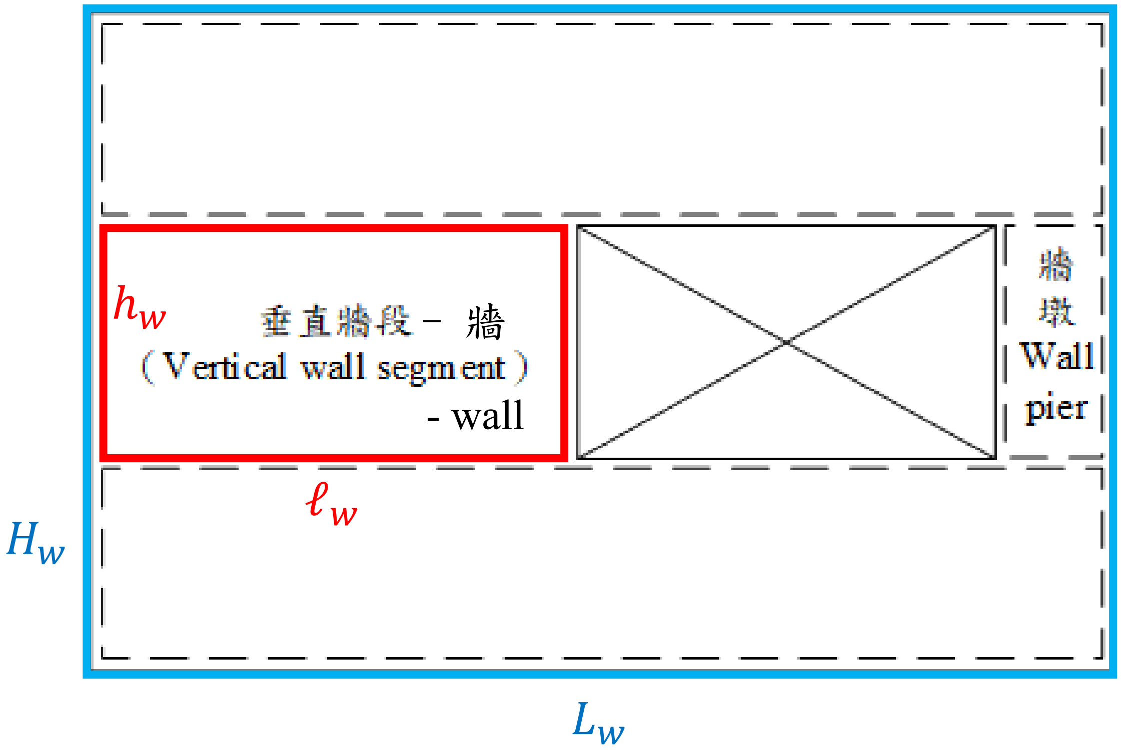
[1] h_w、ℓ_w、b_w分別為牆段中牆板之淨高、水平長度及厚度

垂直牆段之剪力強度 - 土木401-112

$$V_n = \left(\alpha_c \sqrt{f'_c} + \rho_t f_y\right) A_{cv}$$



$$\frac{h}{\ell} = \max\left(\frac{h_w}{\ell_w}, \frac{H_w}{L_w}\right)$$



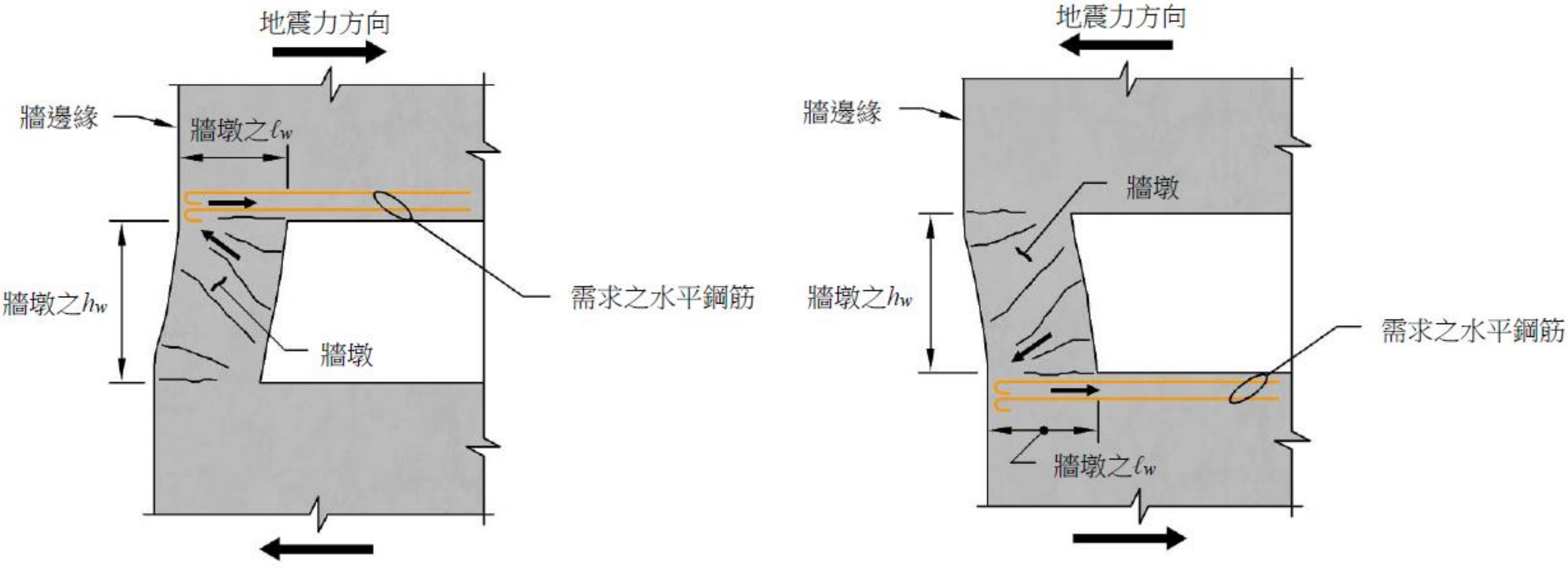
18.7.4.1 結構牆之V_n應不大於：

$$V_n = (\alpha_c \lambda \sqrt{f'_c} + \rho_t f_{yt}) A_{cv} \qquad (18.7.4.1)$$

對$h_w/\ell_w \le$ **1.5**者，係數 α_c之值取0.8 [0.25]；對$h_w/\ell_w \ge$ **2.0**者，係數 α_c之值取0.53 [0.17]；若h_w/ℓ_w介於1.5與2.0之間，則係數 α_c在0.8 [0.25]與0.53 [0.17]之間作線性變化。

18.7.4.2 第18.7.4.1節中用以計算牆段V_n所用之h_w/ℓ_w，應取全牆及所考慮牆段比值之較大者。

規範對牆墩要求配置水平鋼筋



- 對牆墩配置水平鋼筋，讓壓桿近牆邊緣處節點可滿足力平衡要求

垂直牆段亦應配置水平鋼筋

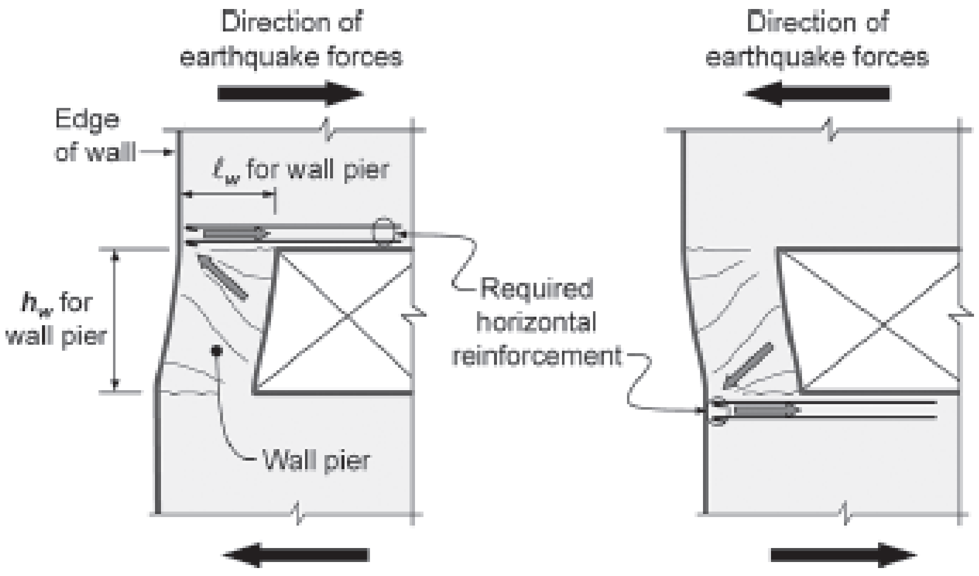
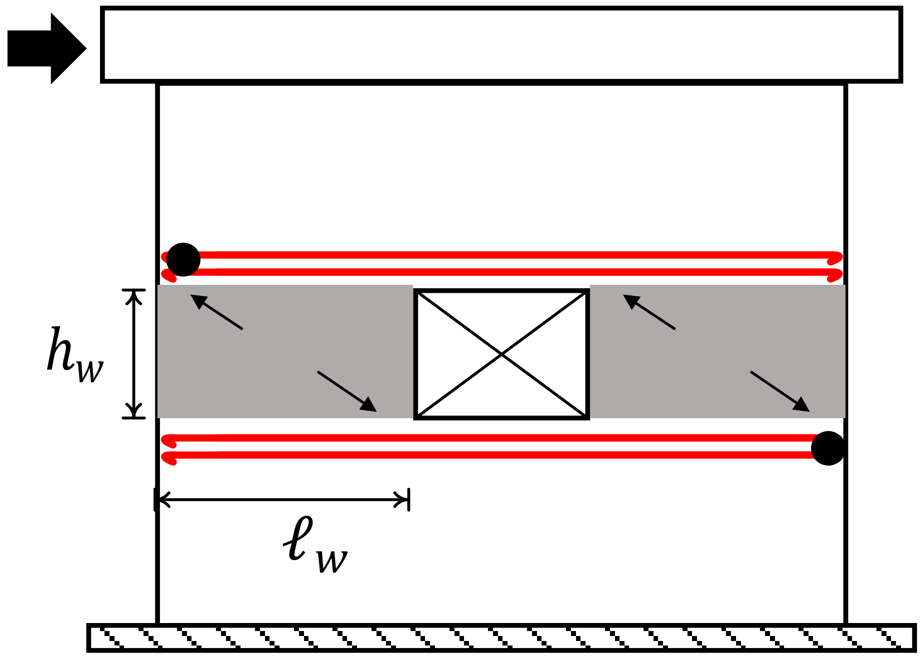


Fig R18.10.8—Required horizontal reinforcement in wall segments above and below wall piers at the edge of a wall.

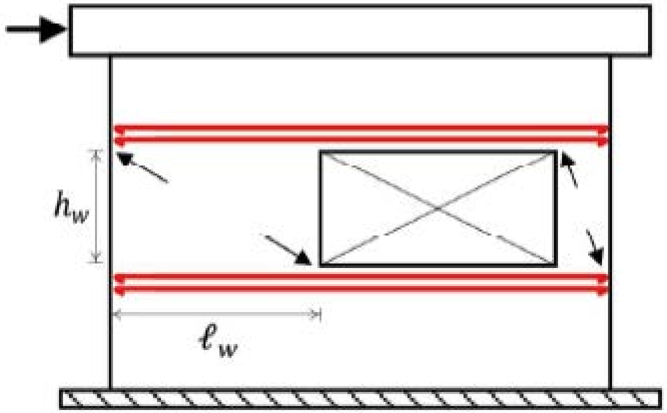


- 對緊鄰邊緣的牆墩需配置水平鋼筋，那垂直牆段是否也應比照辦理？
- 若垂直牆段未配置足夠的水平鋼筋，壓桿端部節點未達力平衡，那垂直牆段的力學高度$h_{w,m}$是否應做調整？

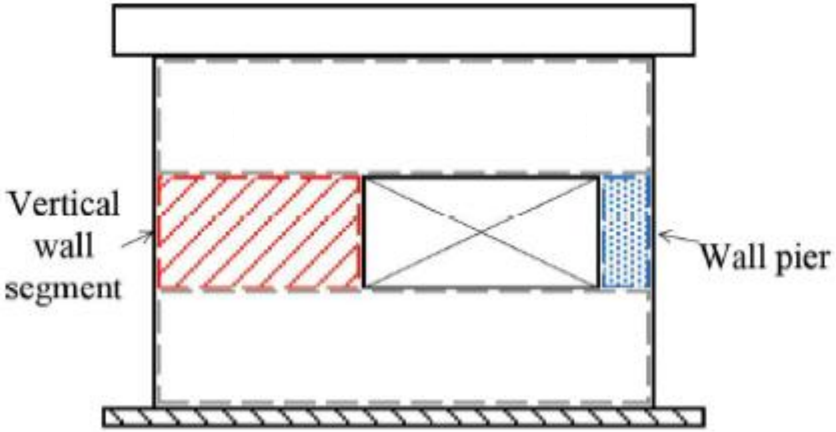
研究目的

水平剪力鋼筋對垂直牆段剪力強度之影響？

垂直牆段之力學高度($h_{w,m}$)是否僅由撓曲臨界斷面所定義？



(b)Special horizontal reinforcement at the edge of a wall



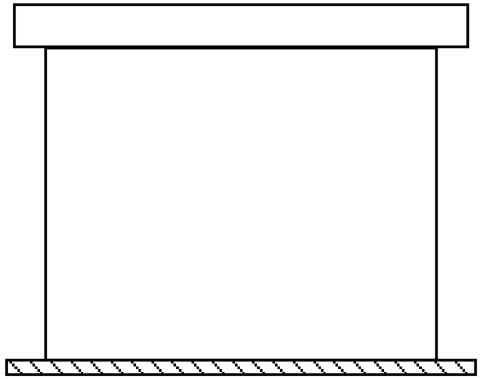
(a) Wall segments

垂直牆段之高度($h_{w,m}$)是否應考慮力平衡條件？ → 水平鋼筋 性質、數量

實驗規劃

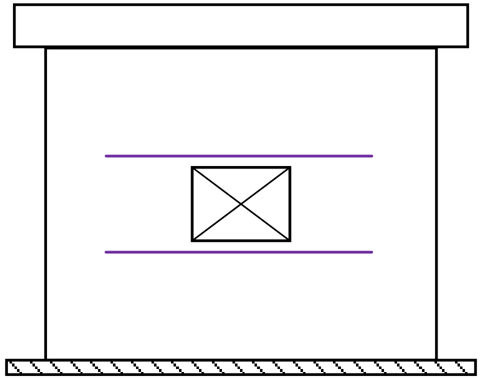
未開孔牆

SW



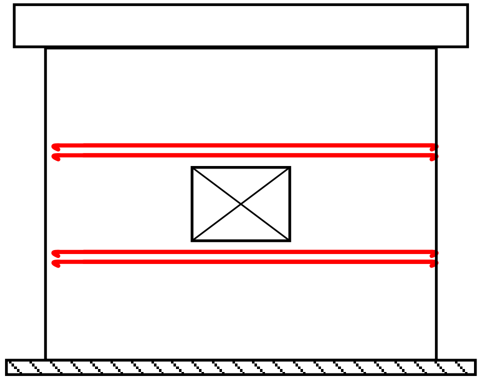
開孔牆

SWO



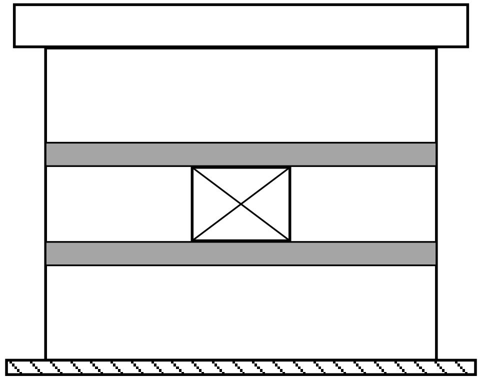
開孔牆+水平鋼筋

SWOS



開孔牆+CFRP

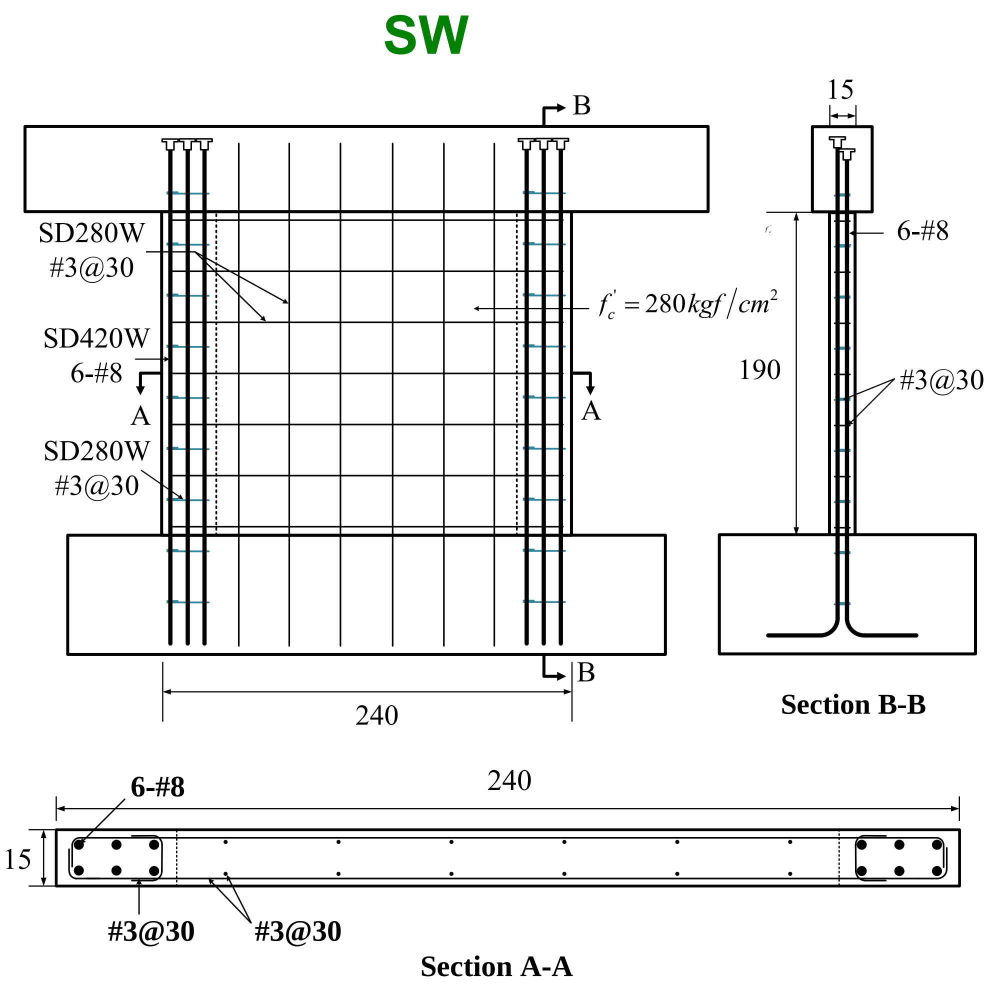
SWOF

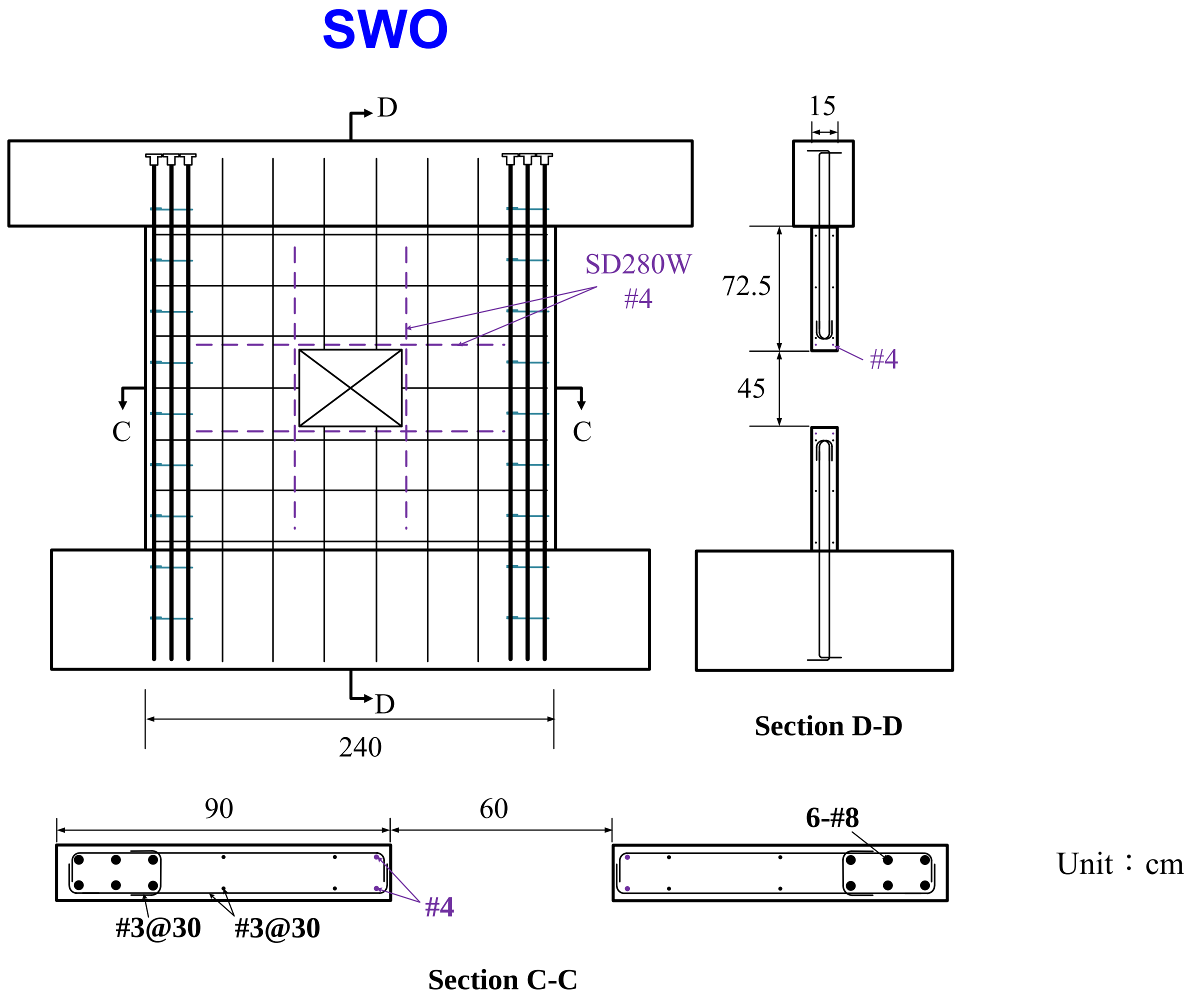


Specimen	f_c', MPa (ksi)	Boundary element: Longitudinal	Boundary element: Transverse	Web shear reinforcement	Strengthening of opening: Amount	Strengthening of opening: Strength, MPa (ksi)
SW	29.7 (4.3)	No. 8 f_y = 463 MPa (67.2 ksi)	No. 3 f_y = 348 MPa (50.5 ksi)	No. 3 f_y = 348 MPa (50.5 ksi)	—	—
SWO	30.9 (4.5)				Two No. 4 detailed around opening with extension of 600 mm from corner	f_y = 343 (49.7)
SWOS	31.9 (4.6)				Four No. 5 special horizontal reinforcement detailed above and below wall segment with 90-degree hook extended to end of wall	f_y = 462 (67.0)
SWOF	32.2 (4.7)				Wrapping around wall panel above and below wall segment with six plies CFRP and 250 mm in width	f_{fe} = 1094 (158.7)

Notes: Specimen code: SW is shear wall; O is with opening; S is steel strengthening; F is CFRP strengthening; 1 mm = 0.0394 in.

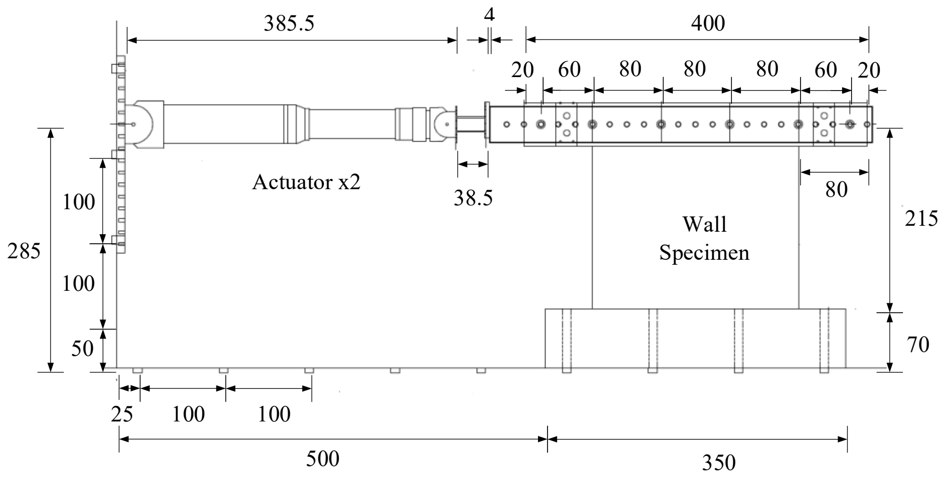
試體尺寸與鋼筋配置



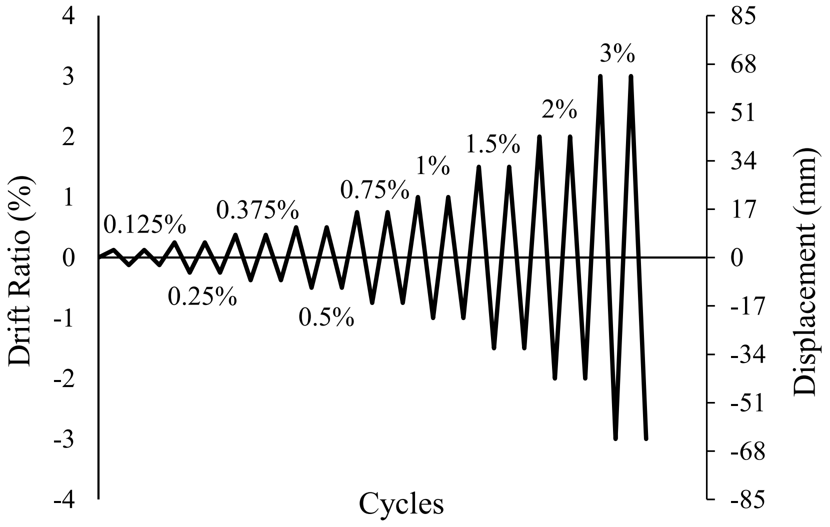


Yeh, R. L., Tseng, C. C., and Hwang, S. J. (2018). "Shear Strength of Reinforced Concrete Vertical Wall Segments under Seismic Loading," ACI Structural Journal, V. 115, No. 5, September, pp. 1485-1494.

測試布置

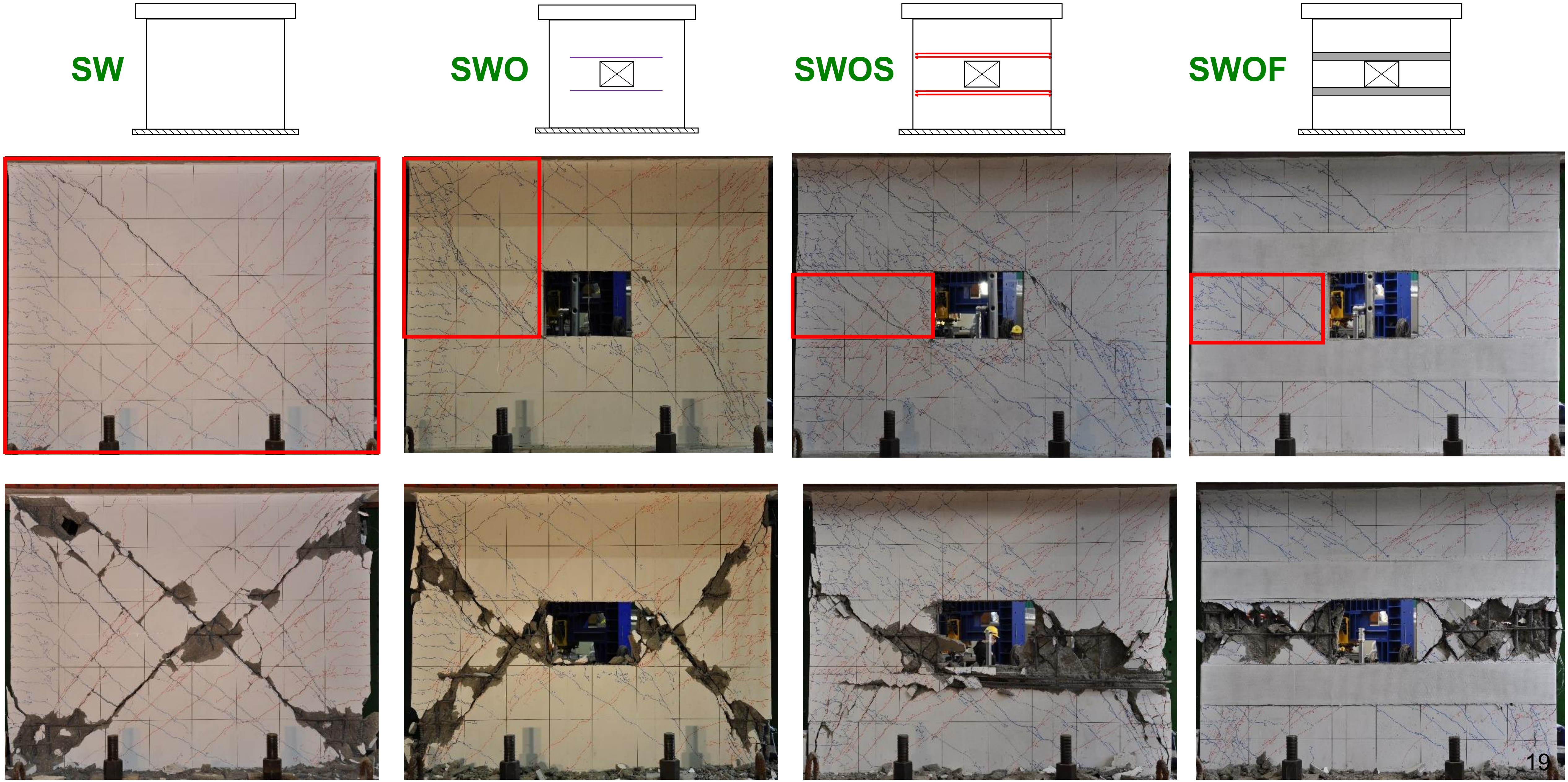


Unit：cm

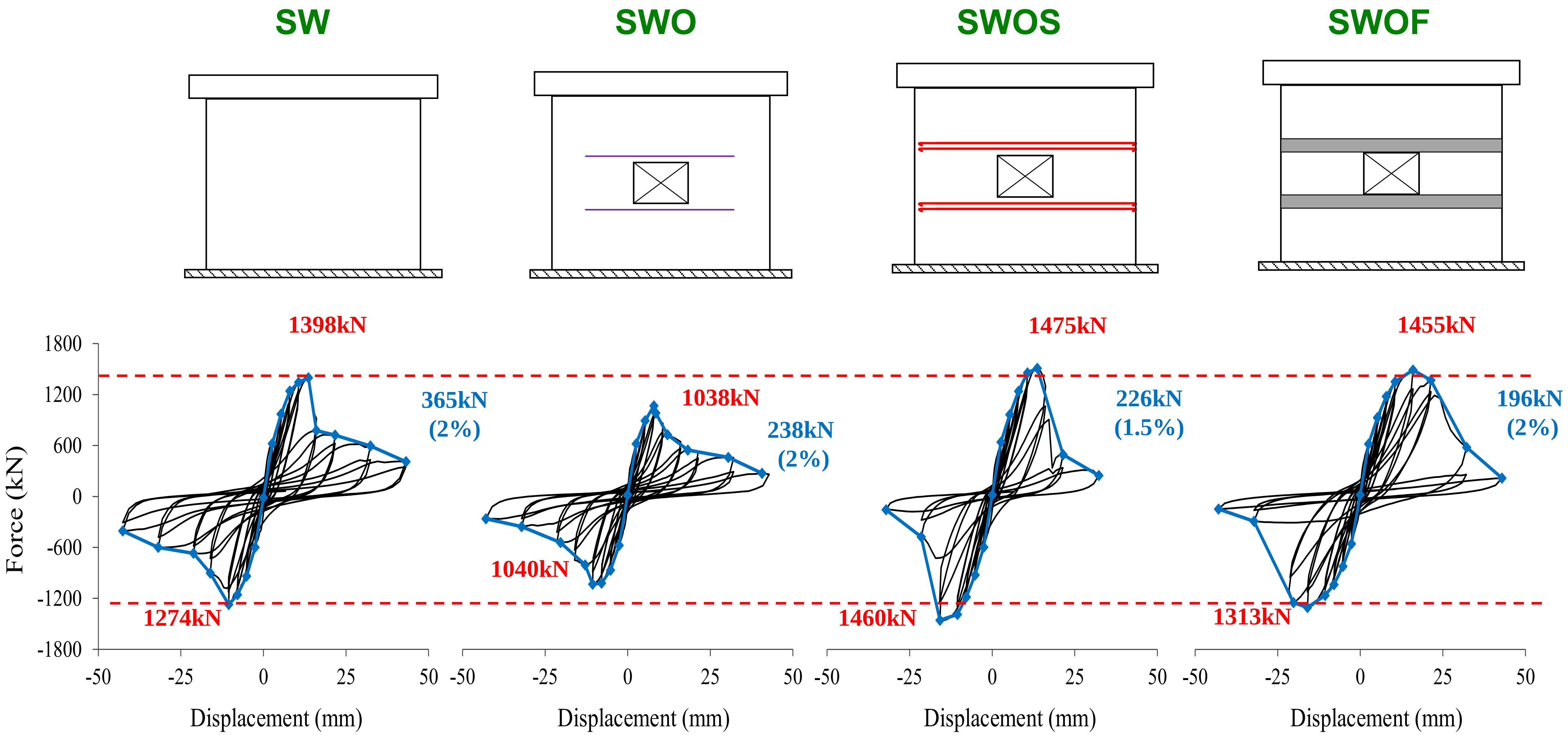


NCREE台北實驗室

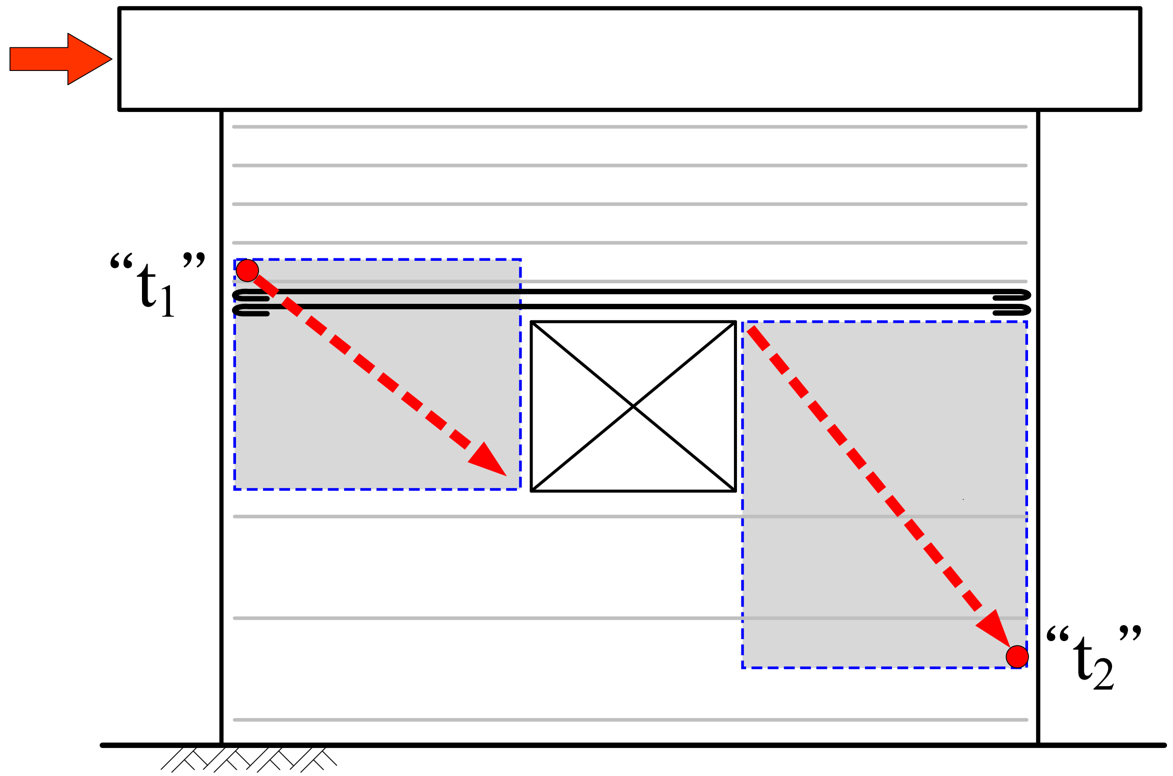
裂縫觀察



試驗結果


SW
SWO
SWOS
SWOF
1398kN
1475kN
1455kN
365kN
(2%)
1038kN
238kN
(2%)
226kN
(1.5%)
196kN
(2%)
1040kN
1274kN
1460kN
1313kN
1800
1200
600
0
-600
-1200
-1800
Force (kN)
-50
-25
0
25
50
Displacement (mm)
Displacement (mm)
Displacement (mm)
Displacement (mm)

實驗成果



- 水平鋼筋參與力平衡，降低垂直牆段的力學高度($h_{w,m}$)，提高剪力強度
- 垂直牆段的力學高度($h_{w,m}$)無法僅由幾何不連續面決定，應考量節點力平衡條件

Yeh, R. L., Tseng, C. C., and Hwang, S. J. (2018). "Shear Strength of Reinforced Concrete Vertical Wall Segments under Seismic Loading," ACI Structural Journal, V. 115, No. 5, September, pp. 1485-1494.

分析理論

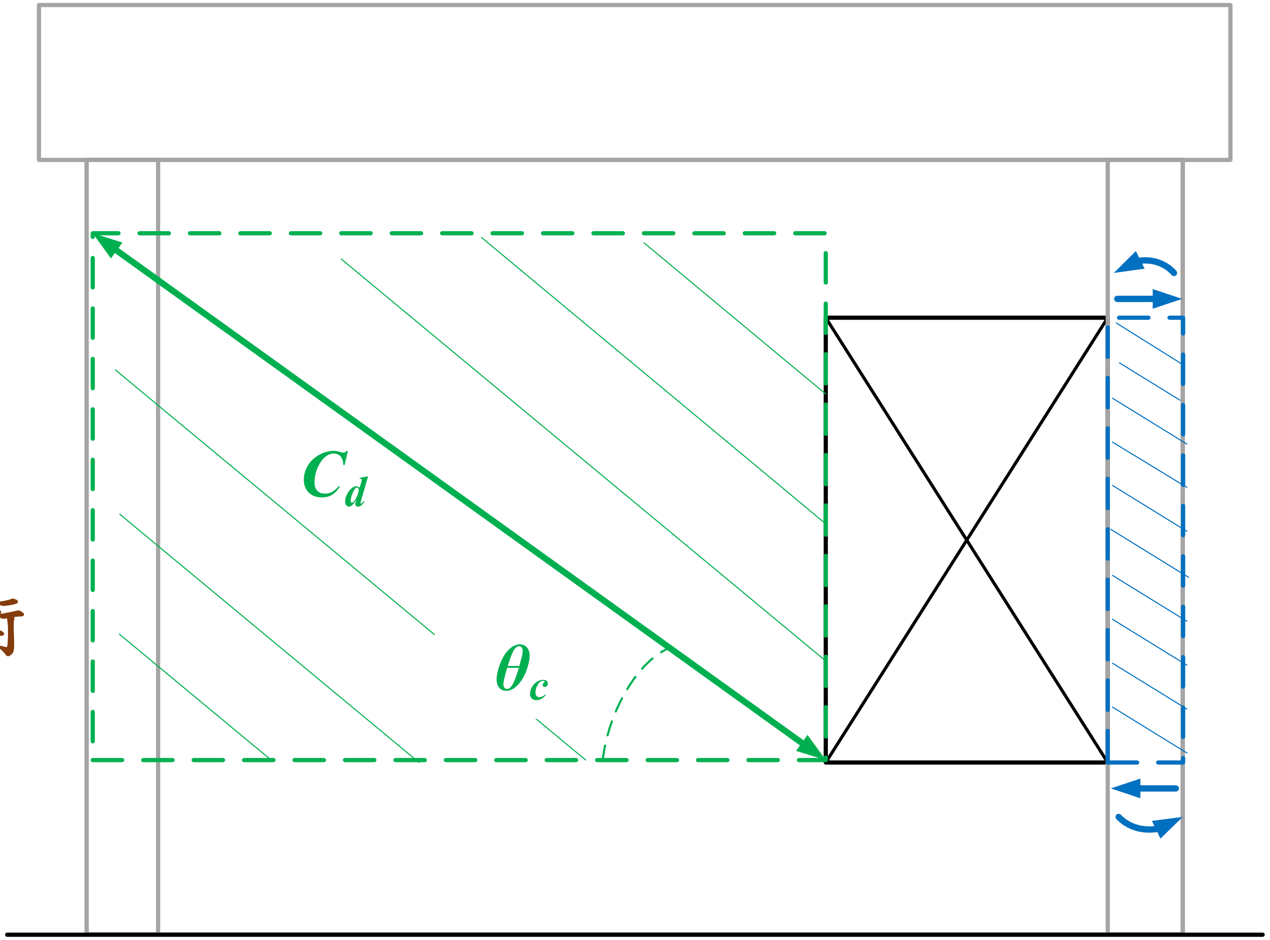
抗剪元素之分類與尺寸

剪力牆段與牆墩

剪力牆段：

剪力行為主控

壓桿節點力平衡

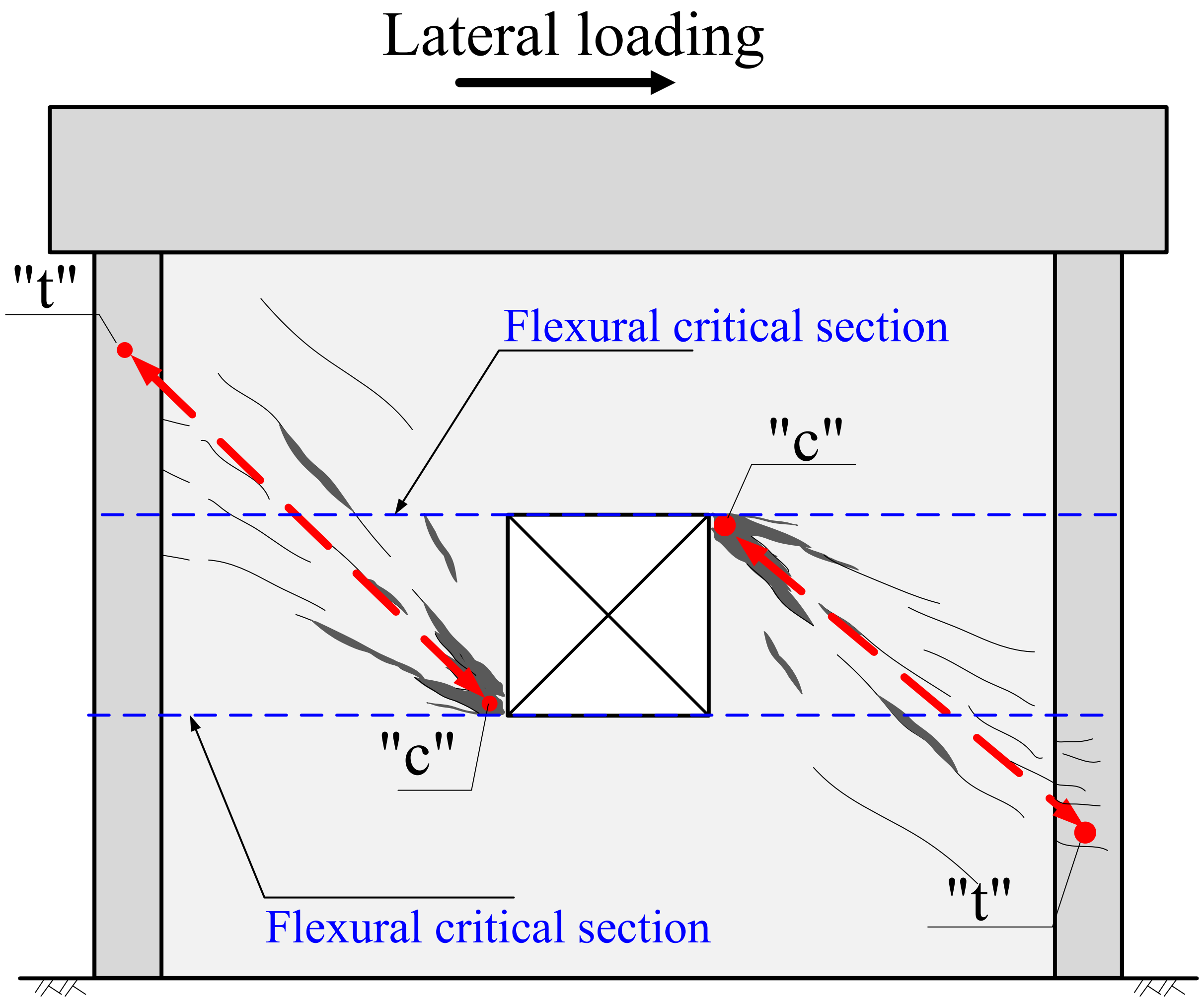


牆墩：

撓曲行為主控

撓曲臨界斷面

垂直牆段



- 近牆開孔處壓桿節點 “c”
- “c” 位於撓曲臨界斷面上，為混凝土擠碎處
- 近牆邊緣處壓桿節點 “t”
- “t” 藉節點力平衡決定

垂直牆段力學高度之選取

T_i : Interface shear capacity

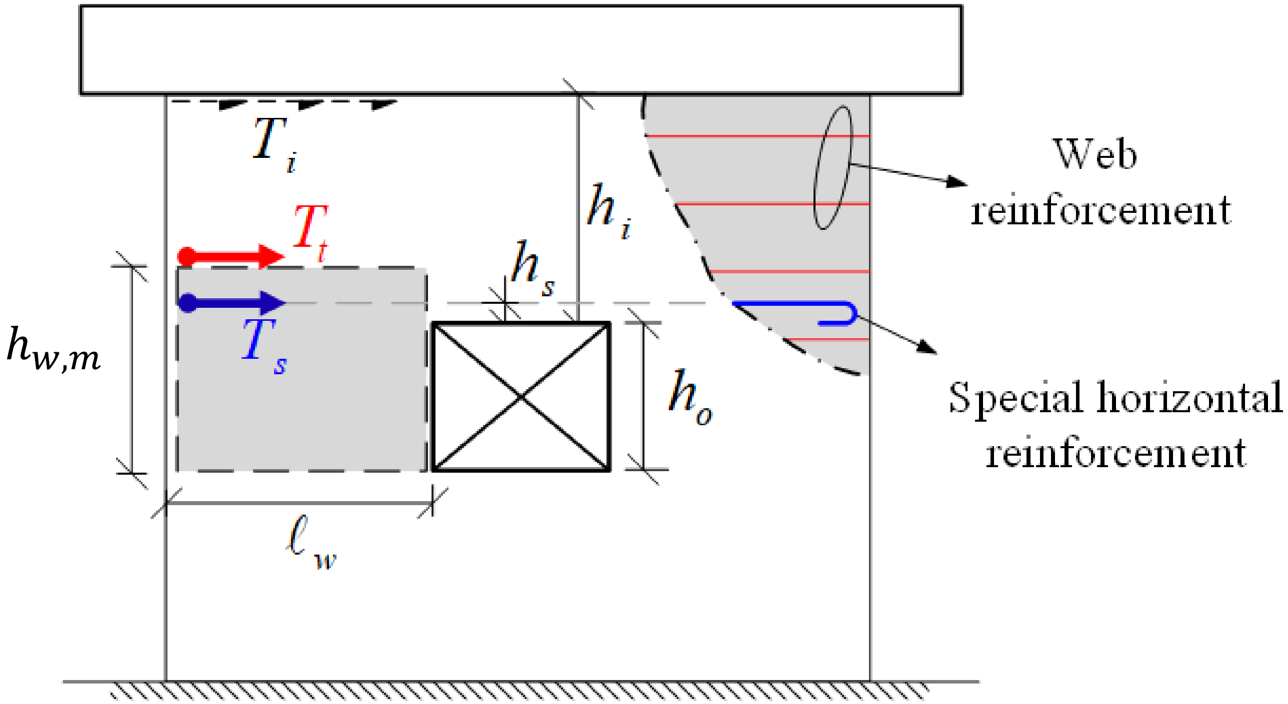
$$T_i = C_d \cos\theta - T_t - T_s$$

T_t : Shear tension force of wall panel

$$T_t = V_c + V_s = \left(0.17\sqrt{f_c'} + \rho_t f_{yt}\right) b_w d$$

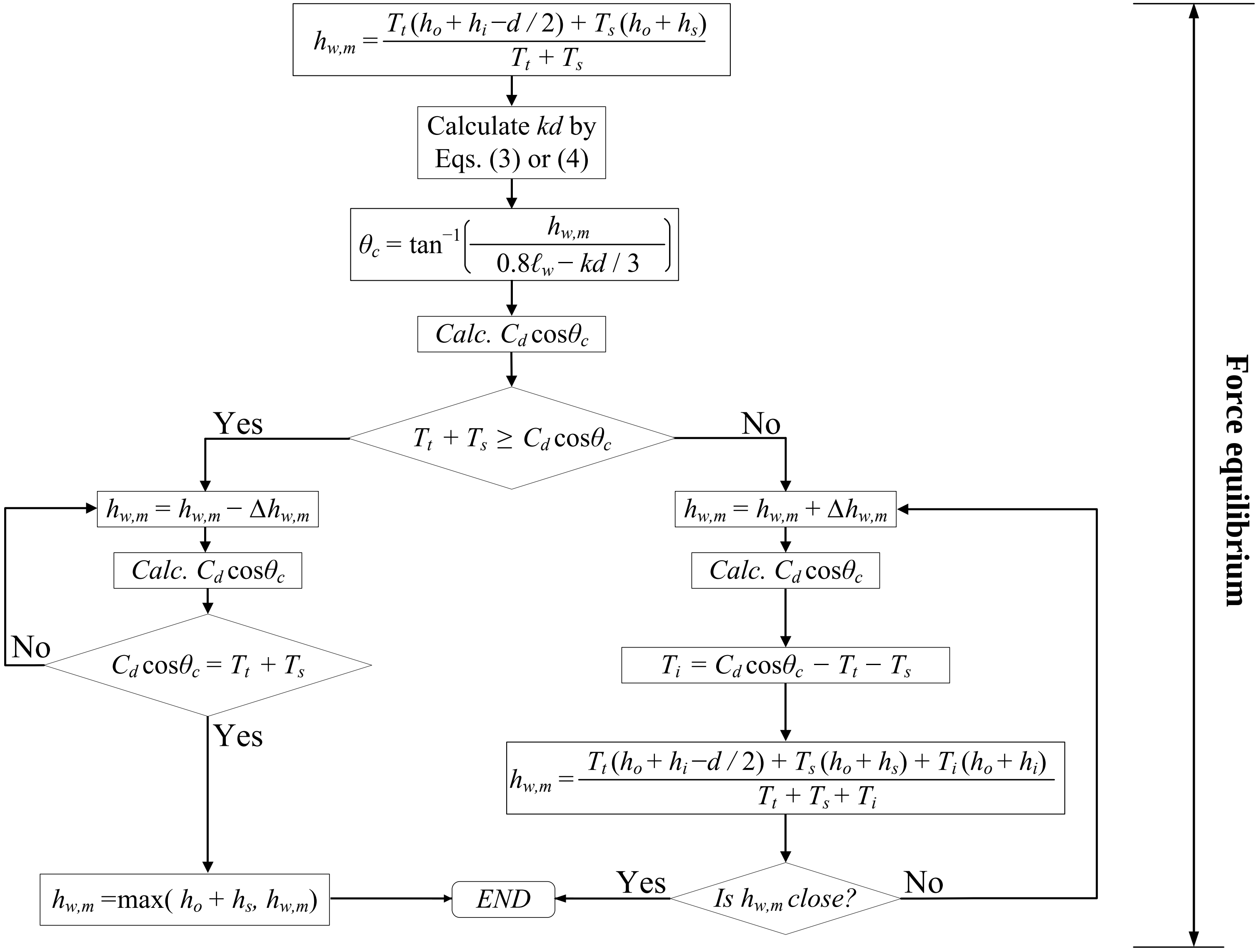
T_s : Force of special horizontal reinforcement

$$T_s = A_s \times f_{ys}$$



Yeh, R. L., Tseng, C. C., and Hwang, S. J. (2018). "Shear Strength of Reinforced Concrete Vertical Wall Segments under Seismic Loading," ACI Structural Journal, V. 115, No. 5, September, pp. 1485-1494.

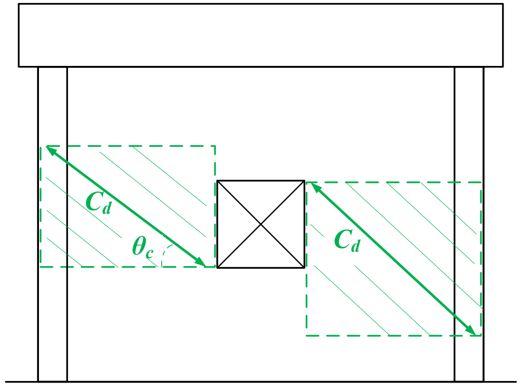
垂直牆段力學高度 $h_{w,m}$ 之計算流程



Yeh, R. L., Tseng, C. C., and Hwang, S. J. (2018). "Shear Strength of Reinforced Concrete Vertical Wall Segments under Seismic Loading," ACI Structural Journal, V. 115, No. 5, September, pp. 1485-1494.

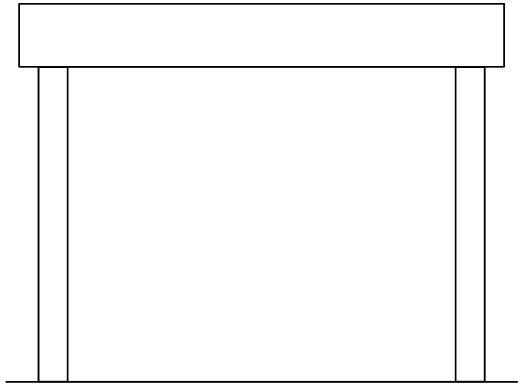
側力位移曲線

垂直牆段側力位移曲線之建立



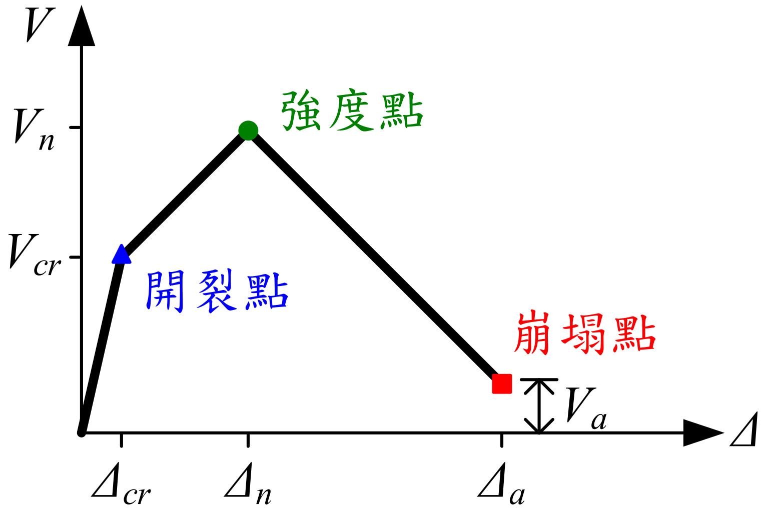
垂直牆段

力學行為
相似

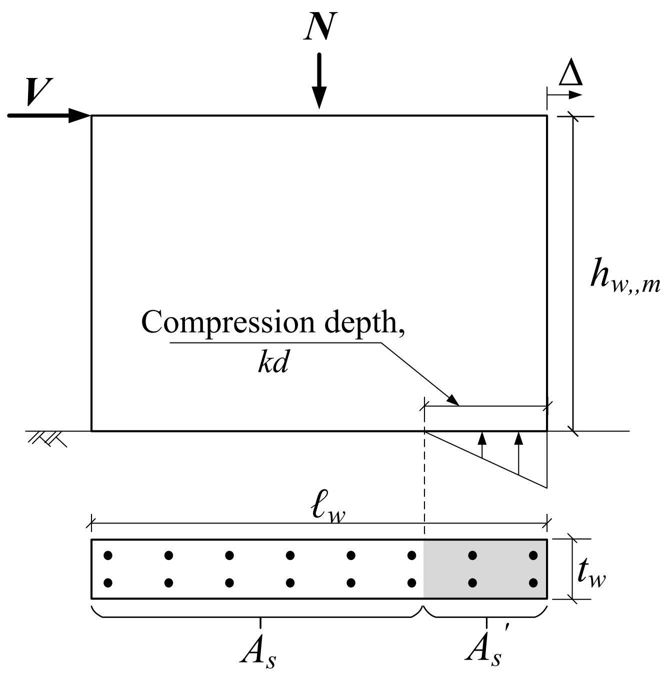


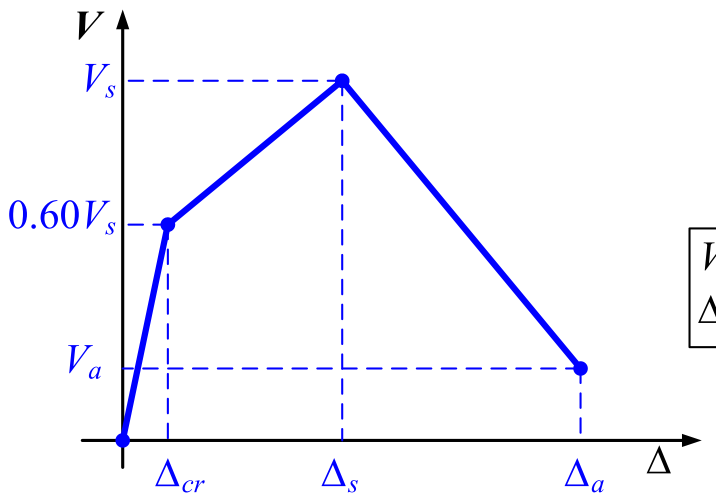
低矮型剪力牆

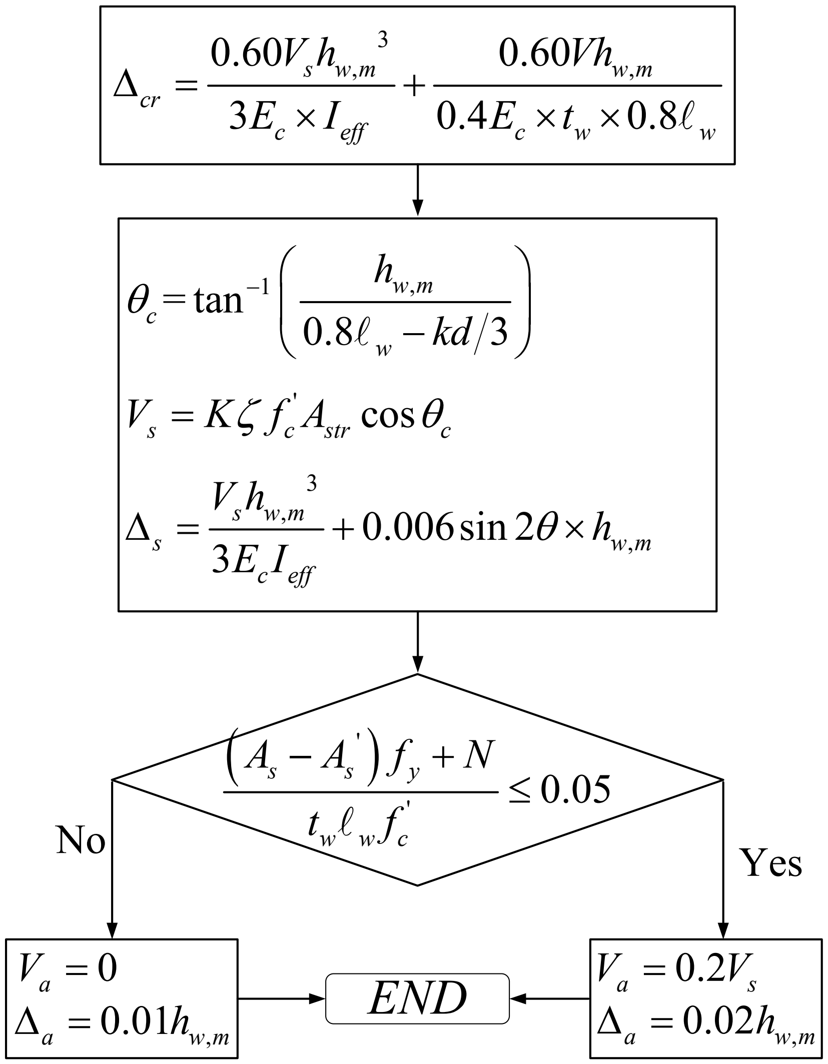
側力位移
曲線



計算流程



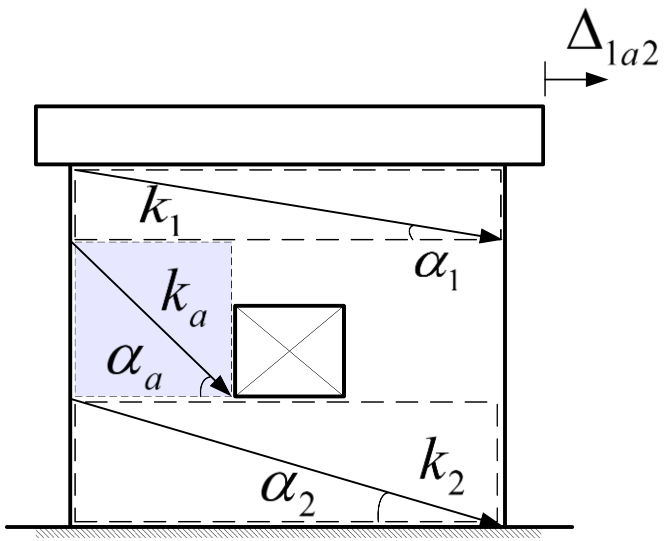


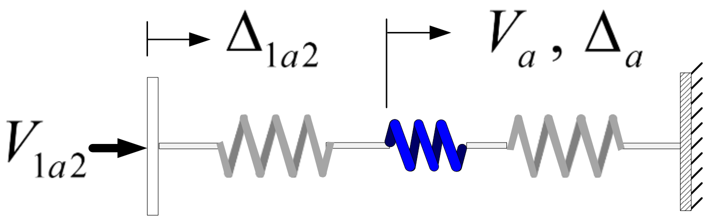


Weng, P. W., Li, Y. A., Tu, Y. S., and Hwang, S. J., (2017) “Prediction of Lateral Load Displacement Curves for Reinforced Concrete Squat Walls Failed in Shear,” Journal of Structural Engineering, ASCE, 143(10), DOI: 10.1061/(ASCE)ST.1943-541X.0001872, 04017141.

傳力路徑之勁度

傳力路徑之彈簧串聯模擬





Load path 1a2 of left segment

剪力元素之勁度?

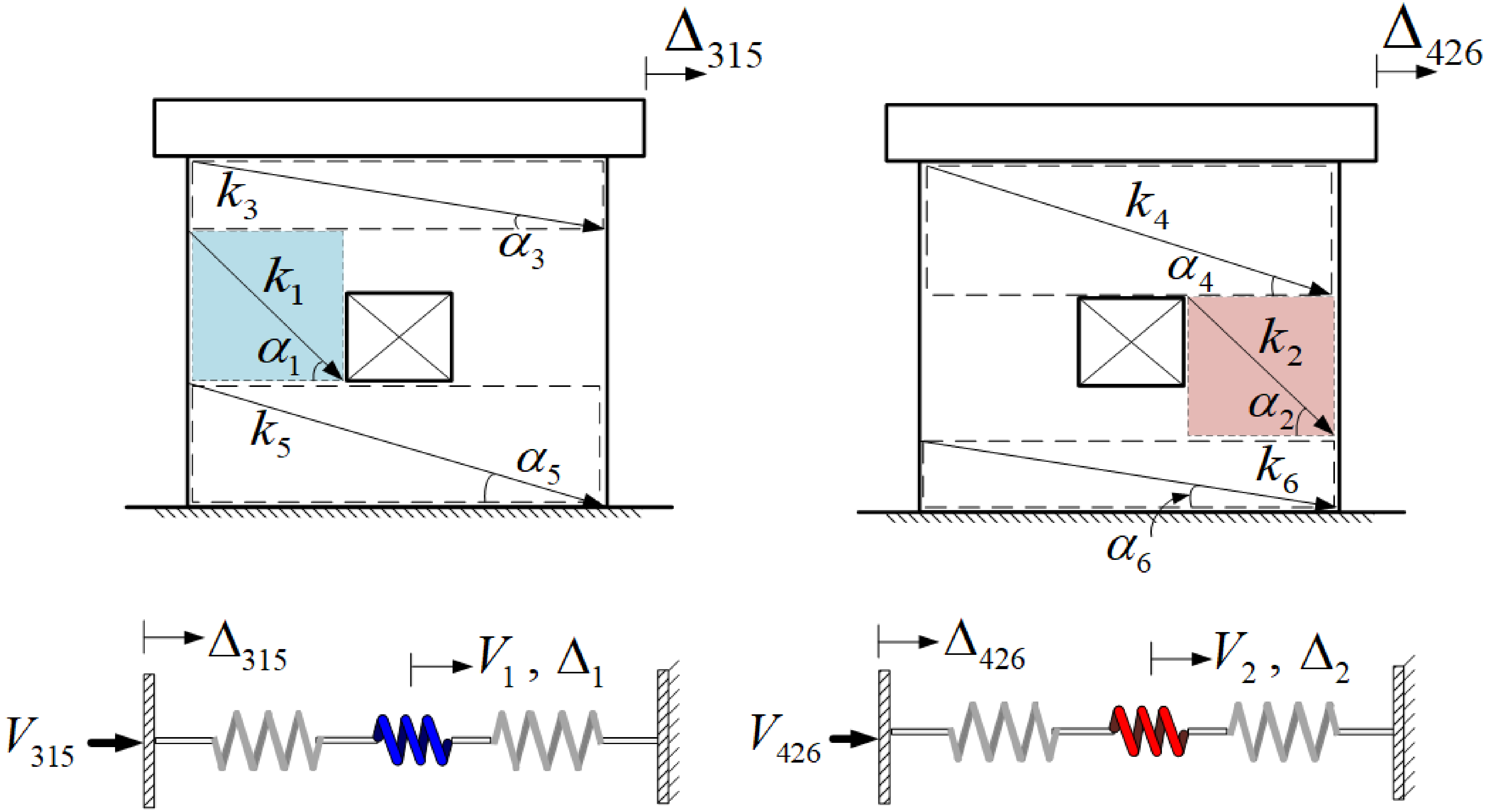
$$k_1, k_a, \ldots..$$

垂直牆段側位移與傳力路徑側位移之關係?

$$\Delta_a \rightarrow \Delta_{1a2}$$

Tsai, R. J., Y. C. Hsu, and S. J. Hwang. 2021. "Prediction of lateral load displacement curve of reinforced concrete walls with openings under shear failure." ACI Struct. J. 118 (5): 275–284.

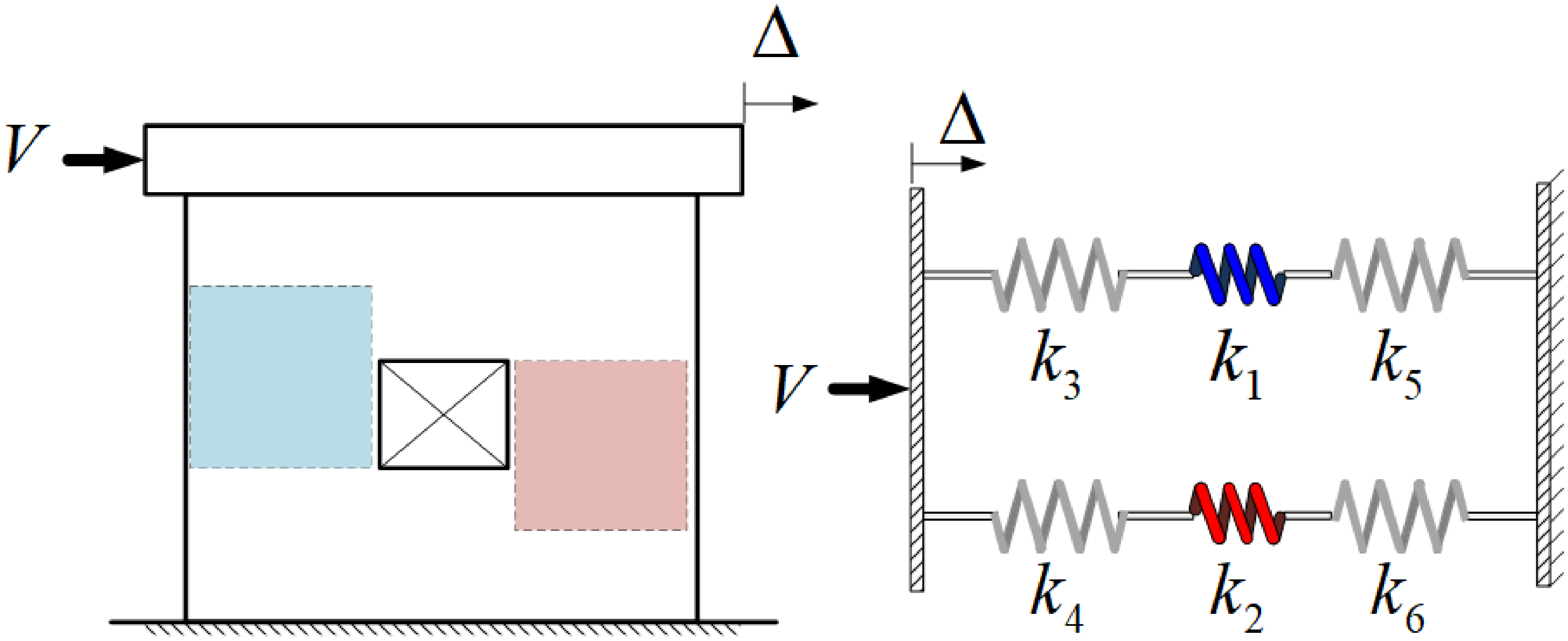
受剪牆版分解為傳力路徑之彈簧串聯系統



(a) Load path 315 of left segment

(b) Load path 426 of right segment

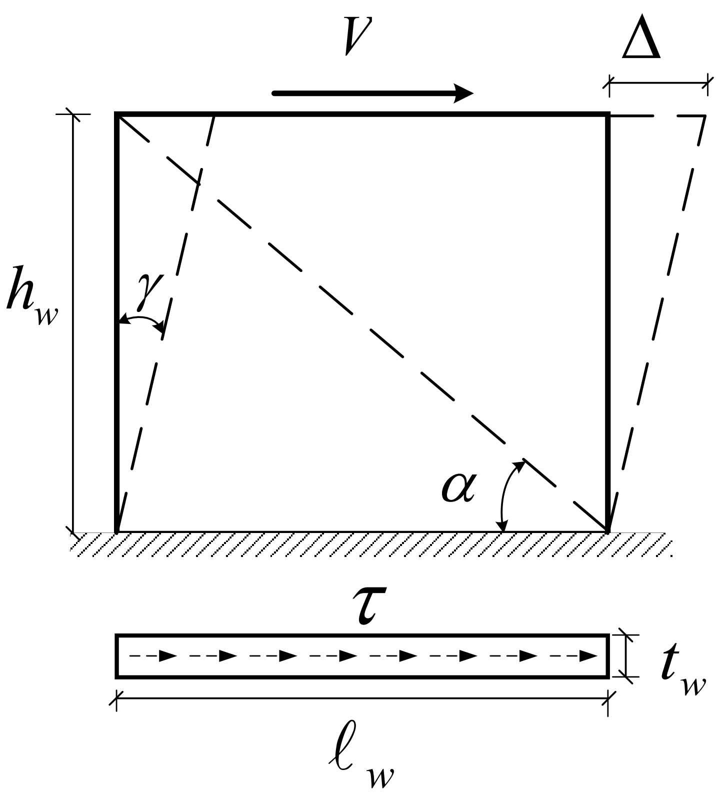
受剪牆版模擬為傳力路徑之彈簧串、並聯系統



(c) System of series and parallel springs

Tsai, R. J., Y. C. Hsu, and S. J. Hwang. 2021. "Prediction of lateral load displacement curve of reinforced concrete walls with openings under shear failure." ACI Struct. J. 118 (5): 275–284.

剪力元素之剪力勁度



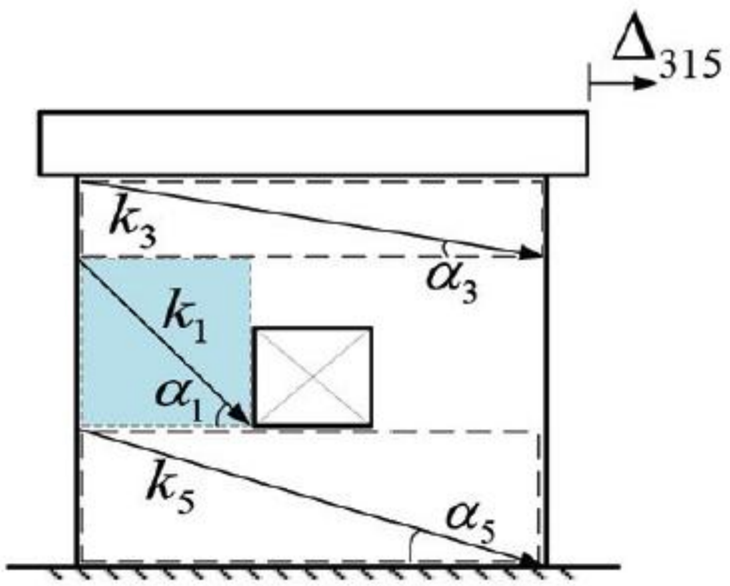
$$V = \tau \times \ell_w t_w = G\gamma \times \ell_w t_w$$

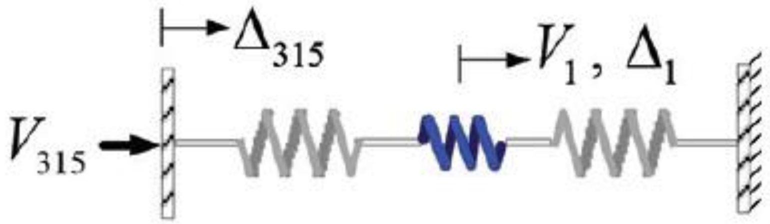
$$\Delta = \gamma \times h_w$$

$$k = \frac{V}{\Delta} = \frac{G\gamma \times \ell_w t_w}{\gamma \times h_w} = G t_w \frac{\ell_w}{h_w} = \frac{G t_w}{\tan \alpha}$$

Tseng, C. C., Hwang, S. J., and Lu, W. Y. (2017). "Shear Strength Prediction of Reinforced Concrete Deep Beams with Web Openings," ACI Structural Journal, V. 114, No. 6, November-December, pp. 1569-1579.

傳力路徑 315 之勁度





Springs in series

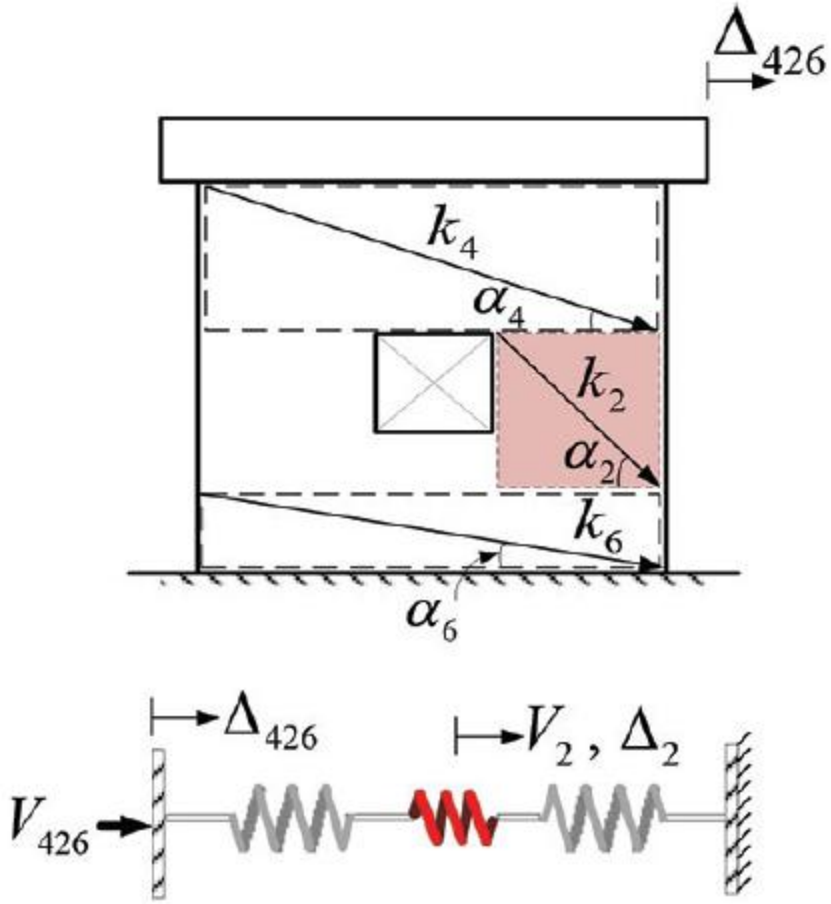
$$V_{315} = V_3 = V_1 = V_5$$

$$\Delta_{315} = \Delta_3 + \Delta_1 + \Delta_5$$

$$\frac{V_{315}}{k_{315}} = \frac{V_3}{k_3} + \frac{V_1}{k_1} + \frac{V_5}{k_5}$$

$$\frac{1}{k_{315}} = \frac{1}{k_3} + \frac{1}{k_1} + \frac{1}{k_5} = \frac{\tan\alpha_3}{Gt_w} + \frac{\tan\alpha_1}{Gt_w} + \frac{\tan\alpha_5}{Gt_w}$$

傳力路徑 426 之勁度



Springs in series

$$V_{426} = V_4 = V_2 = V_6$$

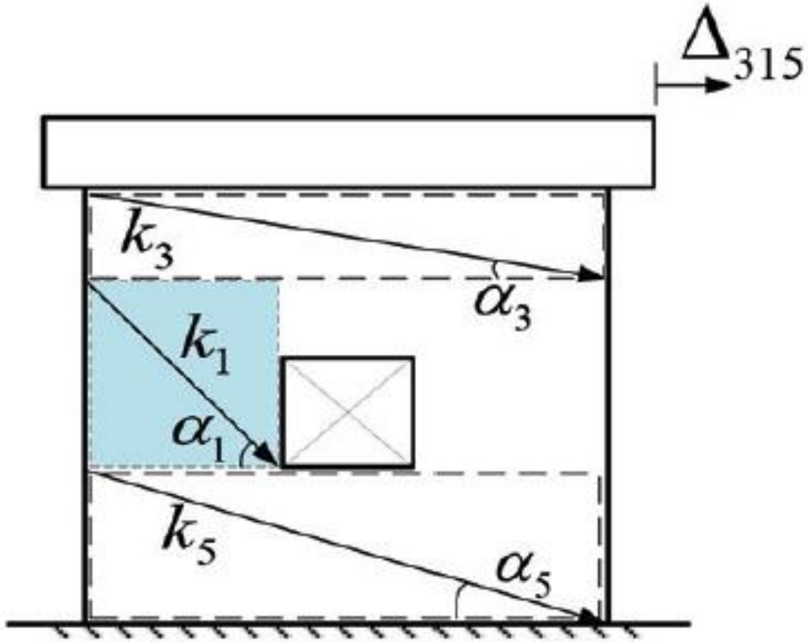
$$\Delta_{426} = \Delta_4 + \Delta_2 + \Delta_6$$

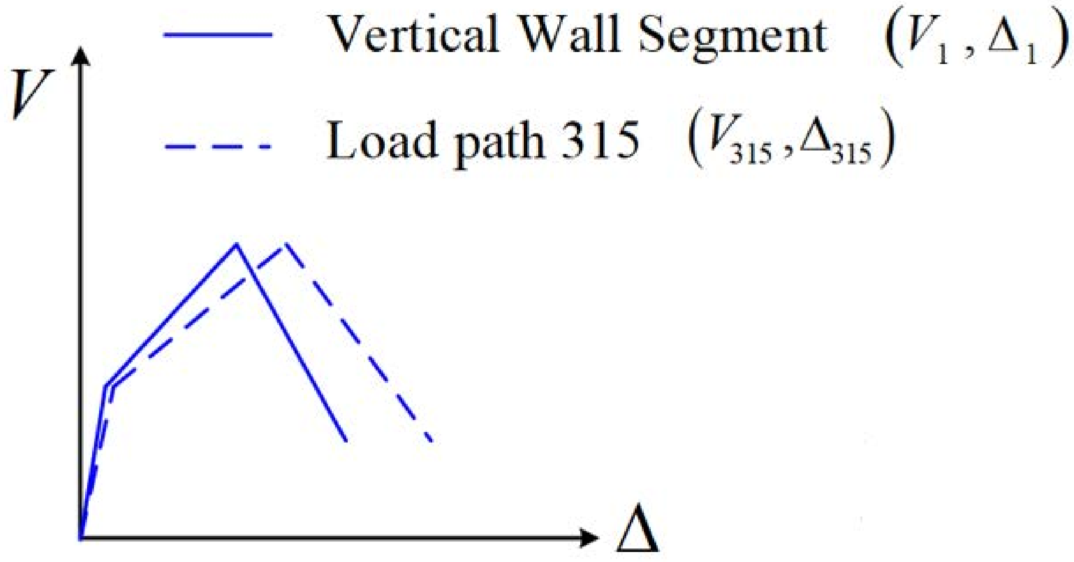
$$\frac{V_{426}}{k_{426}} = \frac{V_4}{k_4} + \frac{V_2}{k_2} + \frac{V_6}{k_6}$$

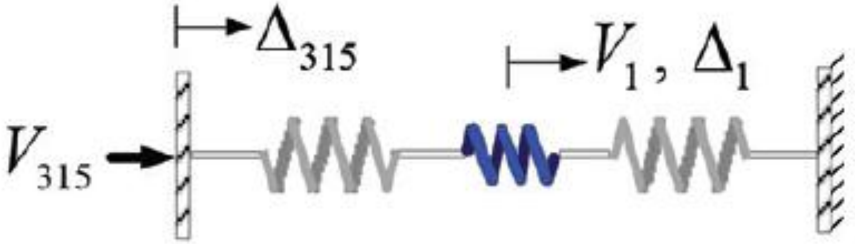
$$\frac{1}{k_{426}} = \frac{1}{k_4} + \frac{1}{k_2} + \frac{1}{k_6} = \frac{\tan\alpha_4}{Gt_w} + \frac{\tan\alpha_2}{Gt_w} + \frac{\tan\alpha_6}{Gt_w}$$

開孔牆傳力路徑之整合

垂直牆段之位移調整為傳力路徑之位移



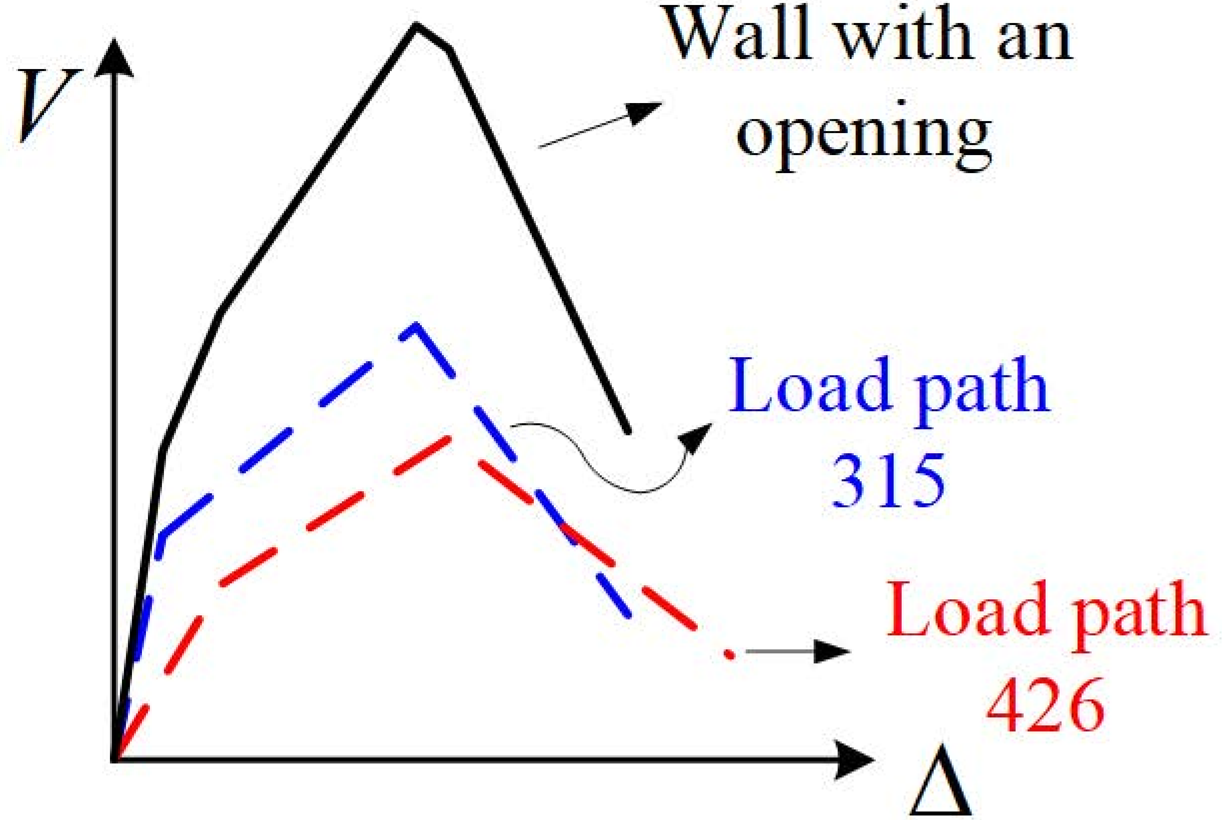


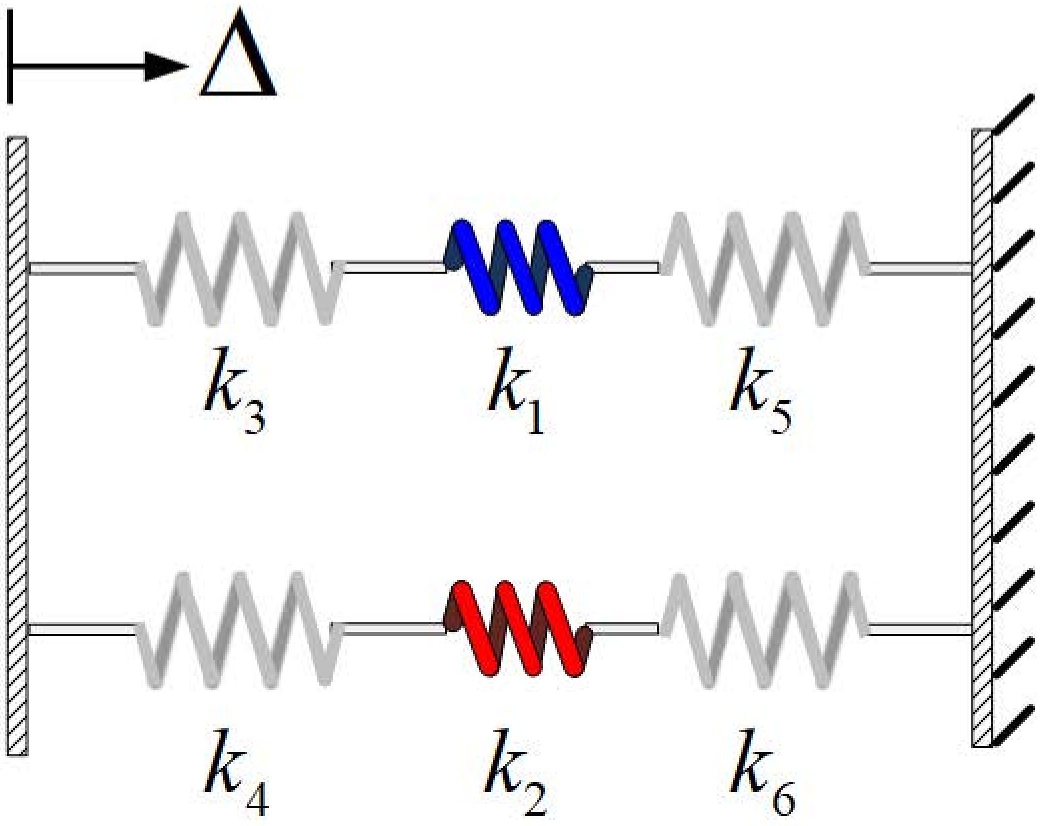


$$V_{315} = V_1 = K_{315} \times \Delta_{315} = K_1 \times \Delta_1$$

$$\Delta_{315} = \frac{K_1}{K_{315}} \times \Delta_1 = \frac{\tan\alpha_3 + \tan\alpha_1 + \tan\alpha_5}{\tan\alpha_1} \times \Delta_1$$

傳力路徑彈簧並聯系統之整合

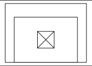
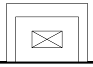
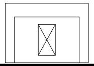
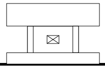
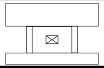
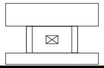
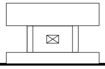
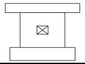
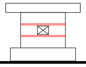


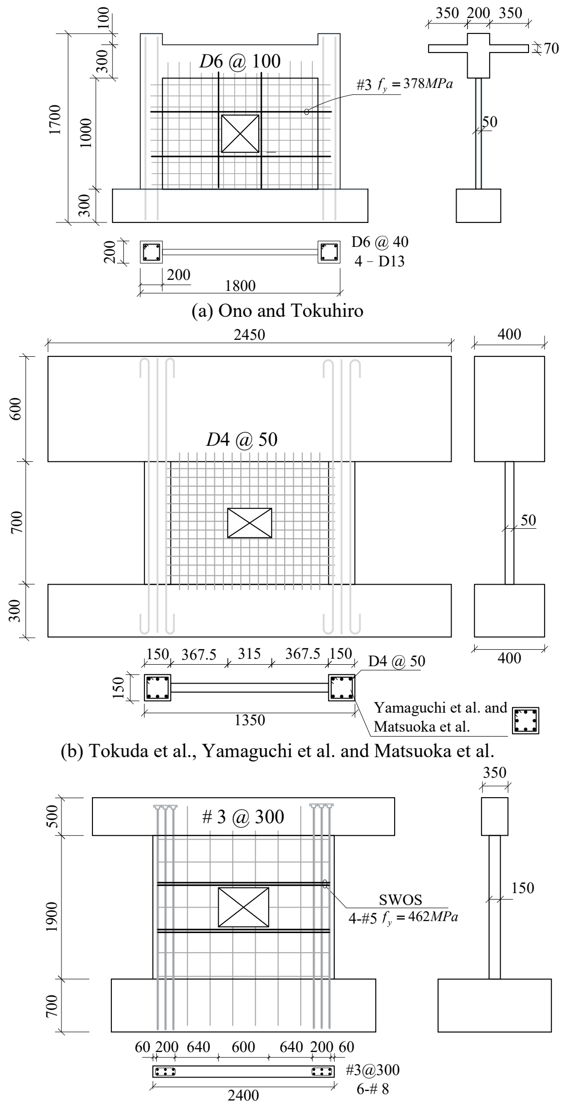


Tsai, R. J., Y. C. Hsu, and S. J. Hwang. 2021. “Prediction of lateral load displacement curve of reinforced concrete walls with openings under shear failure.” ACI Struct. J. 118 (5): 275–284.

單窗開孔牆之驗證

單窗開孔牆之實驗

Specimen	Figure	f_c' [MPa]	$f_{y,web}$ [MPa]	$\frac{N}{A_w f_c'}$
Ono and Tokuhiro (1992)				
FW5-0.273-R-C		27.0	380	0.16
FW5-0.367-S-C		25.6	380	0.17
FW5-0.367-L-C		27.9	380	0.16
Tokuda et al. (2000)				
S1W5-0.28-0.01		30.9	160	0.11
Yamaguchi et al. (2001)				
S1W5-0.28-0.014		30.9	178	0.11
Matsuoka et al. (2003)				
FS1W5-0.28C-2.5		31.7	160	0.10
S1W5-0.28C-2.5		26.6	160	0.12
Yeh et al. (2018)				
SWO		30.9	348	0
SWOS		31.9	348	0



(a) Ono and Tokuhiro

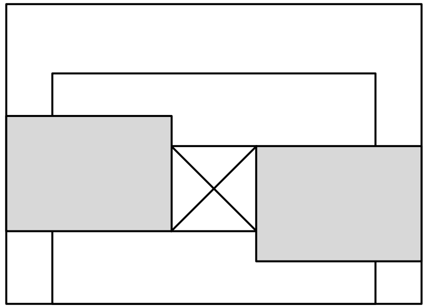
(b) Tokuda et al., Yamaguchi et al. and Matsuoka et al.

(c) Yeh et al.

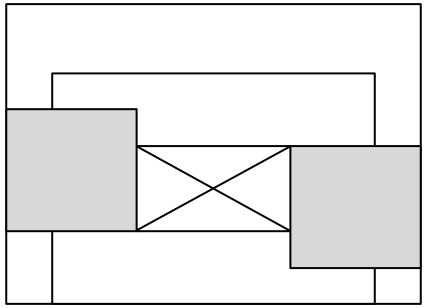
Tsai, R. J., Y. C. Hsu, and S. J. Hwang. 2021. "Prediction of lateral load displacement curve of reinforced concrete walls with openings under shear failure." ACI Struct. J. 118 (5): 275–284.

剪力元素之決定

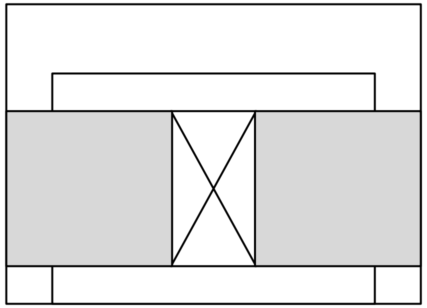
FW5-0.273-R-C



FW5-0.367-S-C

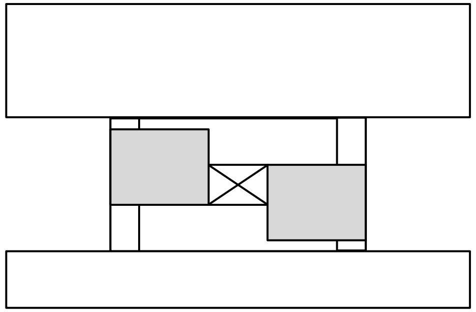


FW5-0.367-L-C

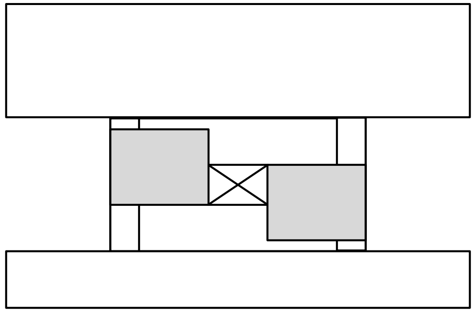


Ono and Tokuhiro (1992)

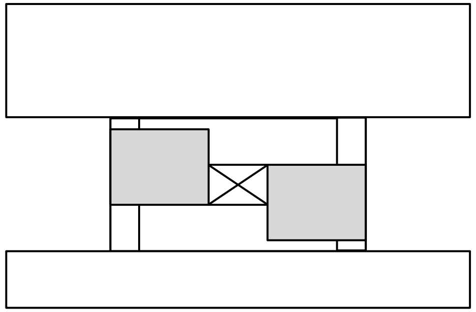
S1W5-0.28-0.01



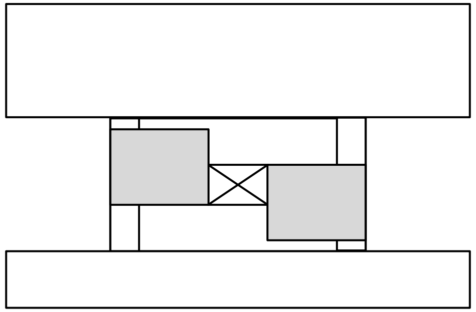
S1W5-0.28-0.014



FS1W5-0.28C-2.5-0.014

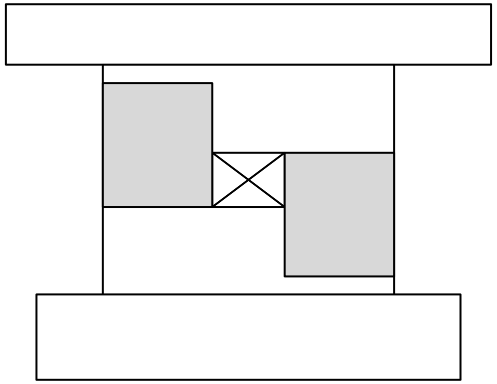


S1W5-0.28C-2.5-0.014

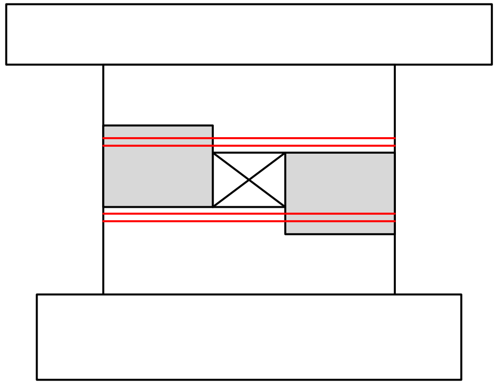


Tokuda et al. (2000), Yamaguchi et al. (2001), Matsuoka et al. (2003)

SWO

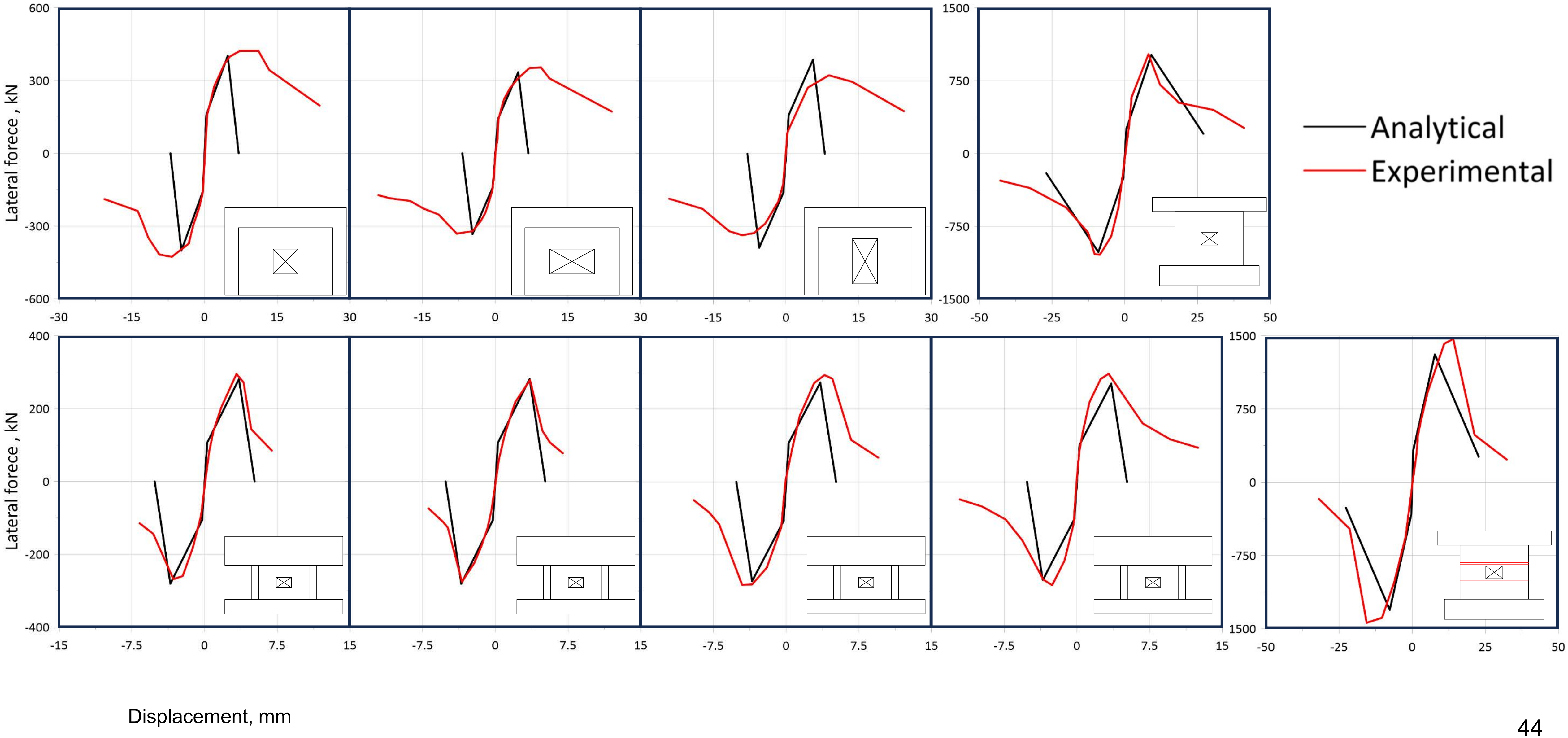


SWOS



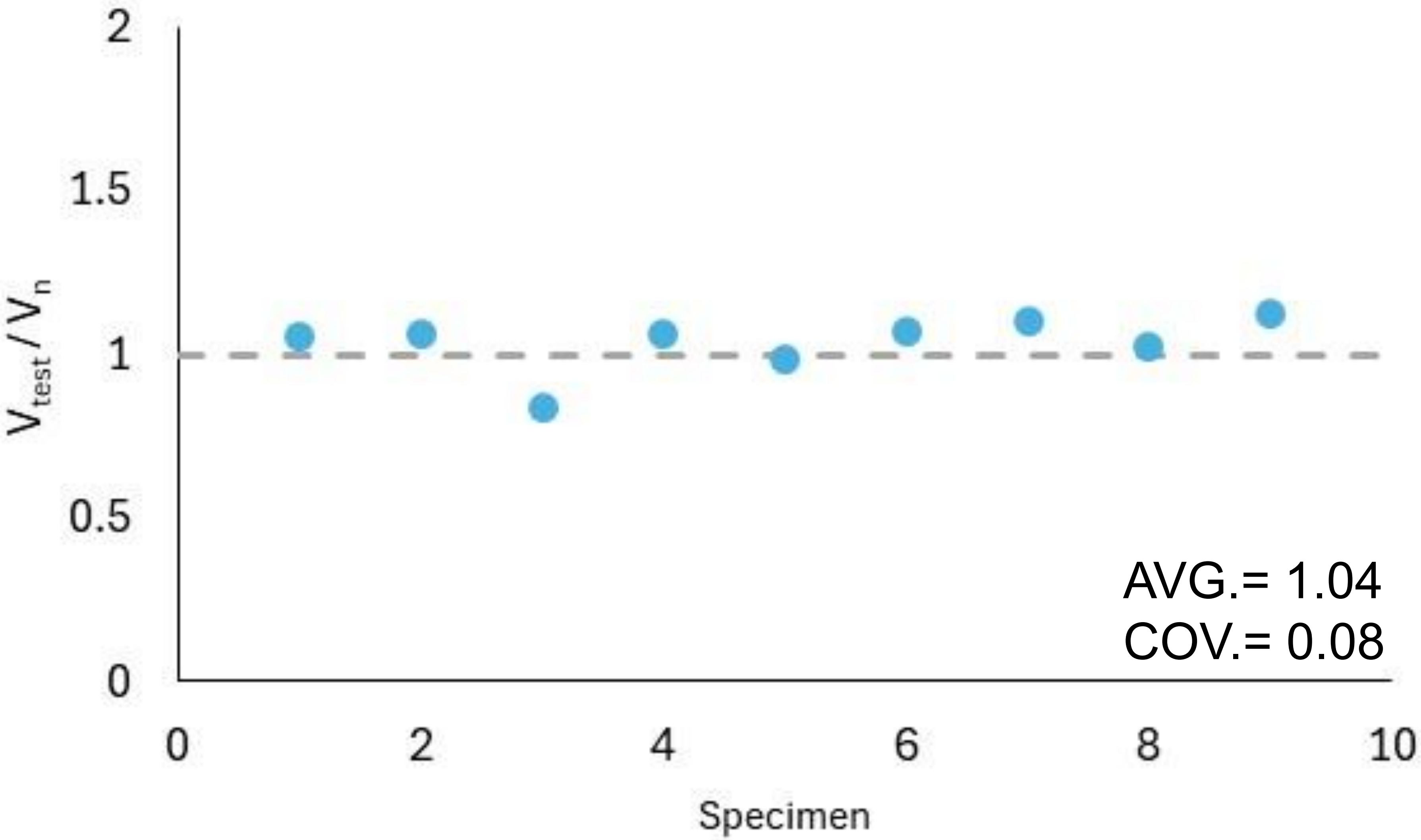
Yeh et al. (2018)

側力位移曲線之驗證

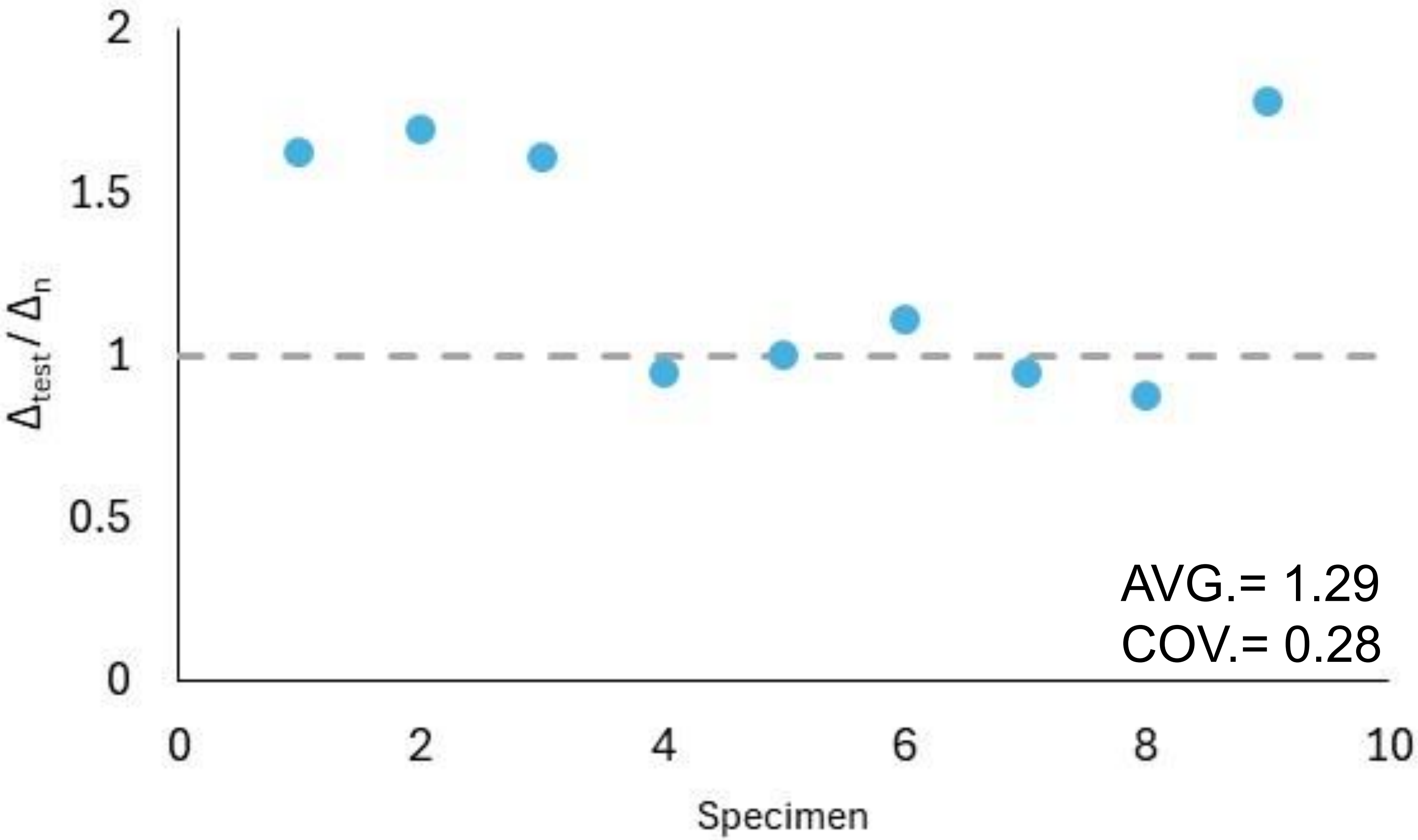


Lateral forece , kN
Lateral force , kN
Displacement, mm
Analytical
Experimental
600
300
0
-300
-600
-30
-15
15
30
1500
750
-750
-1500
-50
-25
25
50
400
200
-200
-400
-7.5
7.5

剪力強度之驗證



強度點位移之驗證

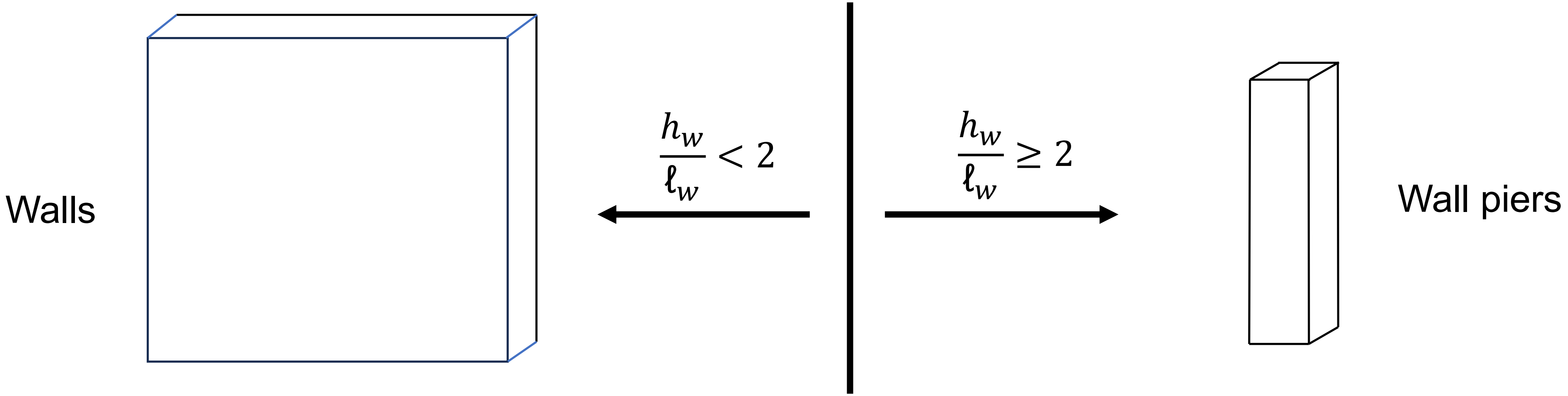


多重開孔牆之模擬與驗證

抗剪元素之分類與尺寸

- 垂直牆段
- 牆墩
- 柱

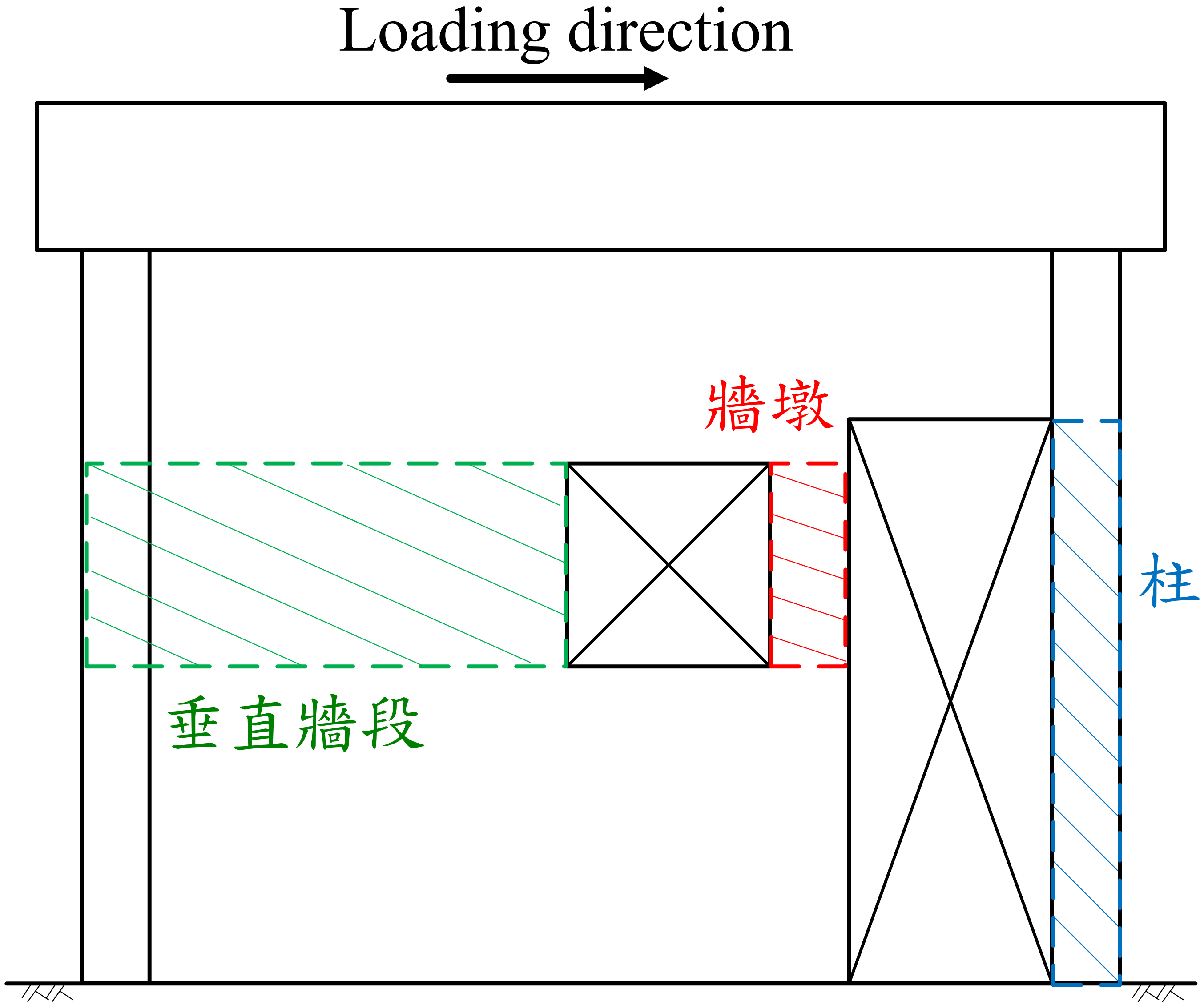
垂直牆段和牆墩之分類



	The ratio between wall length and thickness $\left(\frac{\ell_w}{t_w}\right)$		
	$\frac{\ell_w}{t_w} \leq 2.5$	$2.5 < \frac{\ell_w}{t_w} \leq 6.0$	$\frac{\ell_w}{t_w} > 6.0$
$\frac{h_w}{\ell_w} < 2.0$	Walls	Walls	Walls
$\frac{h_w}{\ell_w} \geq 2.0$	Wall piers	Wall piers	Walls

依幾何不連續面對豎向構材作分類

- 垂直牆段：剪力行為主控
- 牆墩：撓曲行為主控
- 柱:撓曲行為主控



垂直牆段和牆墩之分類

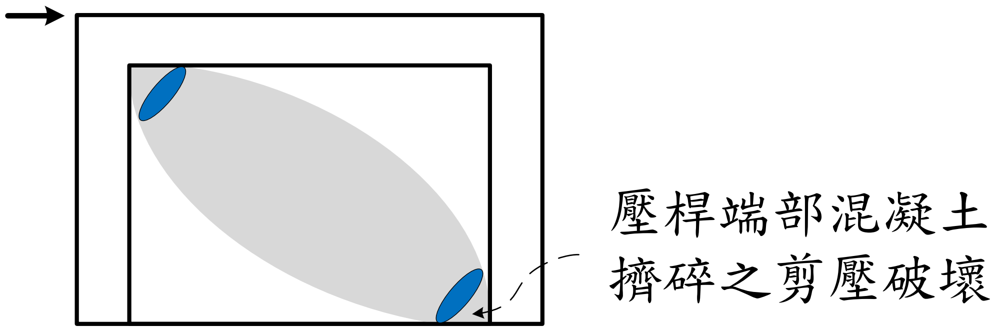
- 開孔造成牆版的幾何不連續，進而產生垂直牆段(Vertical wall Segment) 與牆墩(Wall pier)，如規範之下表所示。

	The ratio between wall length and thickness $\left(\frac{\ell_w}{t_w}\right)$		
	$\frac{\ell_w}{t_w} \leq 2.5$	$2.5 < \frac{\ell_w}{t_w} \leq 6.0$	$\frac{\ell_w}{t_w} > 6.0$
$\frac{h_w}{\ell_w} < 2.0$	Wall	Wall	Wall
$\frac{h_w}{\ell_w} \geq 2.0$	Wall piers	Wall piers	Wall

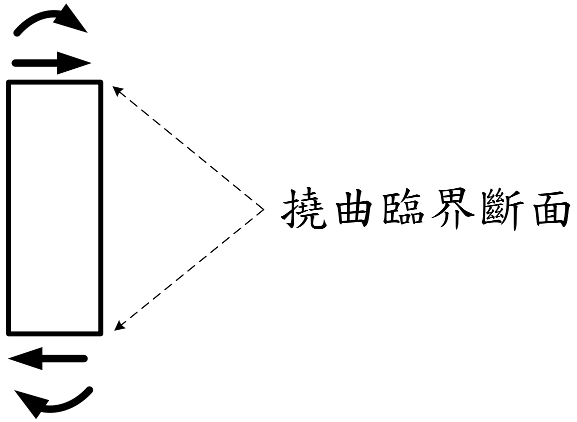
h_w: 考慮牆段或牆墩之淨高

ℓ_w：沿剪力方向考慮之牆段或牆墩長度

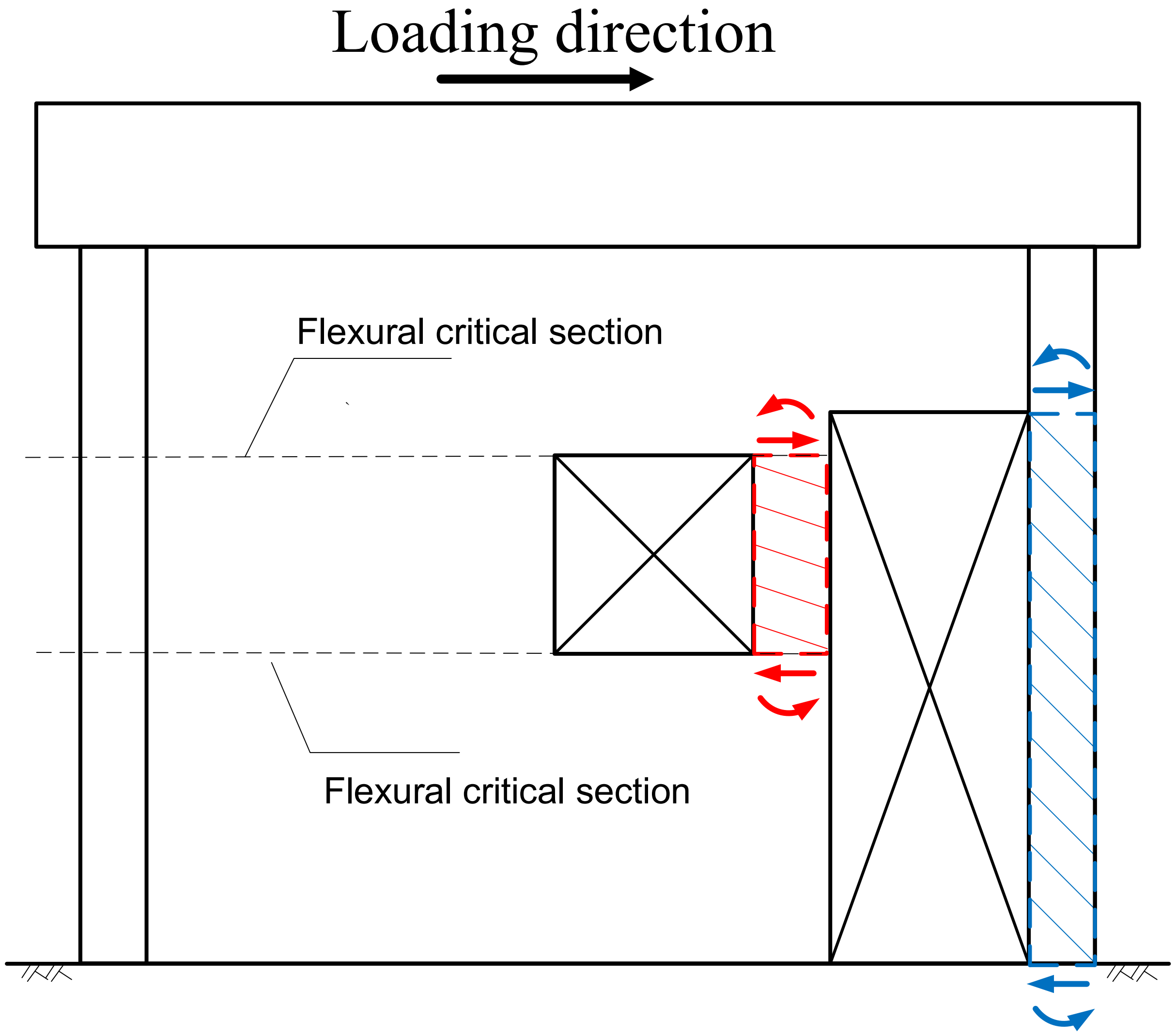
- 垂直牆段(Vertical wall Segment)的力學行為與低矮型剪力牆相似，其如下所示。



- 牆墩(Wall pier)的力學行為與柱相似，其如下所示。

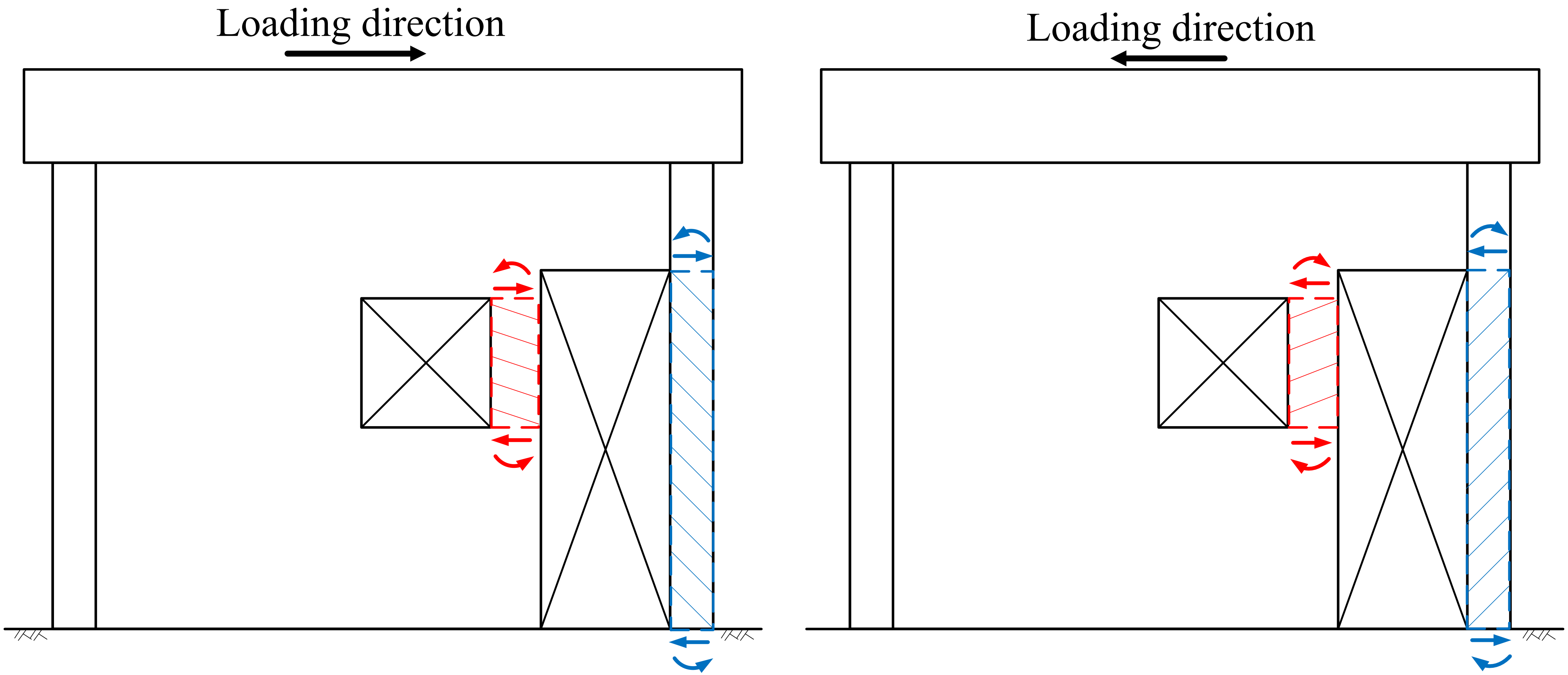


牆墩與柱力學元素高度之選取

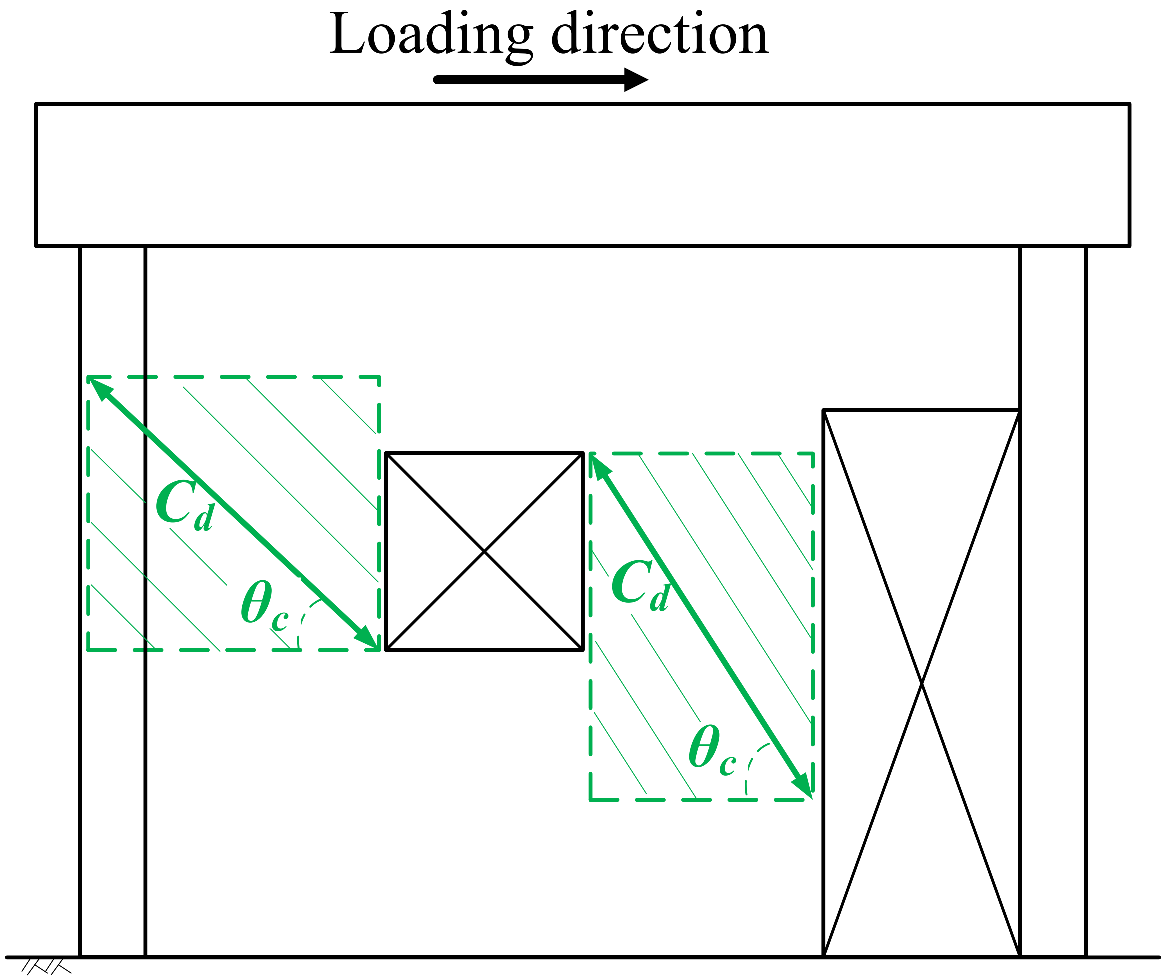


- 力學元素之高度取淨高
- 牆墩與柱之力學行為由撓曲控制
- 淨高係指構件上、下兩端撓曲臨界斷面之距離

撓曲臨界斷面所定義之淨高

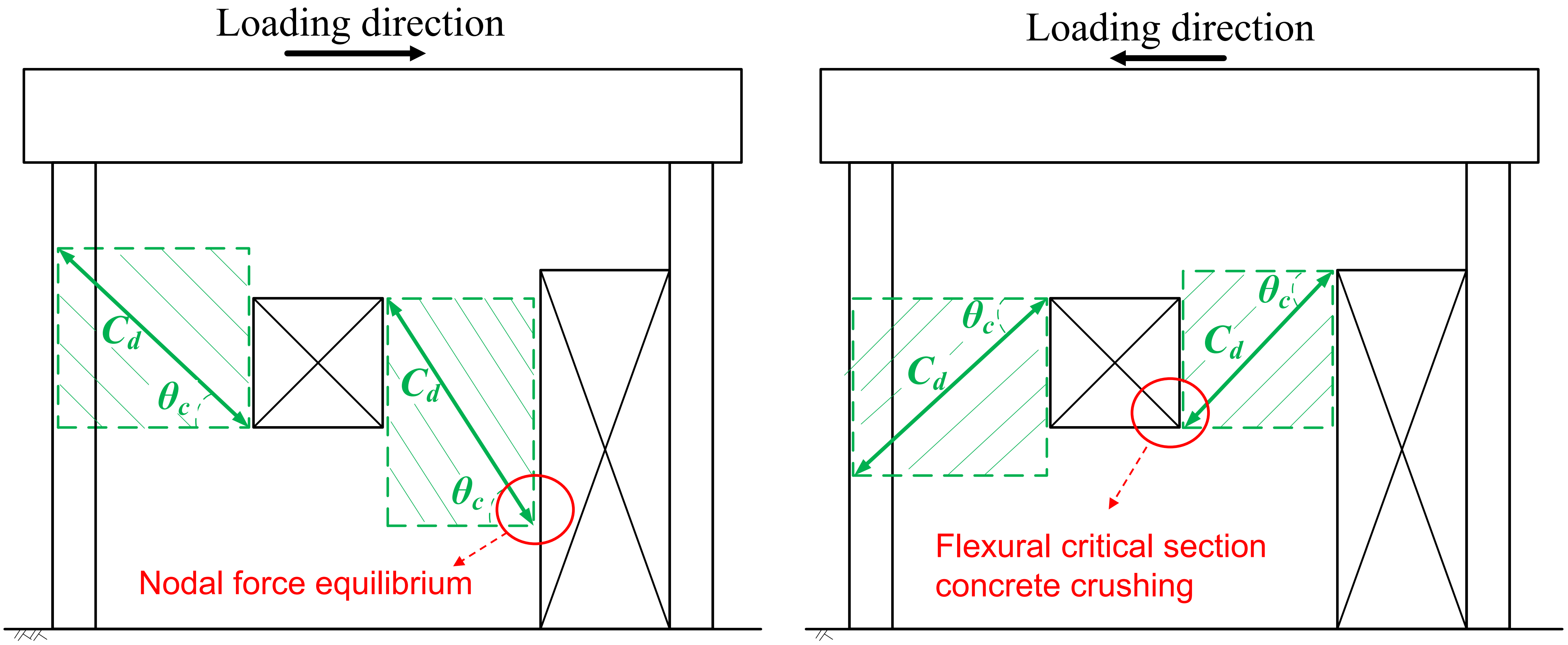


垂直牆段抗剪元素力學高度之認定

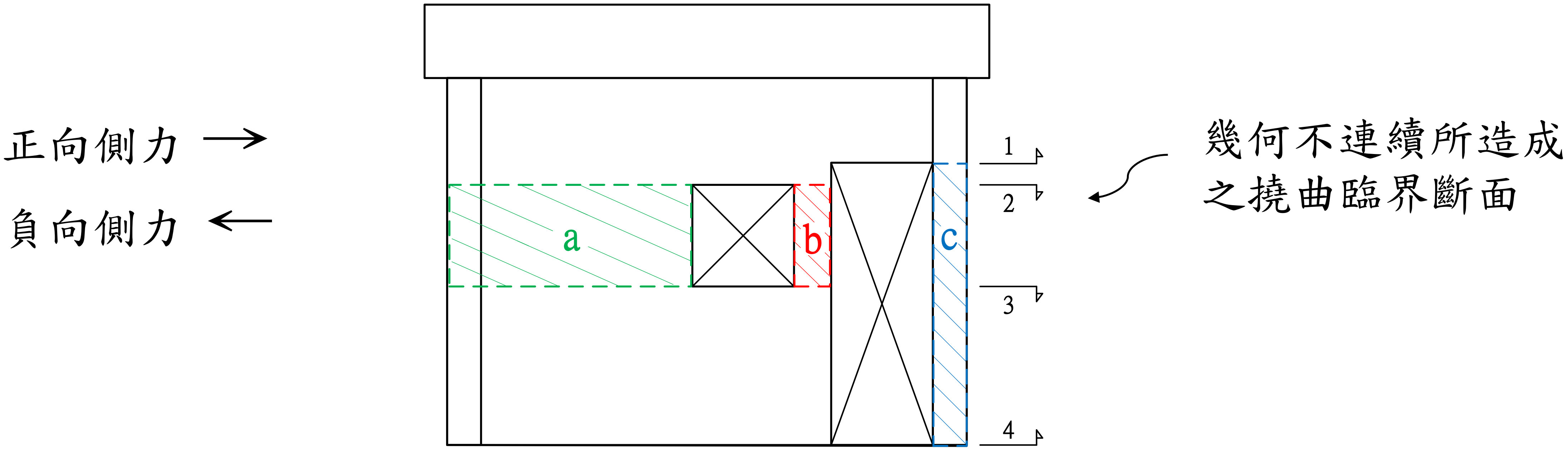


- 混凝土擠碎發生在近開孔處撓曲臨界斷面上。
- 對角壓桿近牆或開孔邊緣之節點需藉牆版內部抗拉作用達到力平衡的狀態。

混凝土擠碎與節點力平衡

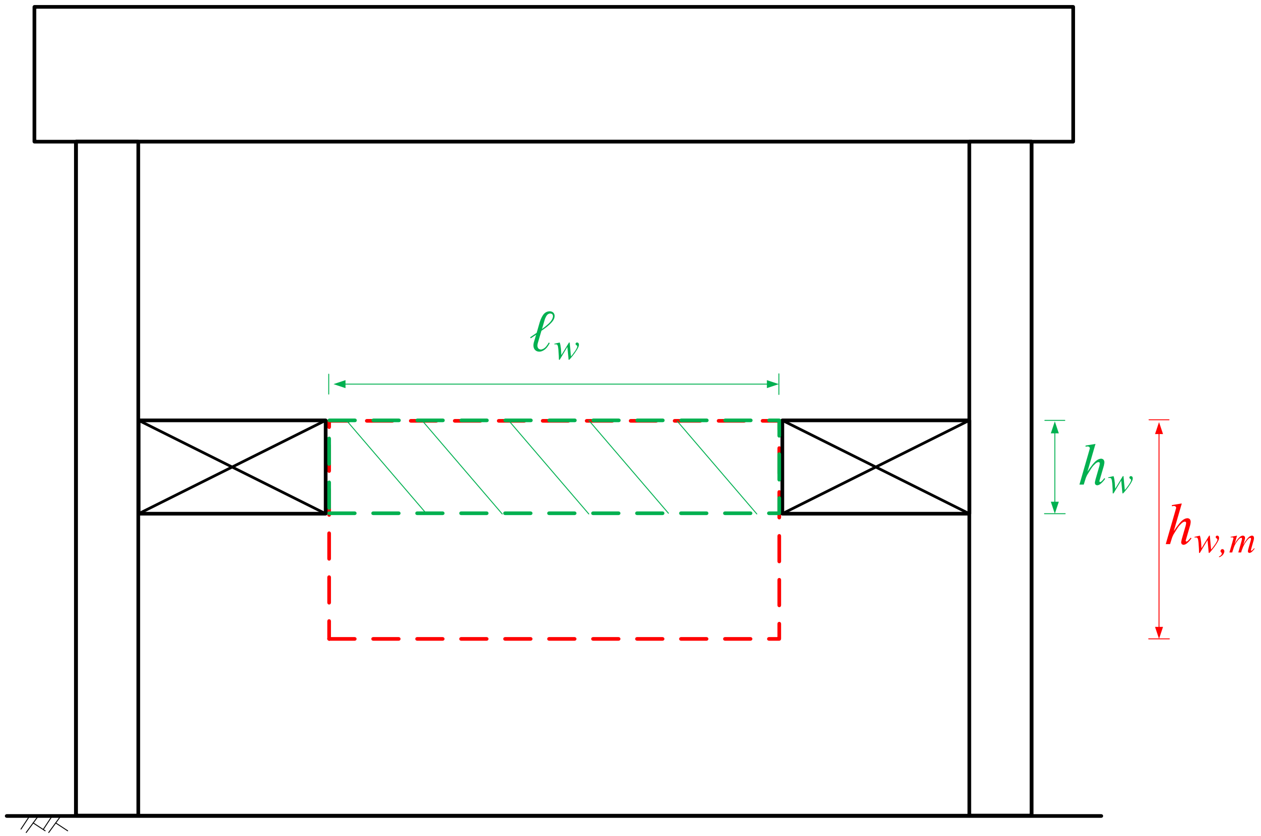


抗剪元素高度之認定



	正向側力		負向側力		說明
	起點	終點	起點	終點	
垂直牆段 “a”	力平衡	斷面3混凝土擠碎處	斷面2混凝土擠碎處	力平衡	1. 混凝土擠碎發生在近開孔處撓曲臨界斷面上。 2. 對角壓桿近牆邊緣之節點需藉牆版內部抗拉作用達到力平衡的狀態。
牆墩 “b”	斷面2	斷面3	斷面2	斷面3	1. 牆墩與柱取淨高，意即其力學元素高度係由構件上、下之撓曲臨界斷面所決定。
柱 “c”	斷面1	斷面4	斷面1	斷面4	

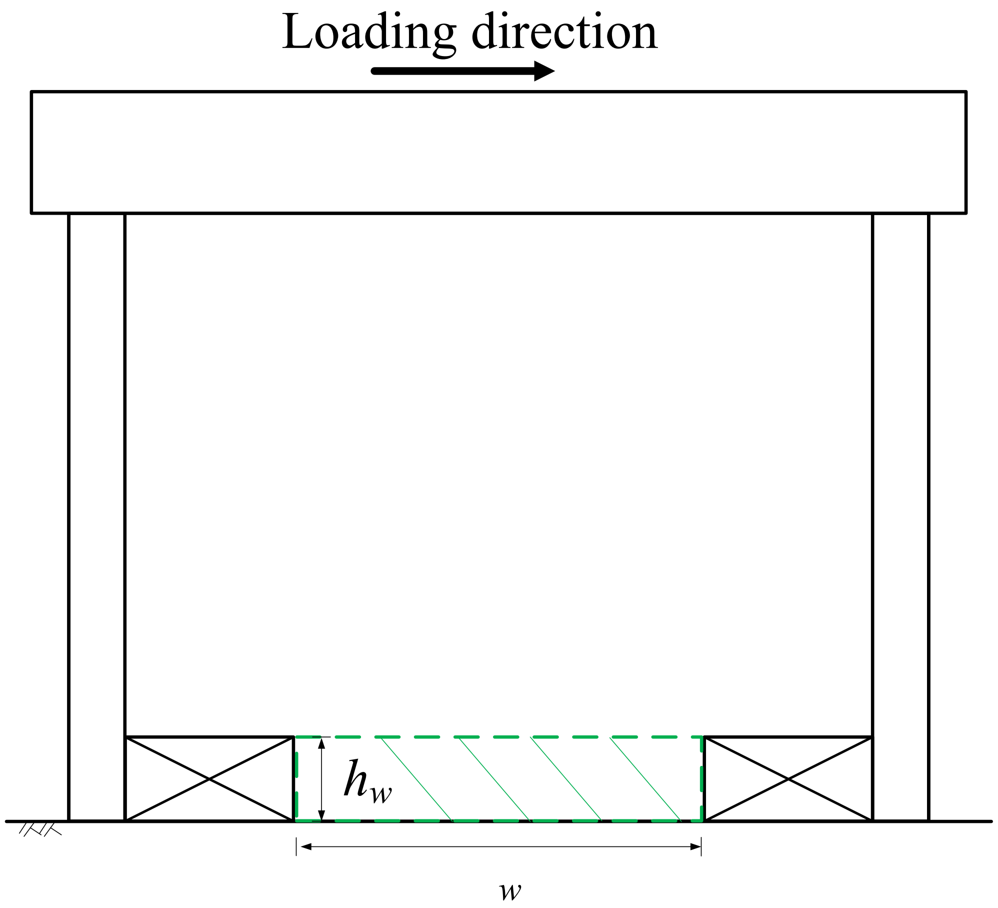
垂直牆段高度之調整

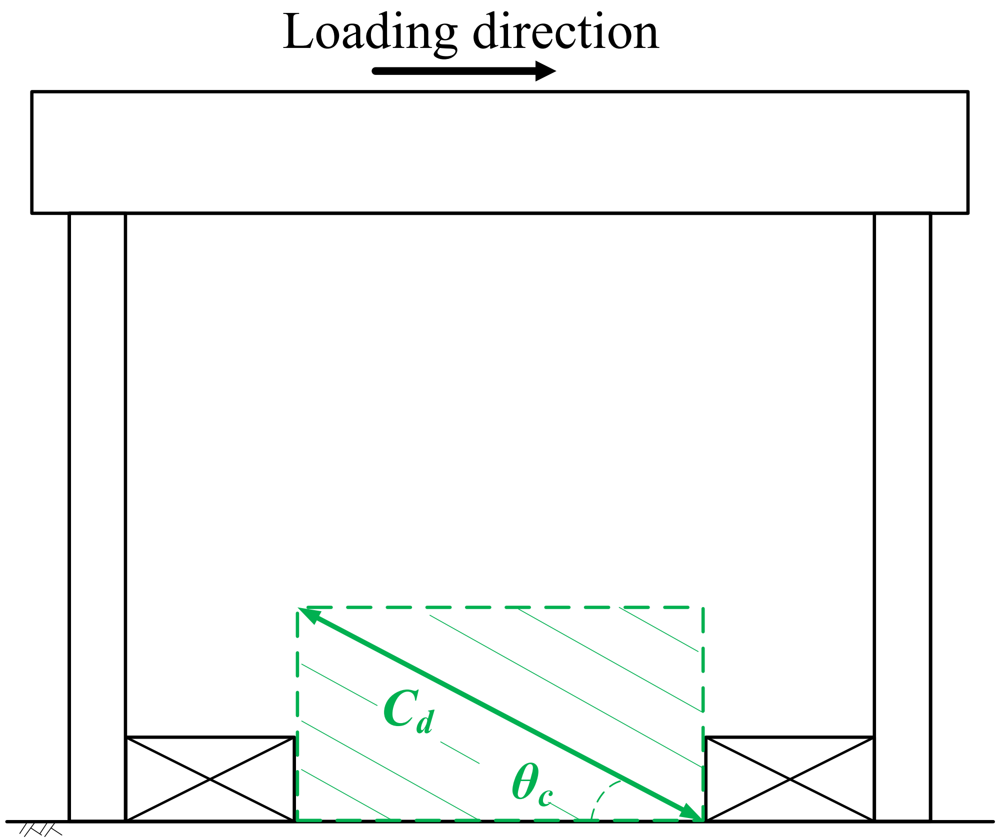


1. 按 Thurlimann 之研究，混凝土開裂角度θ應介於範圍 $[\tan^{-1}(\frac{1}{2})$, $\tan^{-1}(2)]$之間。
2. 左圖中垂直牆段之高度應按近開孔處之臨界斷面所決定，h_w。
3. 但若 $\frac{h_w}{\ell_w} < \frac{1}{2}$ 時，其無法形成低矮型剪力牆之傳力模式。
4. 本研究建議將垂直牆段之高度向下調整，藉牆版內部抗拉作用之力平衡，決定$h_{w,m}$的位置。
5. 設定$\frac{h_{w,m}}{\ell_w} \geq \frac{1}{2}$之限制。

Thürlimann, B., “Shear Strength of Reinforced and Prestressed Concrete-CEB Approach,” Concrete Design: U.S. and European Practices, SP-59, American Concrete Institute, Farmington Hills, MI, 1979, pp. 93-116.

調整對角壓桿的傾斜角至合理的範圍



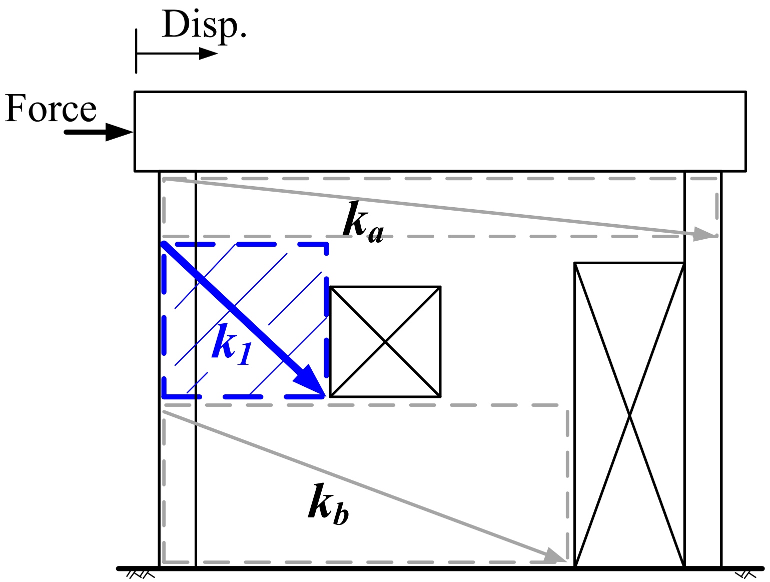


$\frac{h_w}{\ell_w} < 0.5 \rightarrow$ Need modify

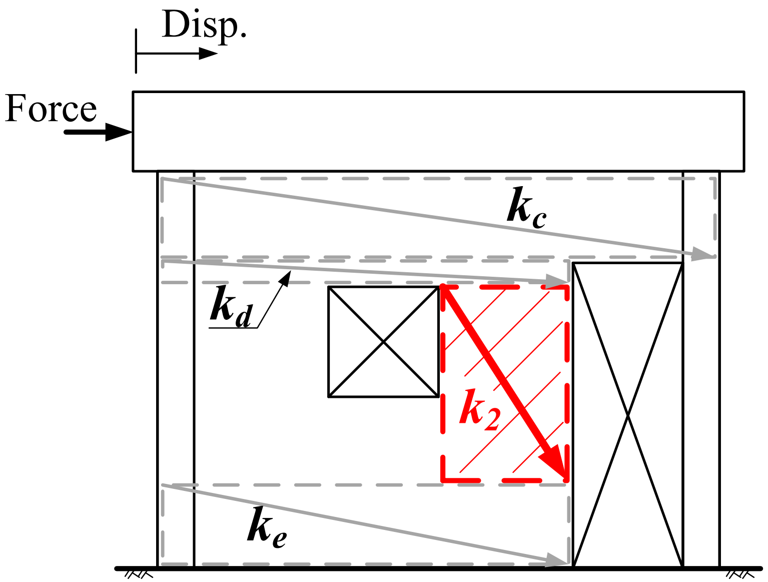
$h_{w,m}$ Determination via Force Equilibrium

開孔牆傳力路徑之整合

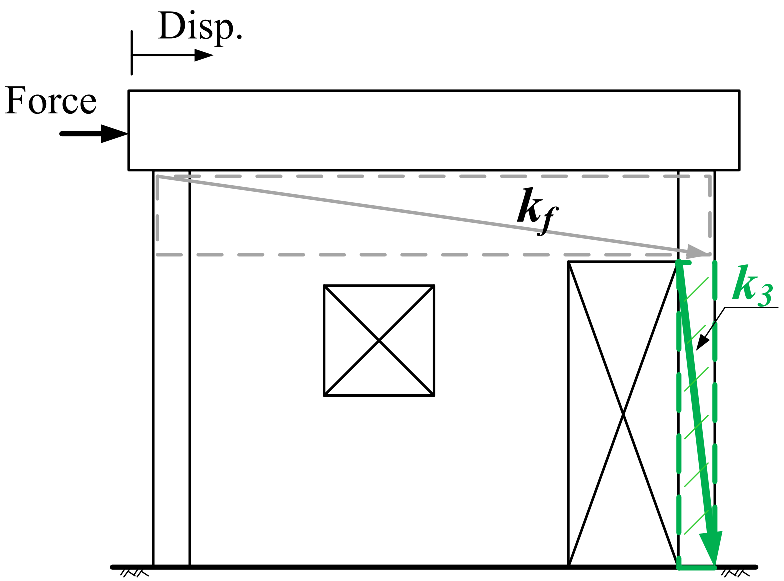
受剪牆版模擬為傳力路徑之彈簧並聯系統



(a) Loading paths for the critical element 1

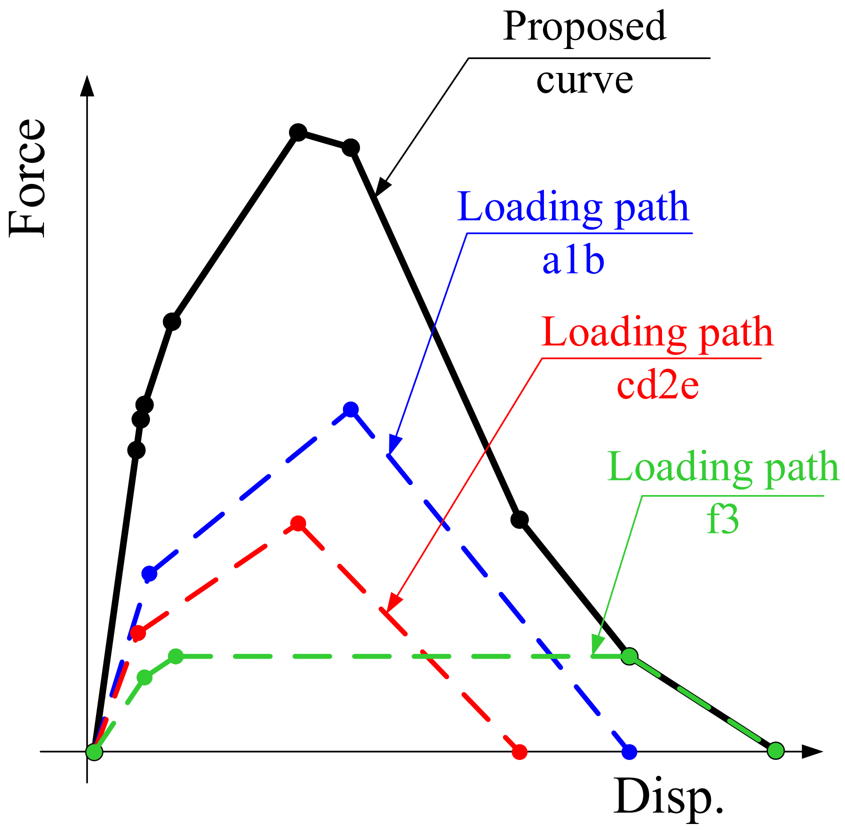


(b) Loading paths for the critical element 2

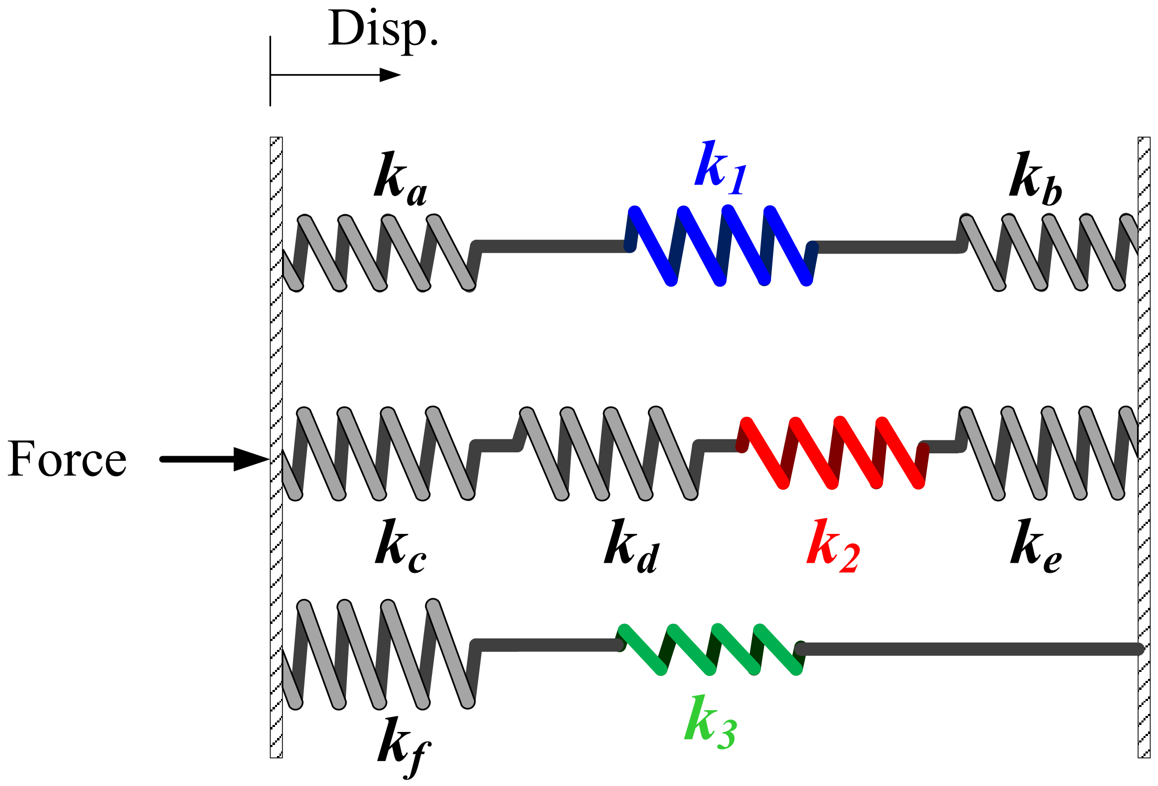


(c) Loading paths for the critical element 3

傳力路徑彈簧串並聯系統之整合

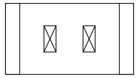
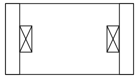
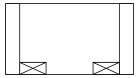
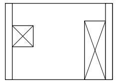
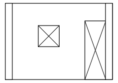
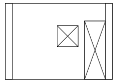
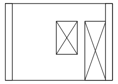
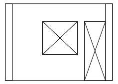


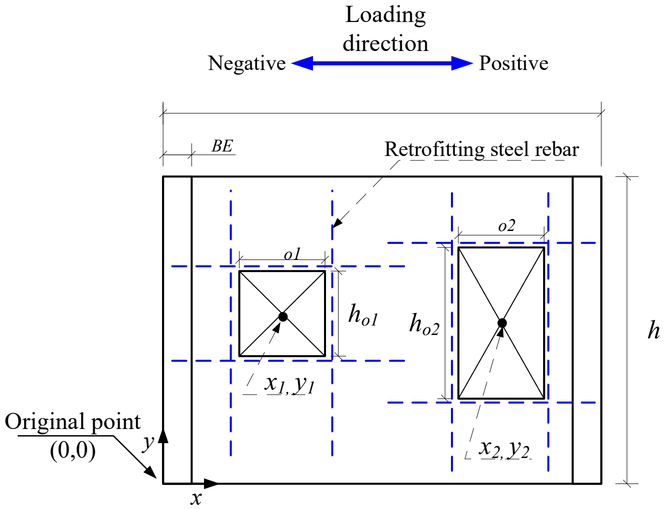
Assemblage of all loading paths



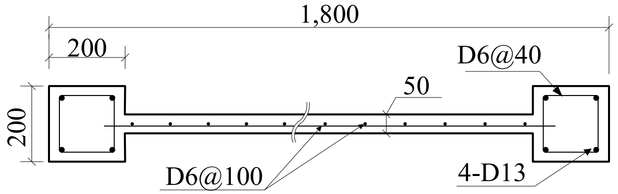
Spring model for entire system

多重開孔牆之實驗

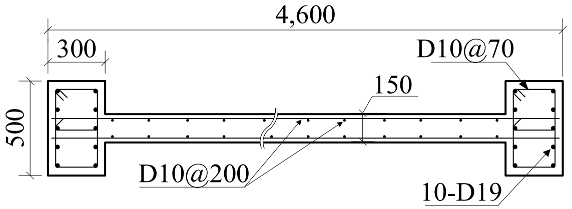
Specimen	Figure	f_c' [MPa]	$f_{y,web}$ [MPa]	$\frac{N}{A_w f_c'}$
Ono and Tokuhiro (1992)				
FW5-0.261 D-C		30.6	380	0.14
FW5-0.261 D-CLR		21.6	380	0.20
FW5-0.261 D-BLR		27.0	380	0.16
Ou et al. (2019)				
W1		35.6	313	0
W2		34.9	313	0
W3		37.4	313	0
W4		37.3	313	0
W5		36.1	313	0



(a) Dimension of RC walls and location of openings

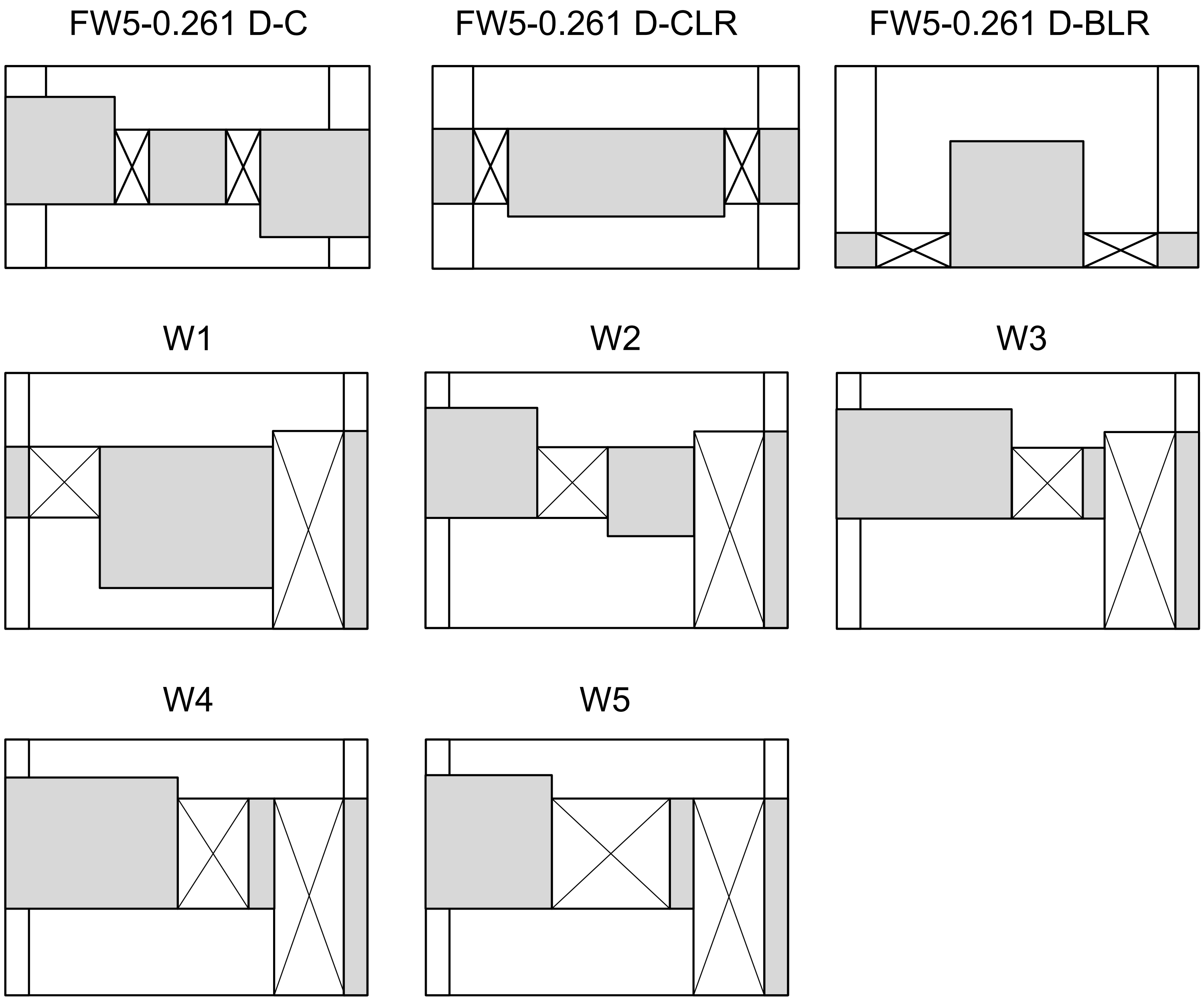


(b) Cross section of specimens reported by Ono an Tokuhiro (1992)



(c) Cross section of specimens reported by Ou et al. (2019)

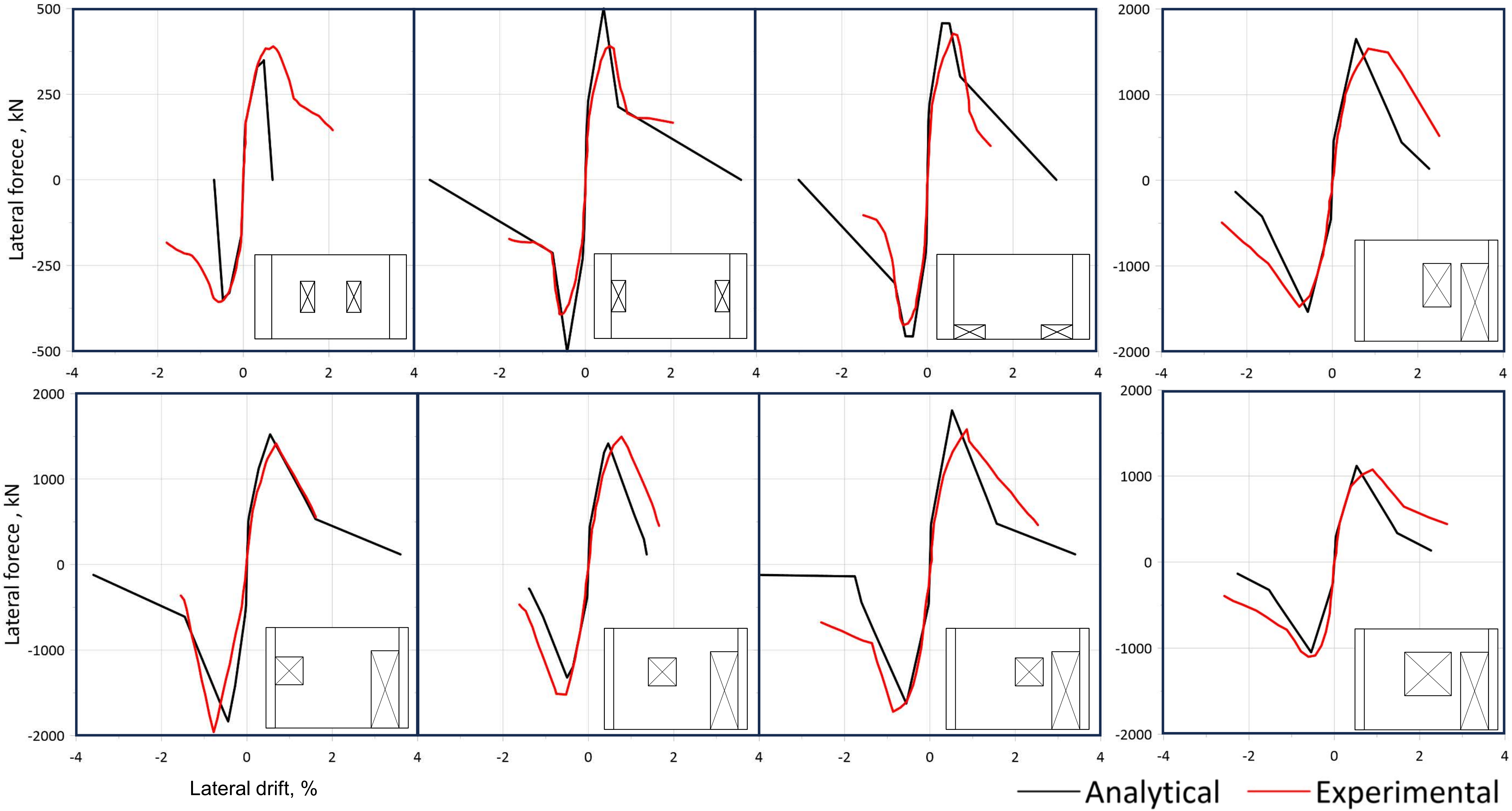
剪力元素之決定



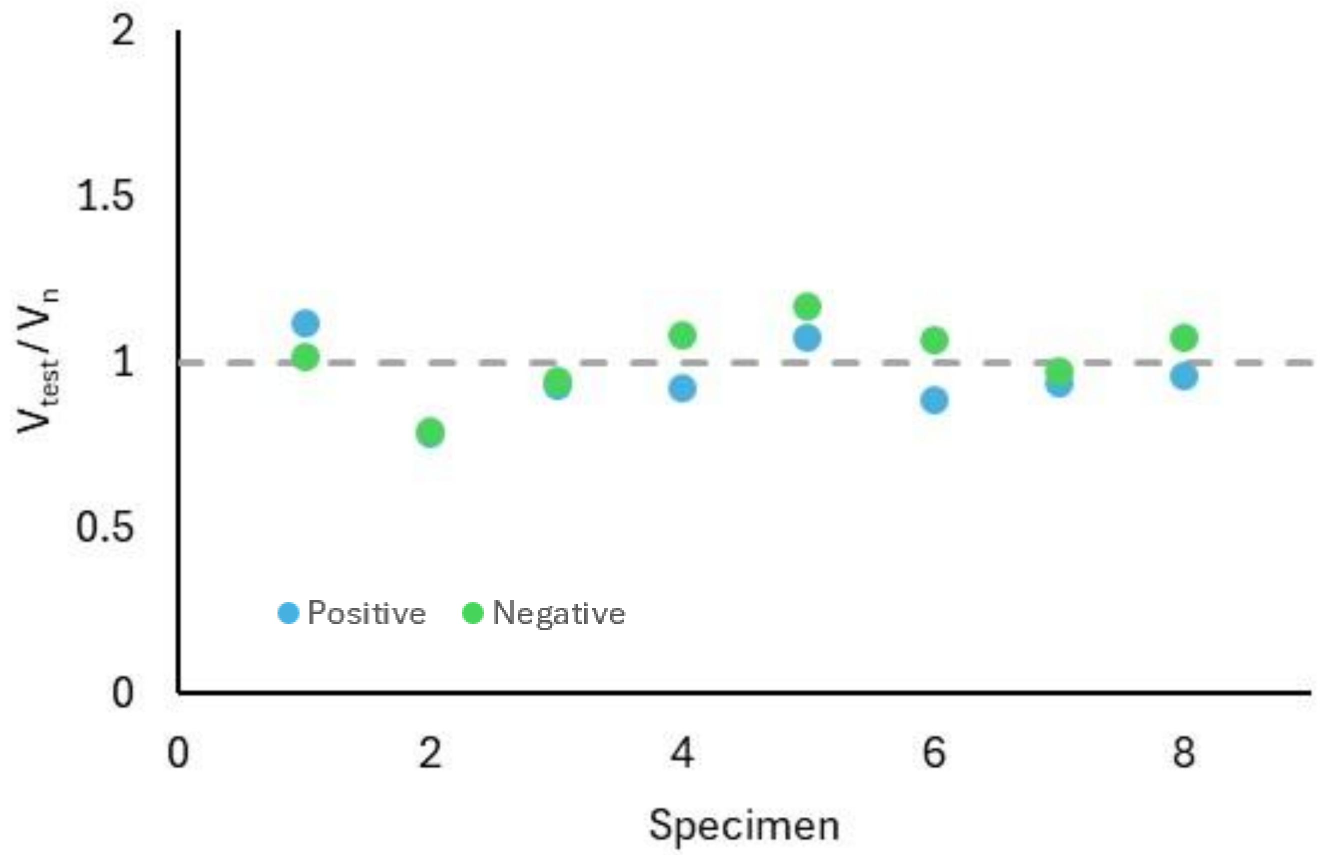
Ono and Tokuhiro (1992)

Ou et al. (2019)

側力位移曲線之驗證

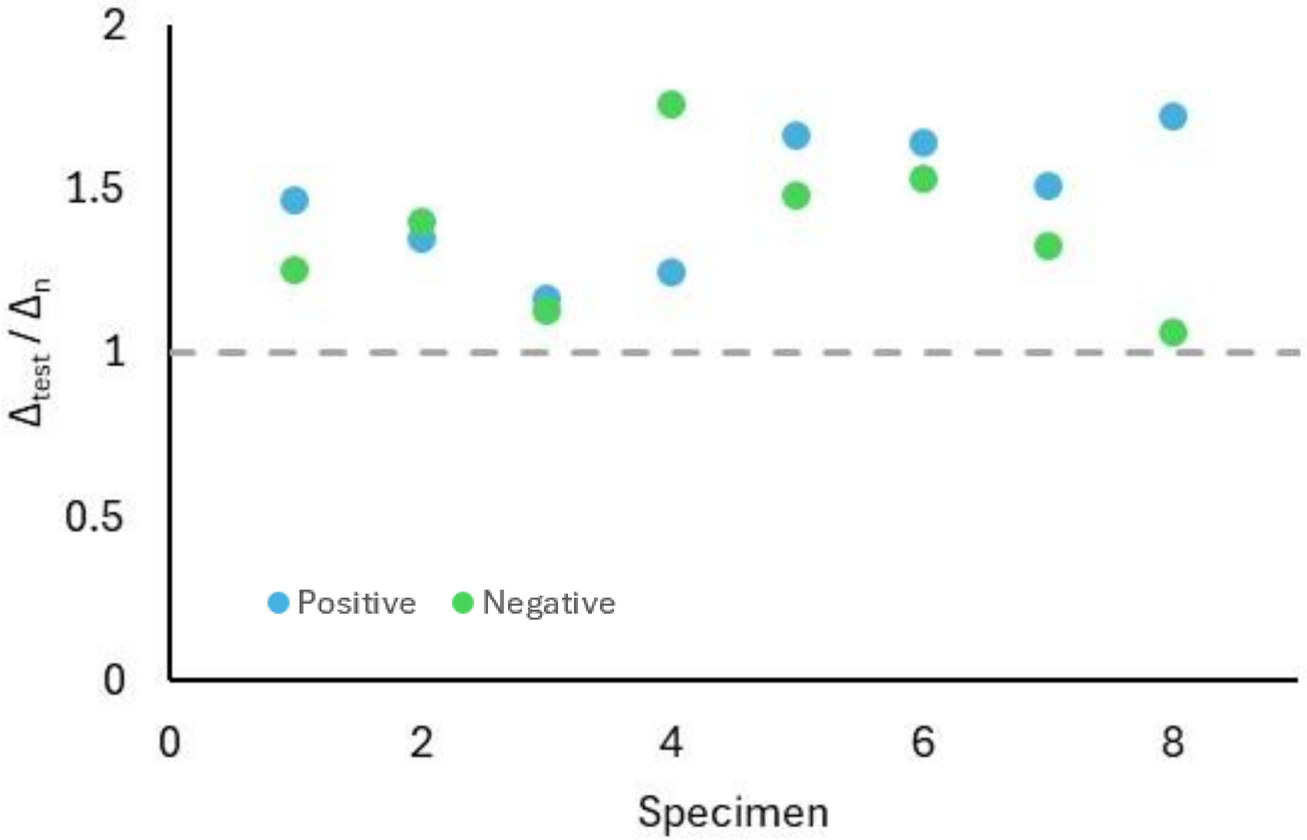


剪力強度之驗證



	AVG.	COV.
Positive	0.95	0.10
Negative	1.02	0.11

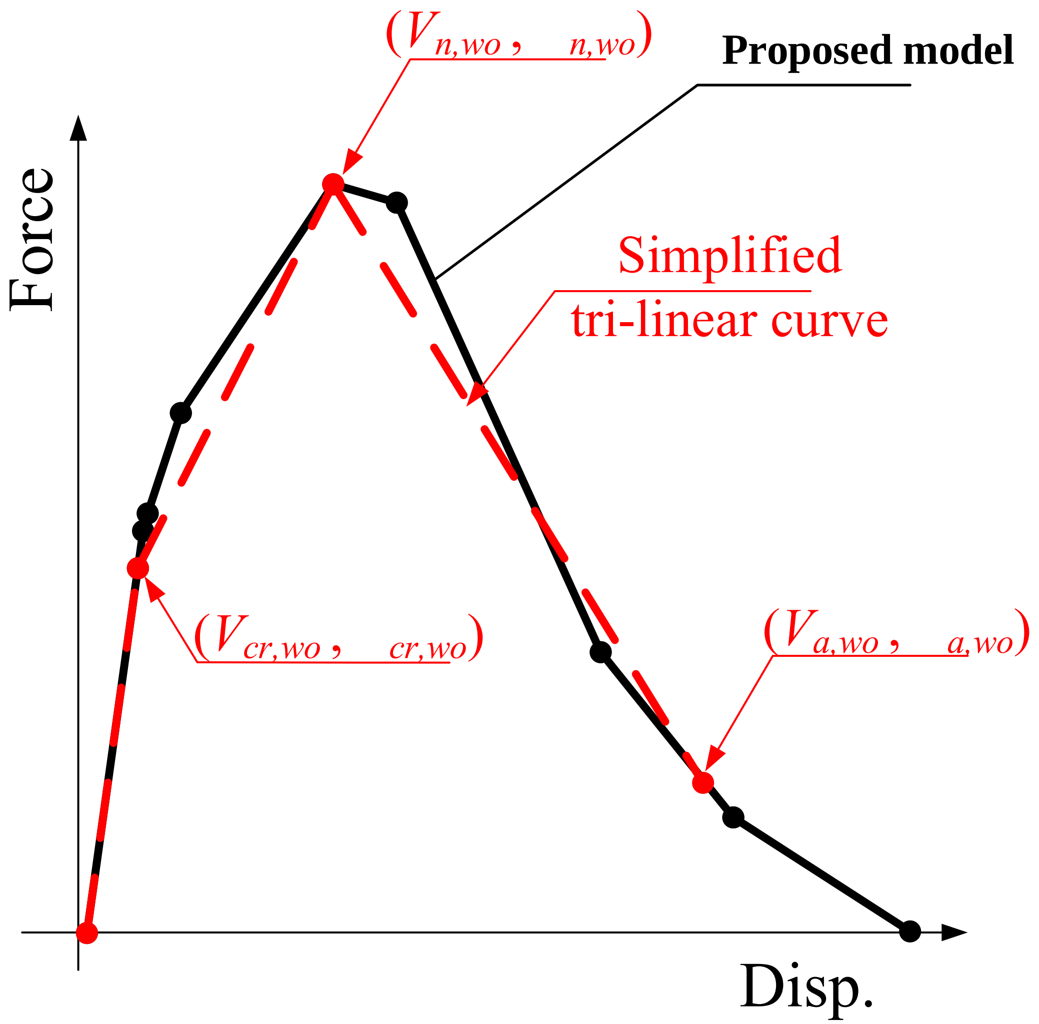
強度點位移之驗證



	AVG.	COV.
Positive	1.47	0.13
Negative	1.37	0.15

電腦分析模型之建議

側力位移曲線之簡化



- **Cracking point ($V_{cr,wo}$, $\Delta_{cr,wo}$) :**

  第一個剪力開裂之轉折點

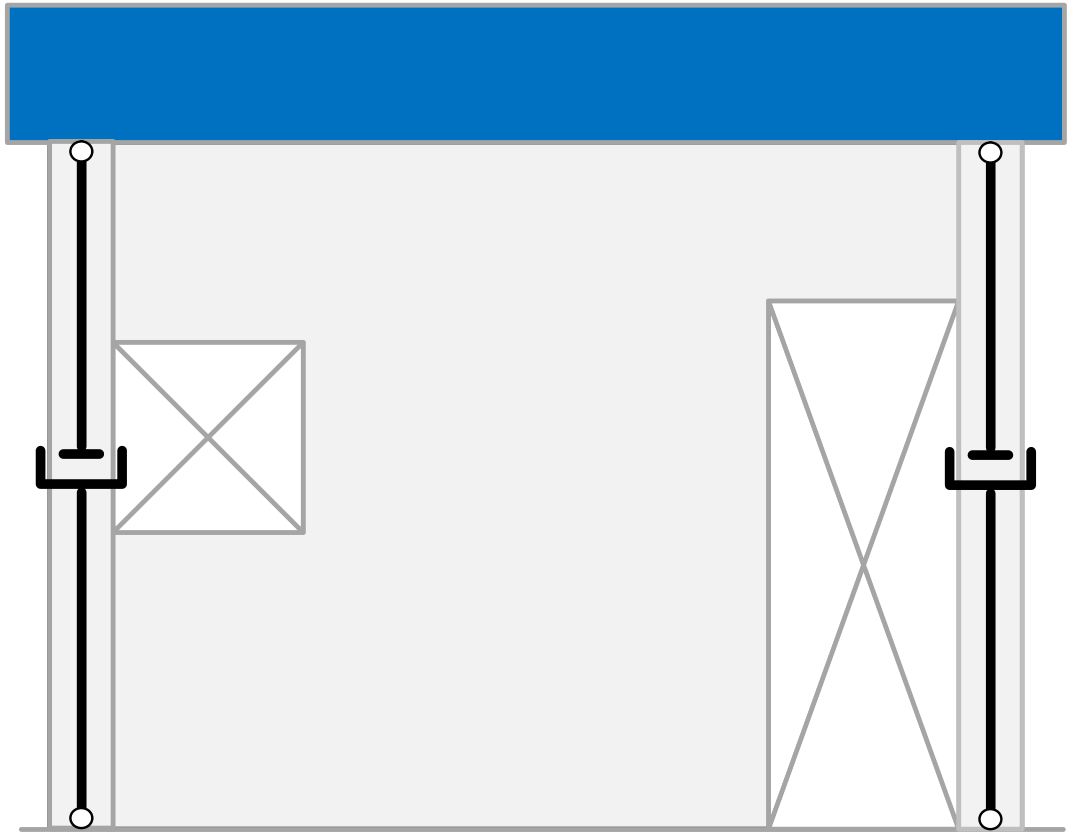
- **Strength point ($V_{n,wo}$, $\Delta_{n,wo}$) :**

  最大側力強度

- **Collapse point ($V_{a,wo}$, $\Delta_{a,wo}$) :**

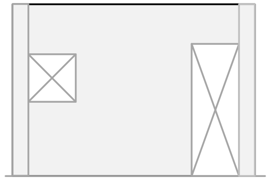
  $V_{a,wo} = 0.2\ V_{n,wo}$

電腦模擬



Modeling Assumptions

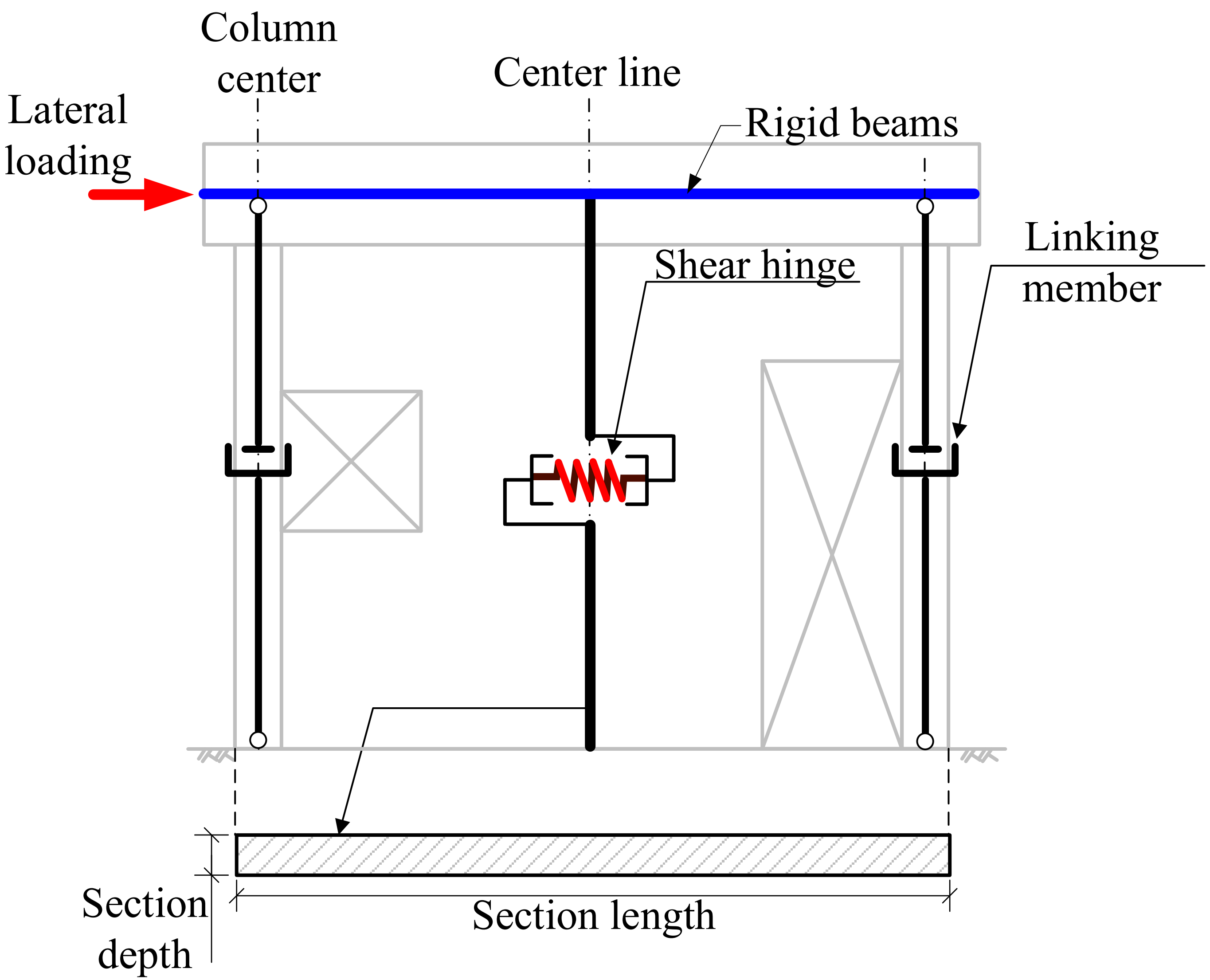
Beam → **Rigid beam**



Entire wall (including boundary column)
→ **Equivalent column with shear hinges**

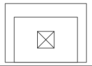
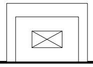
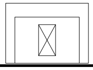
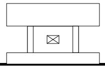
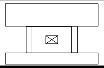
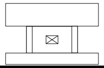
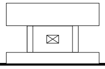
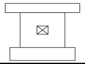
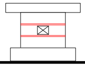
Two-force member placed at the boundary column

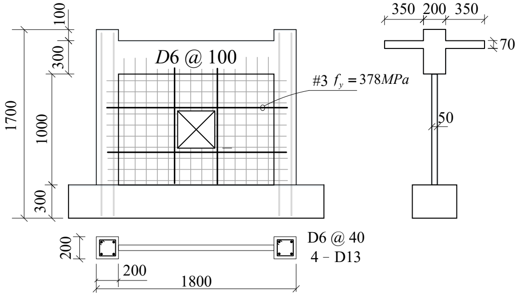
ETABS Model



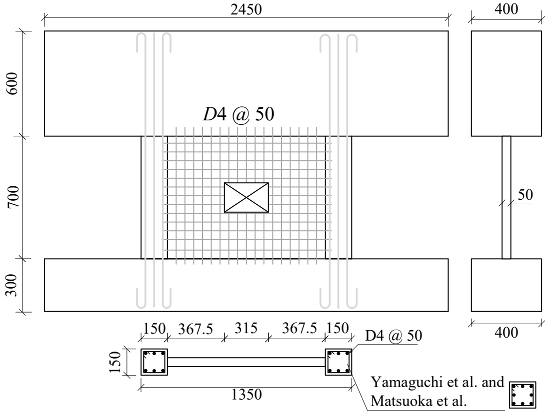
- Rigid beam connection at wall top
- Equivalent column with shear hinges for wall modeling (Consider the wall & boundary column)
- Two-force members at boundary columns

單窗開孔牆之實驗

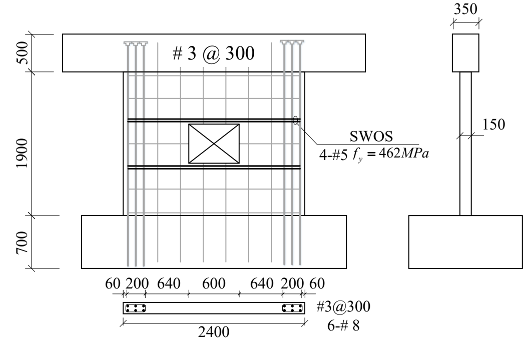
Specimen	Figure	f_c' [MPa]	$f_{y,web}$ [MPa]	$\frac{N}{A_w f_c'}$
Ono and Tokuhiro (1992)				
FW5-0.273-R-C		27.0	380	0.16
FW5-0.367-S-C		25.6	380	0.17
FW5-0.367-L-C		27.9	380	0.16
Tokuda et al. (2000)				
S1W5-0.28-0.01		30.9	160	0.11
Yamaguchi et al. (2001)				
S1W5-0.28-0.014		30.9	178	0.11
Matsuoka et al. (2003)				
FS1W5-0.28C-2.5		31.7	160	0.10
S1W5-0.28C-2.5		26.6	160	0.12
Yeh et al. (2018)				
SWO		30.9	348	0
SWOS		31.9	348	0



(a) Ono and Tokuhiro



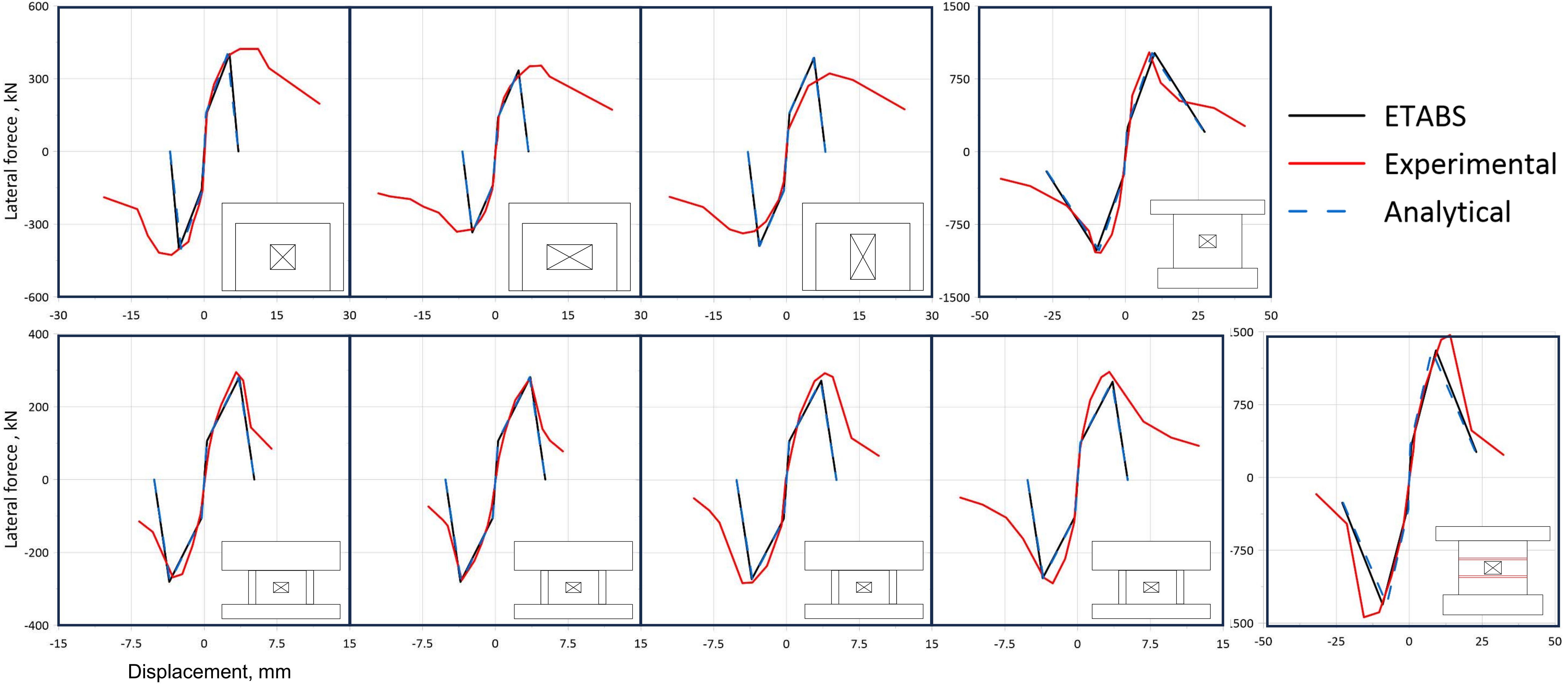
(b) Tokuda et al., Yamaguchi et al. and Matsuoka et al.



(c) Yeh et al.

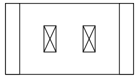
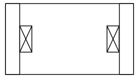
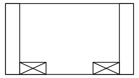
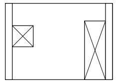
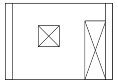
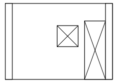
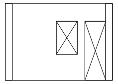
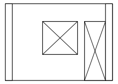
Tsai, R. J., Y. C. Hsu, and S. J. Hwang. 2021. "Prediction of lateral load displacement curve of reinforced concrete walls with openings under shear failure." ACI Struct. J. 118 (5): 275–284.

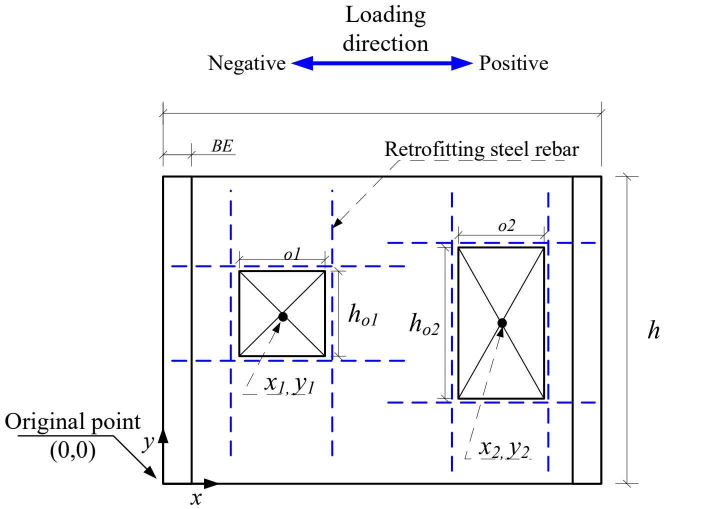
側力位移曲線之驗證



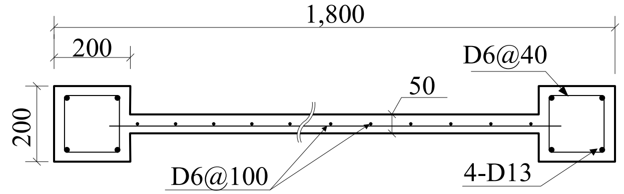
Lateral forece , kN
Displacement, mm
ETABS
Experimental
Analytical
600
300
0
-300
-600
1500
750
-750
-1500
400
200
-200
-400
-30
-15
15
30
-50
-25
25
50
-7.5
7.5

多重開孔牆之實驗

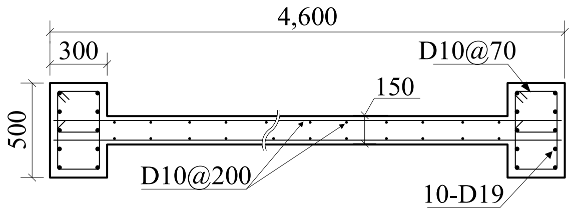
Specimen	Figure	f'_c [MPa]	$f_{y,web}$ [MPa]	$\frac{N}{A_w f'_c}$
Ono and Tokuhiro (1992)				
FW5-0.261 D-C		30.6	380	0.14
FW5-0.261 D-CLR		21.6	380	0.20
FW5-0.261 D-BLR		27.0	380	0.16
Ou et al. (2019)				
W1		35.6	313	0
W2		34.9	313	0
W3		37.4	313	0
W4		37.3	313	0
W5		36.1	313	0



(a) Dimension of RC walls and location of openings

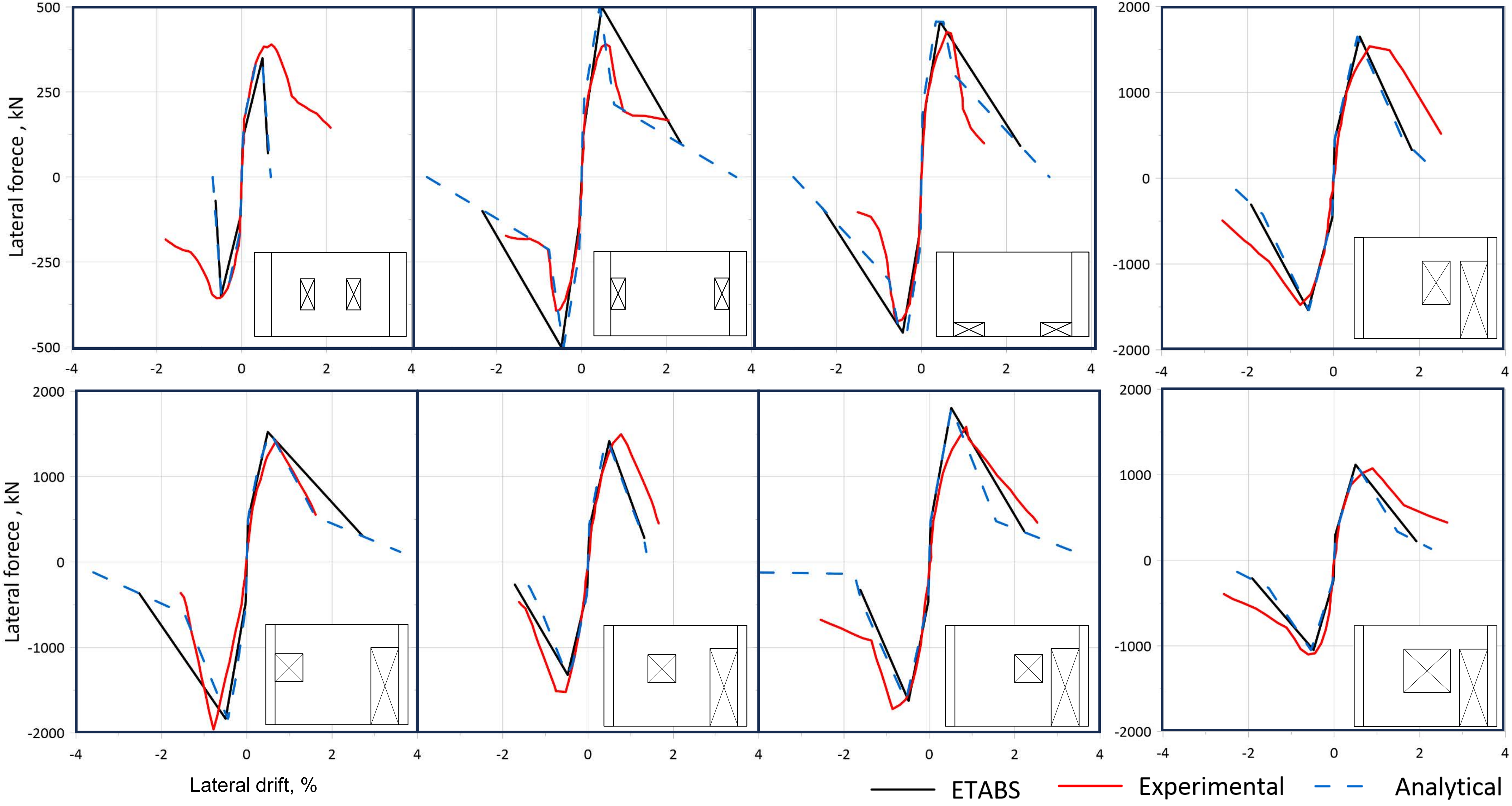


(b) Cross section of specimens reported by Ono an Tokuhiro (1992)



(c) Cross section of specimens reported by Ou et al. (2019)

側力位移曲線之驗證



Lateral forece , kN
500
250
0
-250
-500
2000
1000
0
-1000
-2000
-4
-2
0
2
4
Lateral drift, %
ETABS
Experimental
Analytical

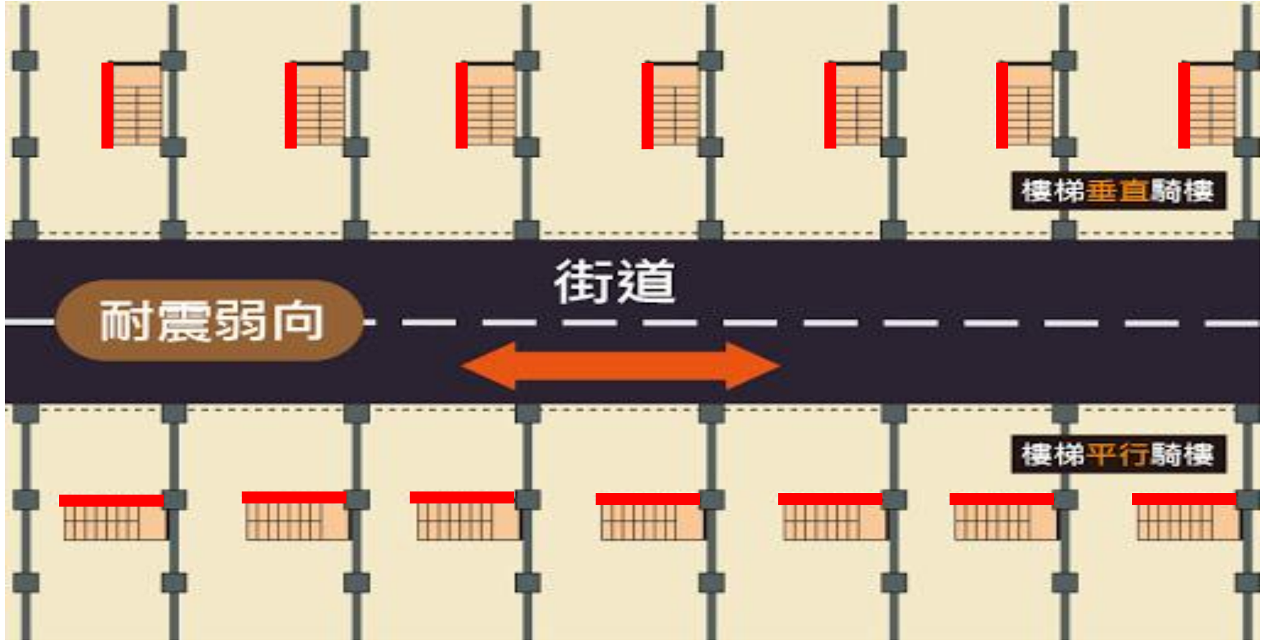
耐震評估與補強之應用

台灣街屋之屋後牆

杜昱石，(2014)「低矮鋼筋混凝土街屋具典型開口外牆之耐震行為研究」，碩士論文，國立台灣科技大學，營建工程學系，臺北，臺灣。

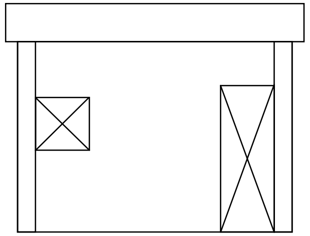
鄰街店鋪型住宅於921集集地震之耐震檢驗

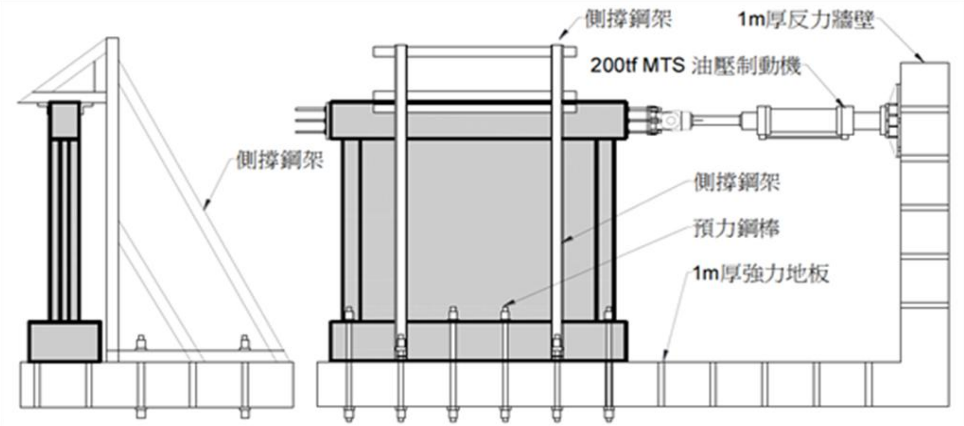
樓梯牆垂直騎樓
對耐震弱軸無效 → 倒塌



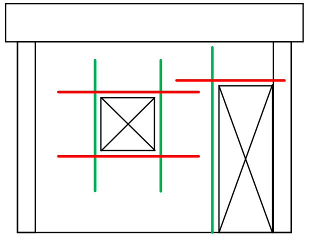
樓梯牆平行騎樓
對耐震弱軸有效 → 安全

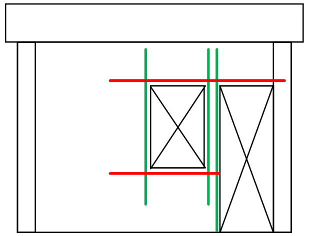
台灣街屋屋後開孔牆之實驗測試

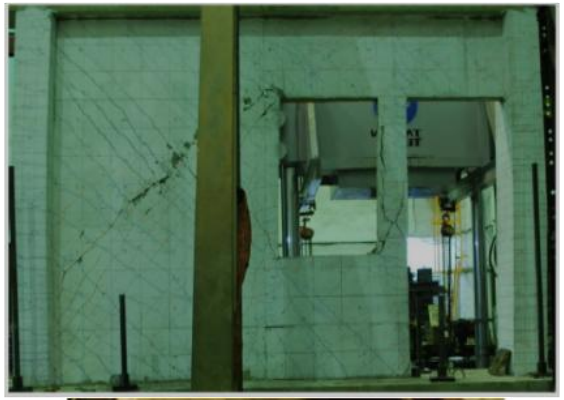


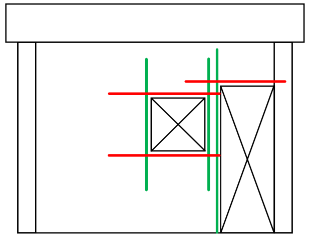


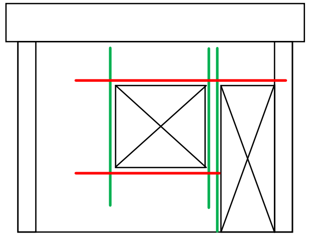
杜昱石 (2014)



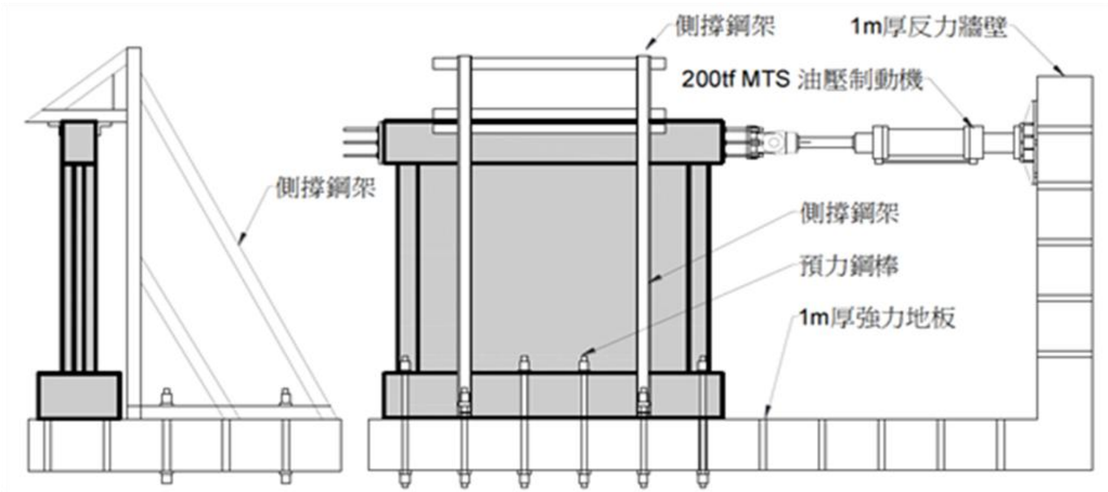




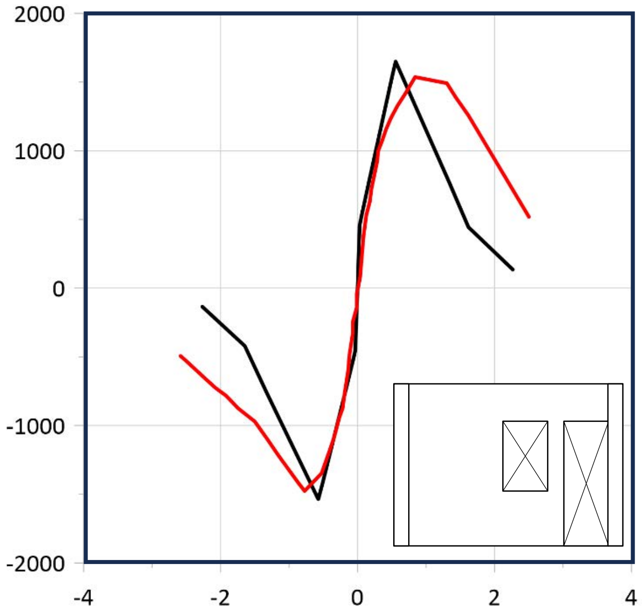


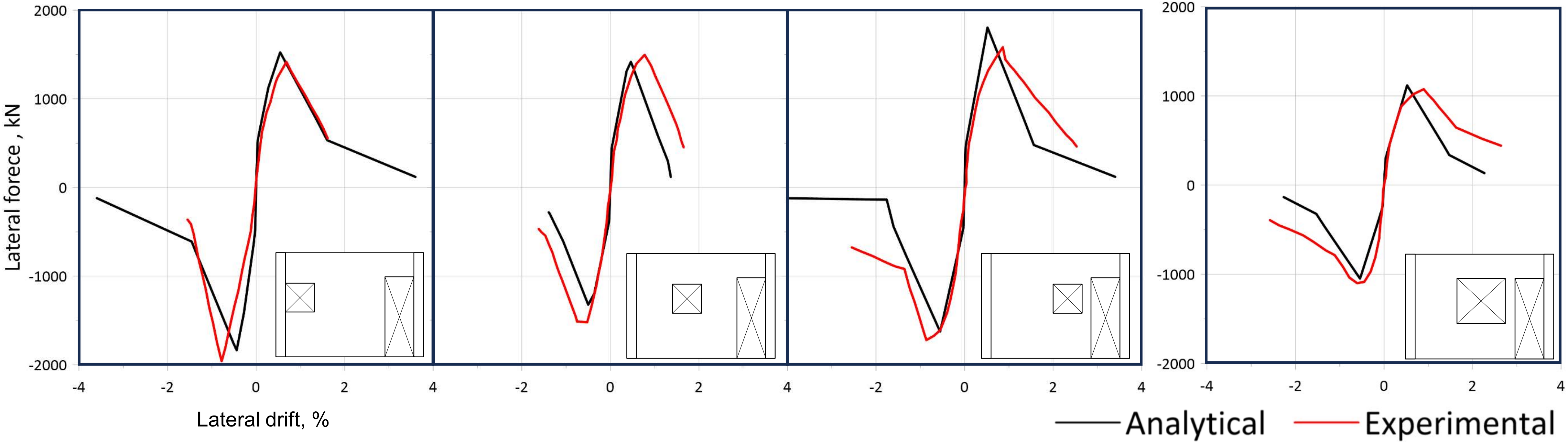


側力位移曲線之驗證

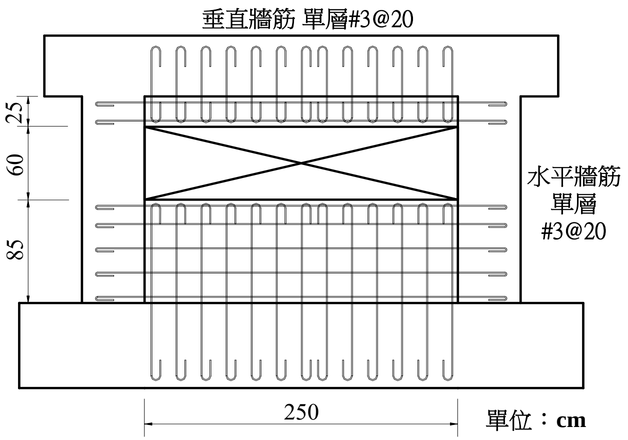


杜昱石 (2014)

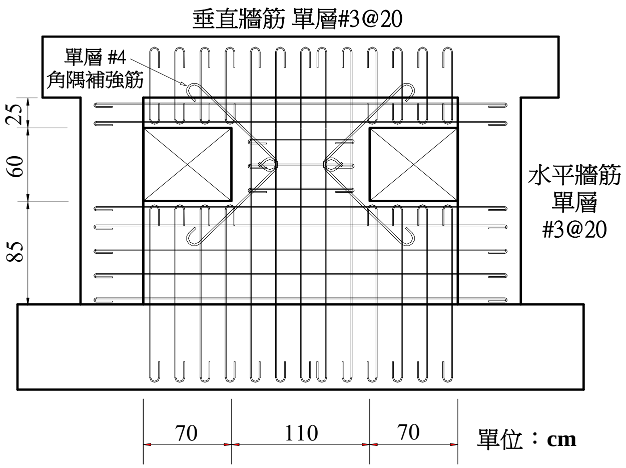




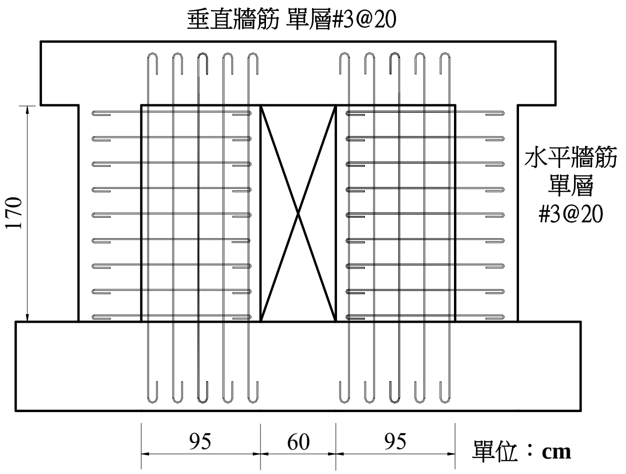
開孔剪力牆之實驗測試



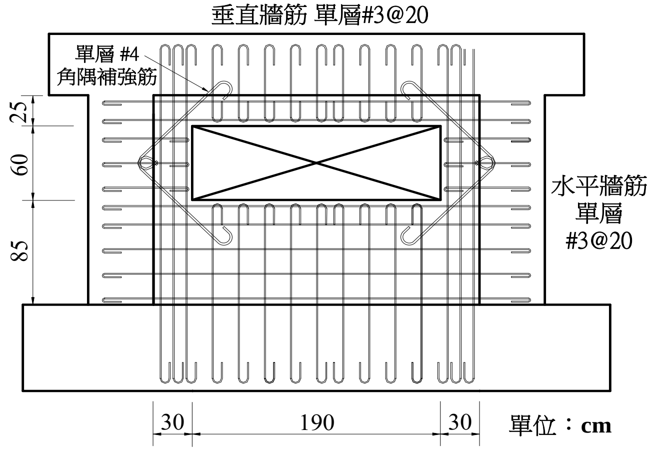
WO1W



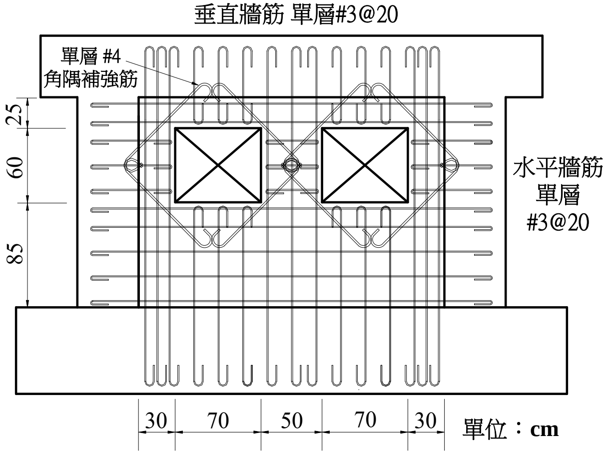
WO2W



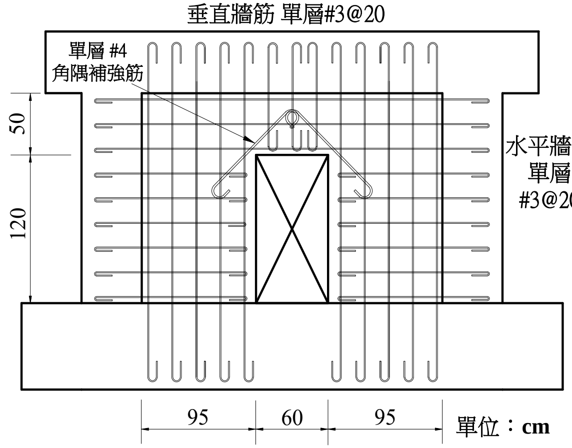
WO1D



WO1WF



WO2WF



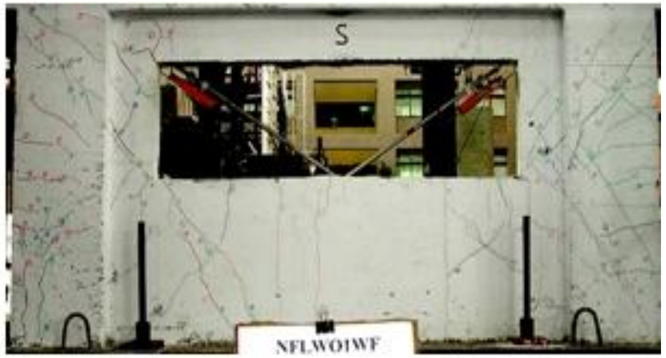
WO1DF

陳力平，(2002)「含開口RC牆非韌性構架之耐震行為研究」，碩士論文，國立台灣科技大學，營建工程學系，臺北，臺灣。

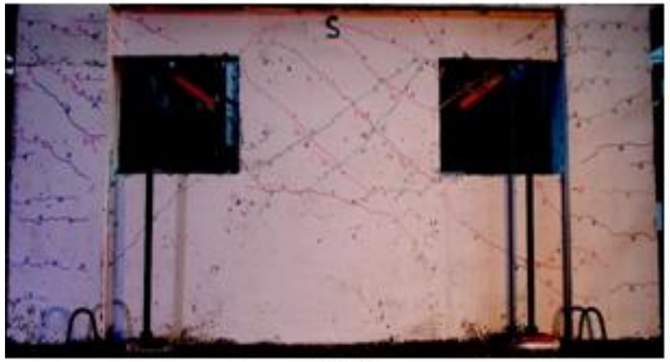
實驗之裂縫觀測

陳力平 (2002)

(a) WO1W試體 (0.75%)



(b) WO1WF試體 (0.75%)



(c) WO2W試體 (0.75%)

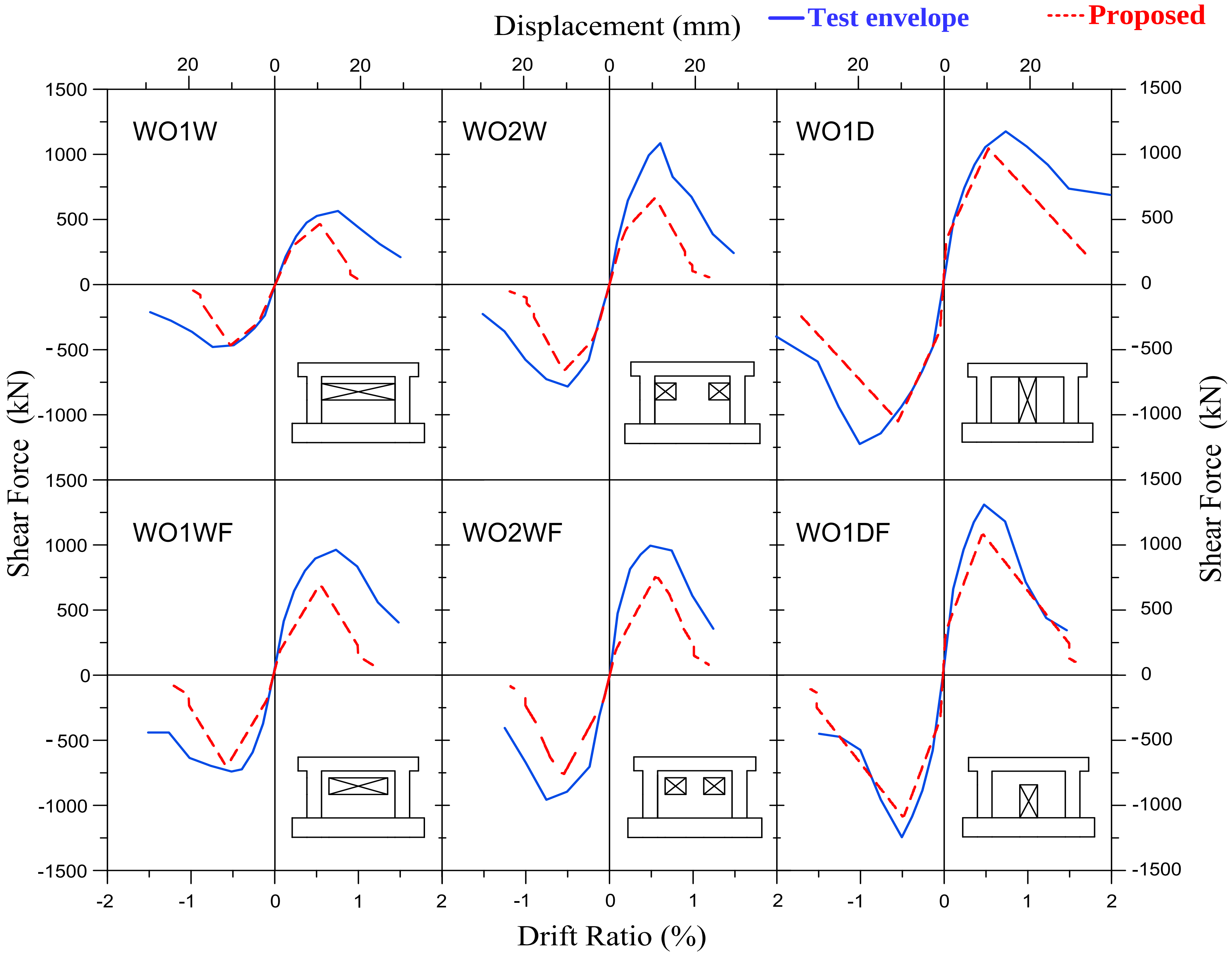
(d) WO2WF試體 (0.5%)

(e) WO1D試體 (-1.0%)

(f) WO1DF試體 (1.5%)

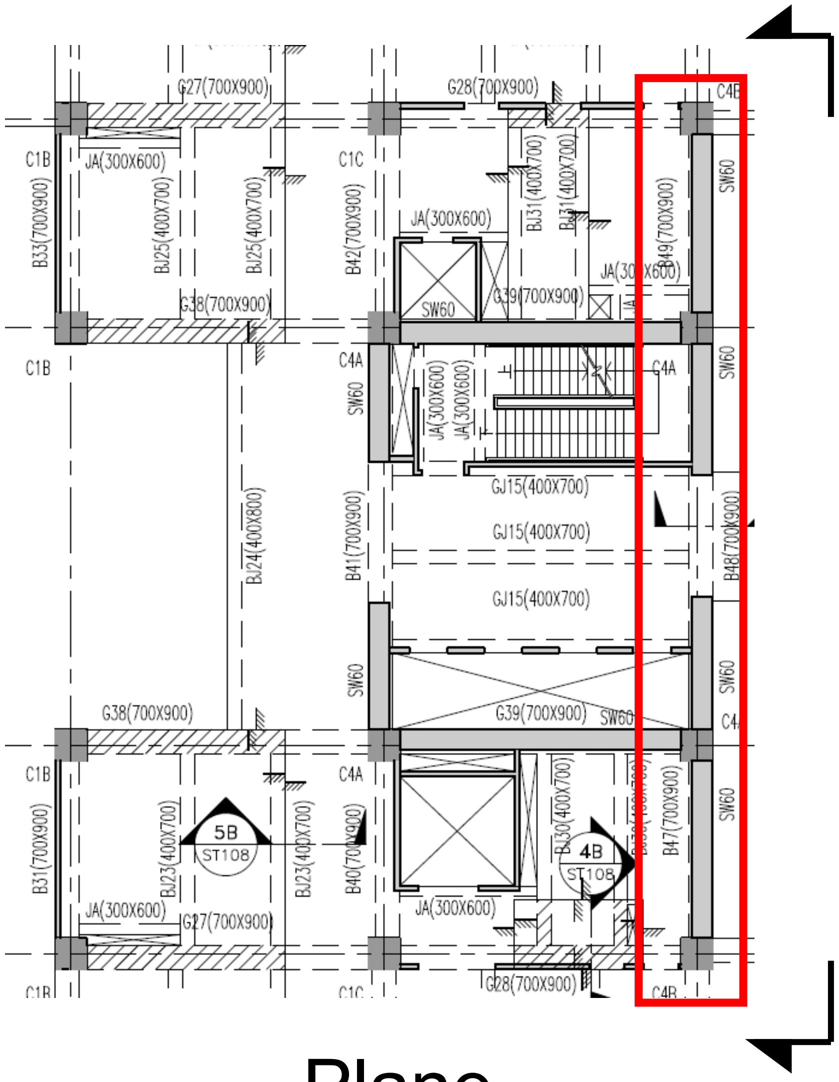
試驗驗證

曾建創、陳力平、黃世建，(2018)「含開口鋼筋混凝土牆非韌性構架試驗研究」，結構工程，第三十三卷，第一期，第68-83頁。

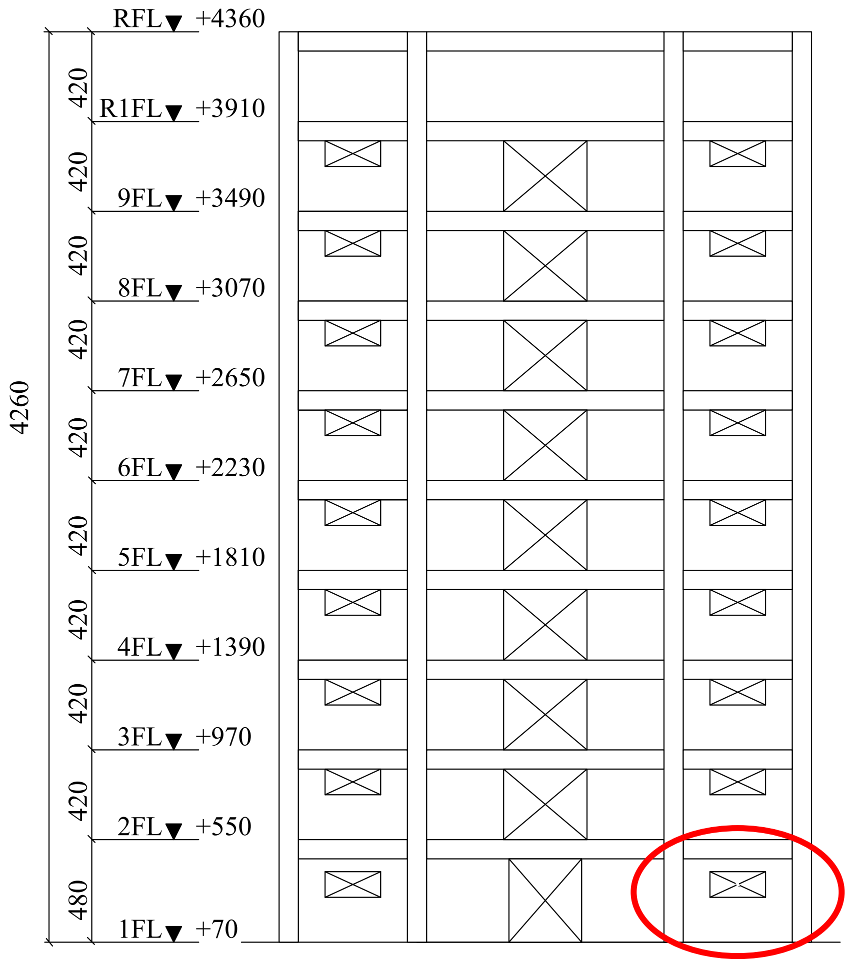


開孔牆之壓拉桿配筋方法

平面與立面

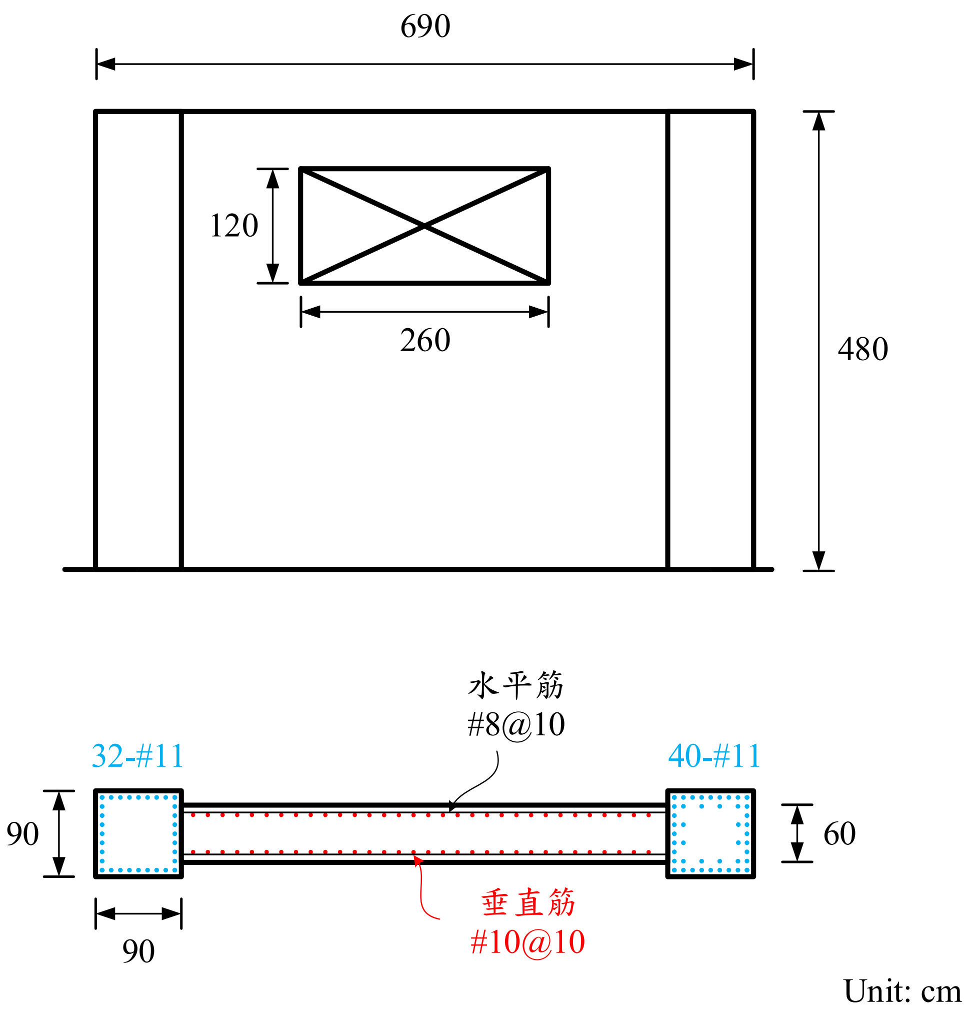


Plane



Elevation

設計參數



$f_c' = 280\ kgf/cm^2$

$f_y = 4200\ kgf/cm^2$

$N_u = 2{,}340\ tf$

$V_u = 1{,}140\ tf$

$\rho_\ell = 2.71\ \%$

$\rho_t = 1.69\ \%$

$\rho_{BE,L} = 2.50\ \%$

$\rho_{BE,R} = 3.12\ \%$

土木401-112規範方法

$$V_{n,ACI} = \left(\alpha_c \sqrt{f_c'} + \rho_t f_{yt}\right) t_w \ell_w = 2{,}175\ tf$$

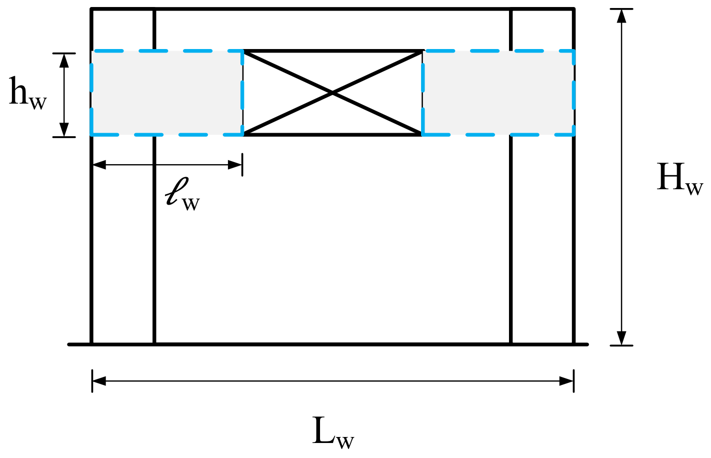
$$1 \le \alpha_{shape} = 0.7\left(1 + \frac{b_{f,eff} t_f}{A_{cv}}\right)^2 \le 1.2$$

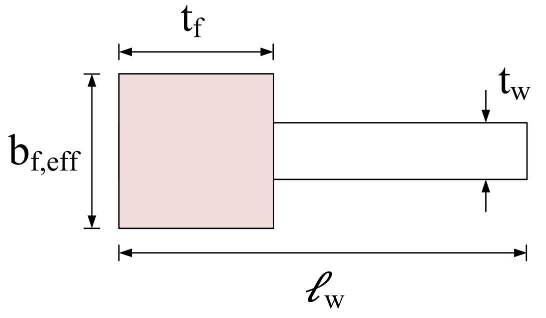
$$V_{max,ACI} = \alpha_{shape}\ 0.83\sqrt{f_c'}\ t_w \ell_w = 1{,}259\ tf$$

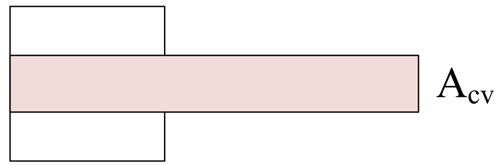
$$\Rightarrow V_{n,ACI} = V_{max,ACI} = 1{,}259\ tf$$

$\Phi = 0.75$

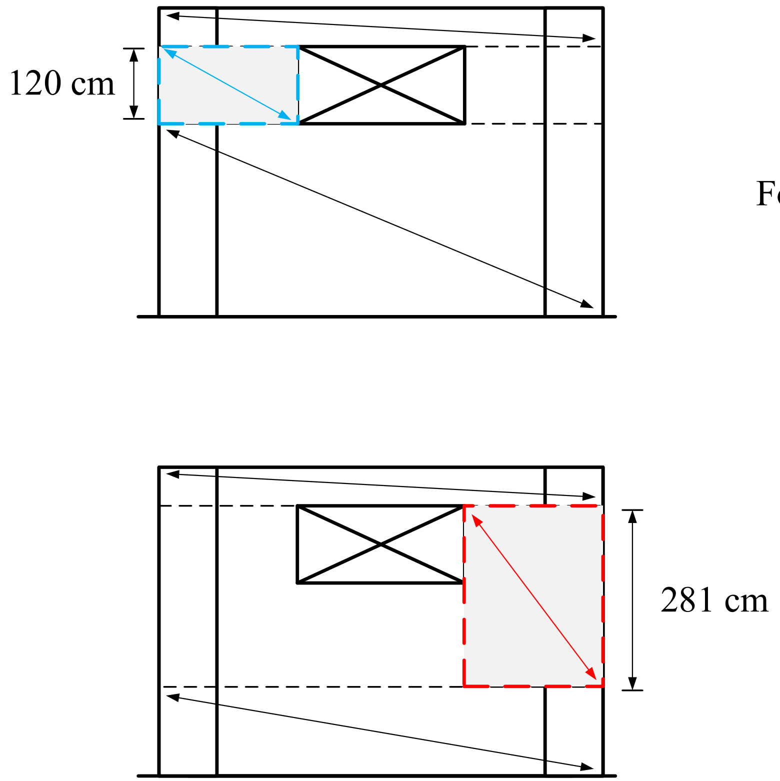
$$V_u = 1{,}140\ tf > \Phi V_{n,ACI} = 944\ tf \Rightarrow NG$$

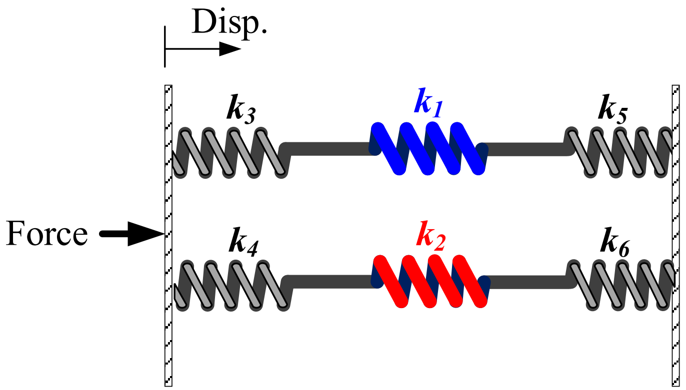


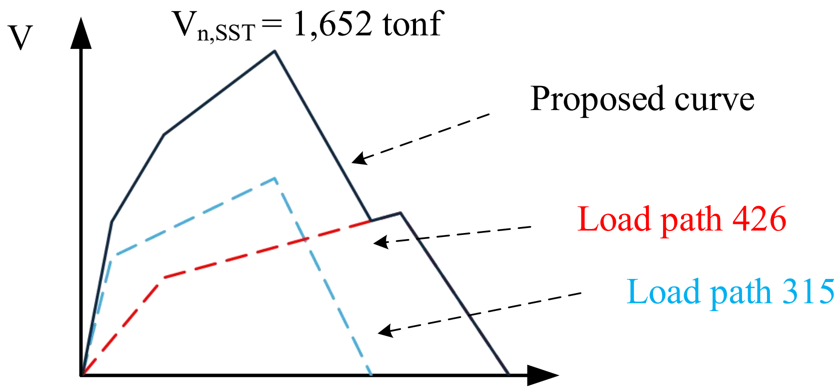




建議分析方法SST模型



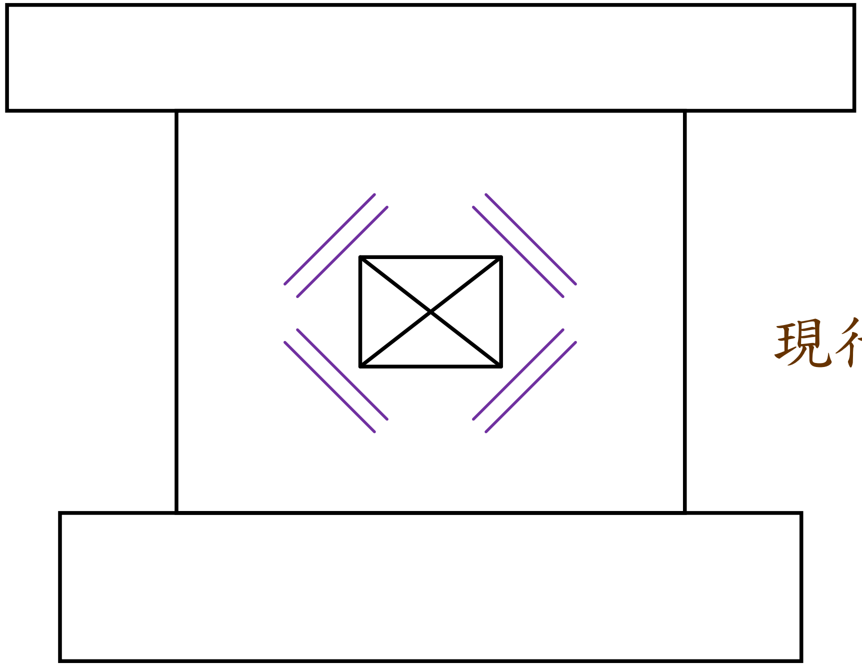




$$V_u = 1{,}140\ tf < \Phi V_{n,SST} = 1{,}239\ tf \Rightarrow ok$$

剪力牆開孔補強筋

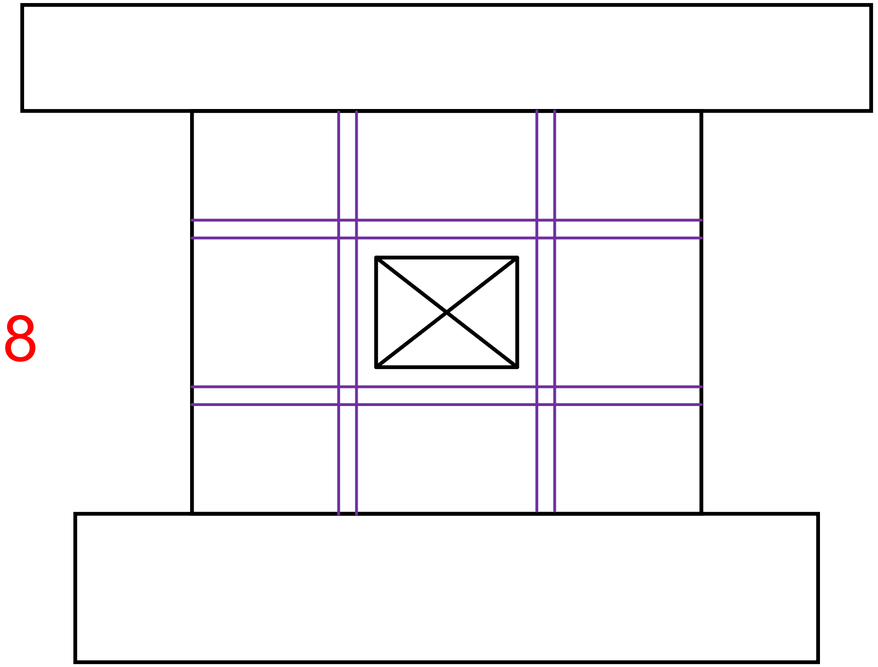
裂縫控制



斜向鋼筋

現行規定 ⟷ 日本 AIJ 2018 規定

力學需求與裂縫控制



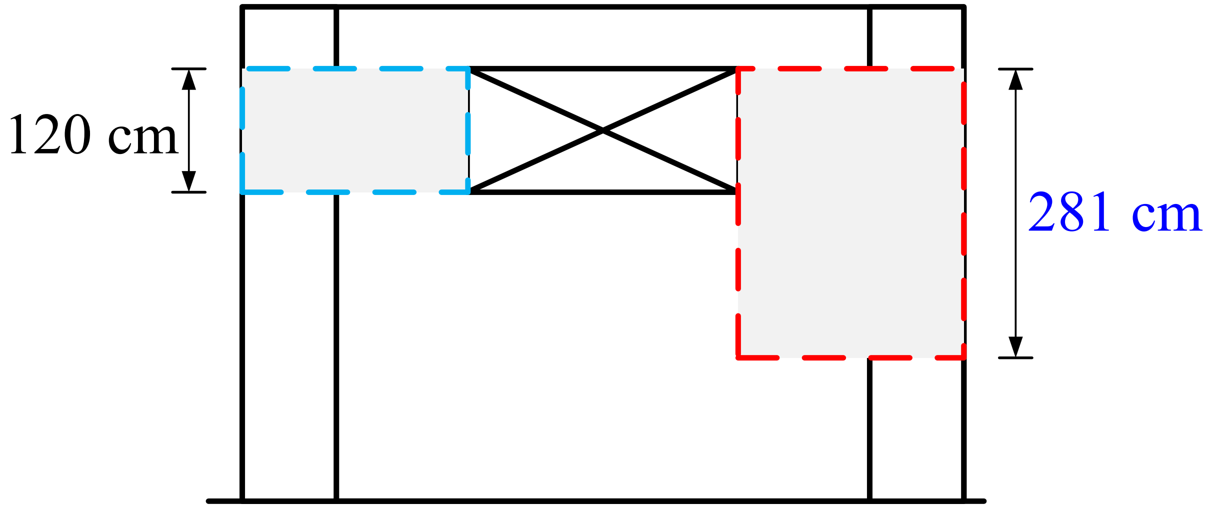
具端部錨定之水平與垂直鋼筋

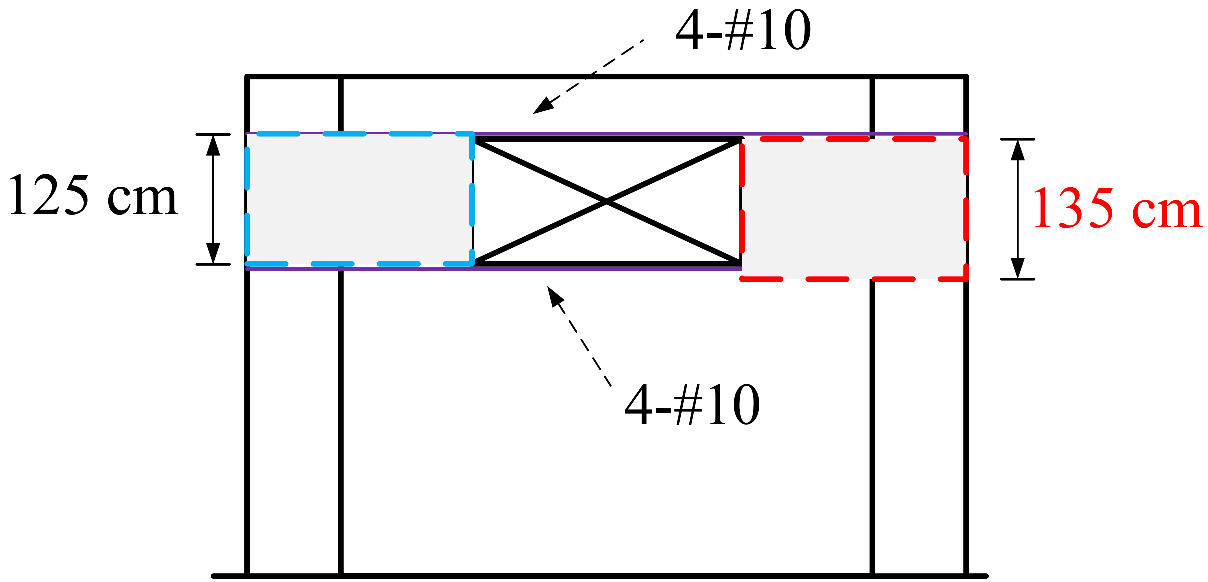
Architectural Institute of Japan (AIJ). 2018. “AIJ standard for structural calculation of reinforced concrete structures (revised),” Architectural Institute of Japan, Tokyo, Japan.

設計建議（詳設計案例）

未設置特殊水平鋼筋

設置特殊水平鋼筋，降低垂直牆段力學高度，增加剪力強度



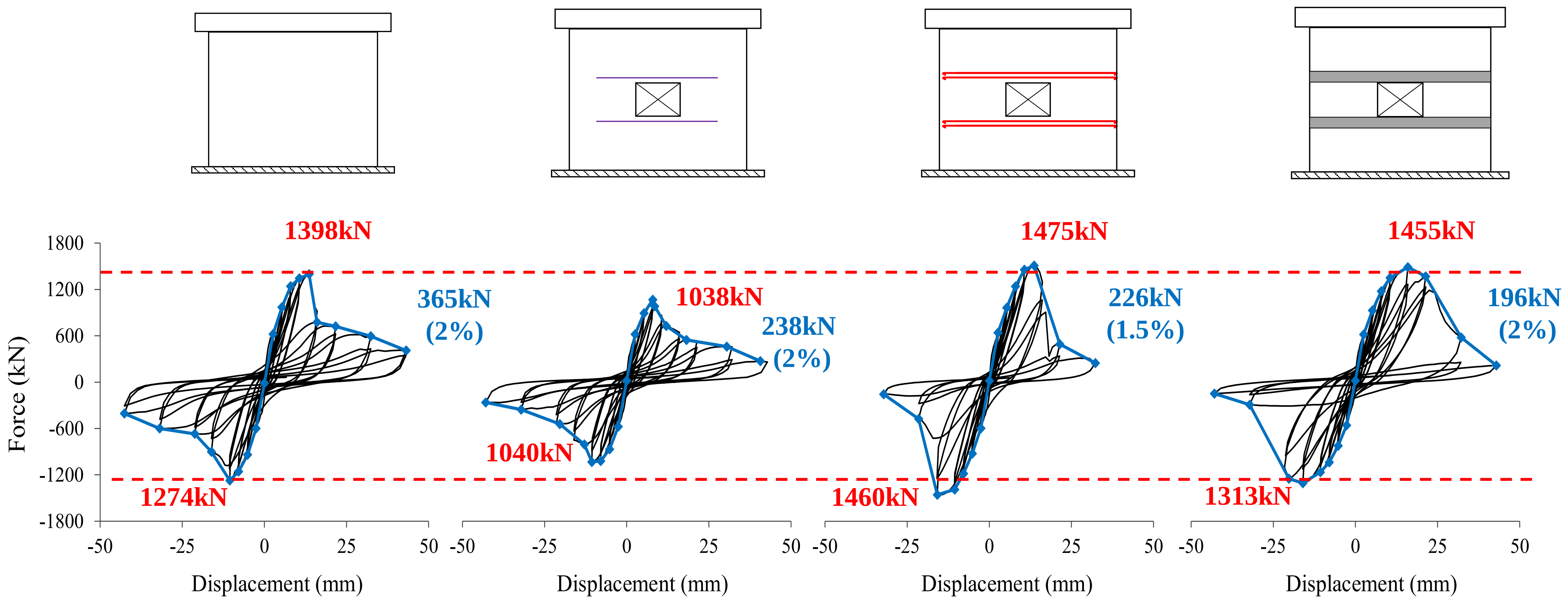


$V_n = 1,652\ tf$ → $V_n = 1,923\ tf$

增加 16%

結論

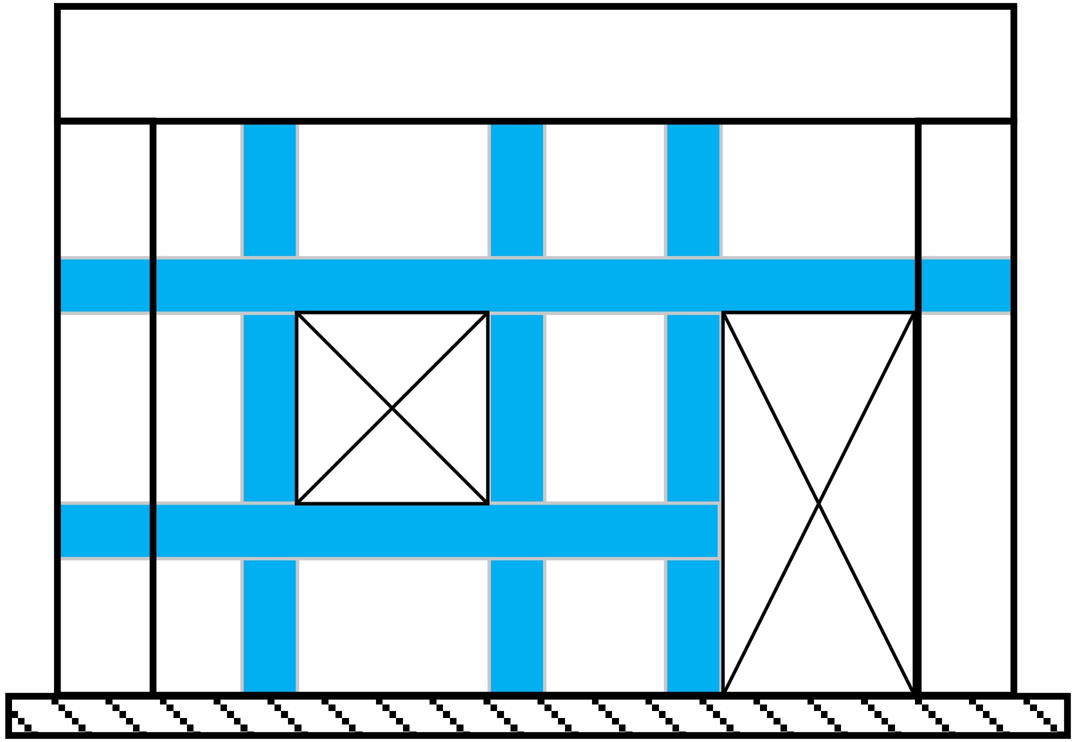
- 垂直牆段之力學高度$h_{w,m}$較低，則其剪力強度較高，此類似低矮型剪力牆之強度行為



- 建議方法可應用在既有結構之耐震評估與補強

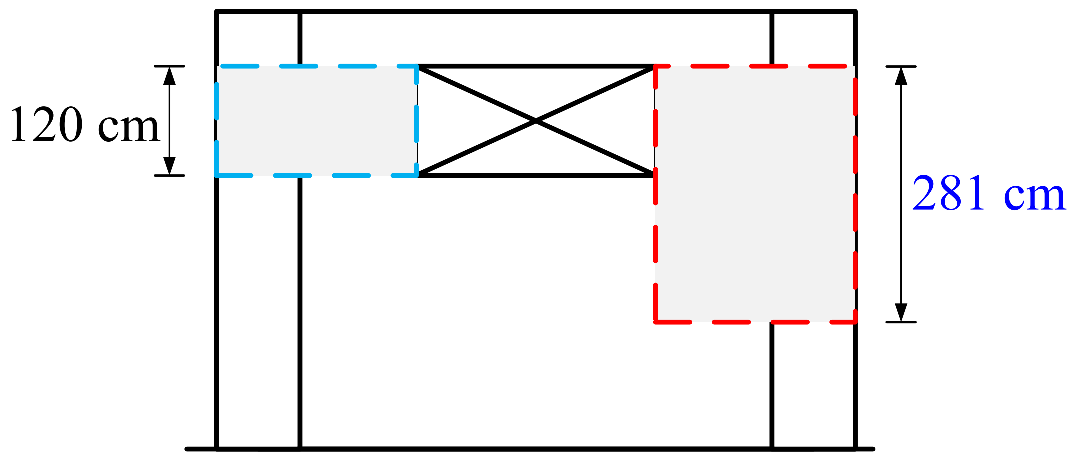
街屋開孔牆

FRP耐震補強



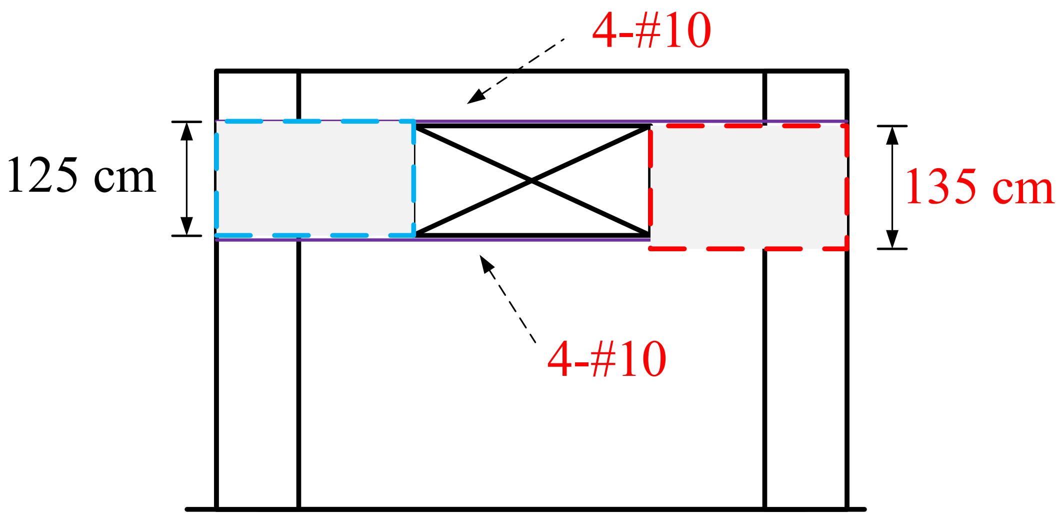
- 建議方法可應用在新建結構之含牆結構分析、開孔剪力牆設計、非結構開孔牆之破壞控制

未設置特殊水平鋼筋



$V_n = 1,652\ tf$

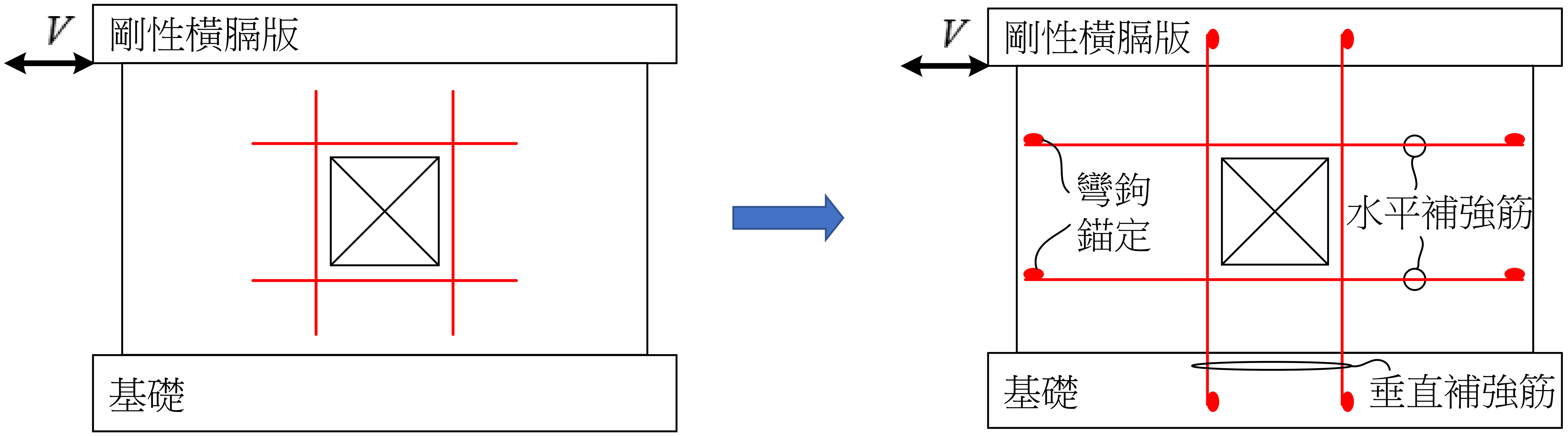
設置特殊水平鋼筋



$V_n = 1,923\ tf$

增加 16%

- 建議開孔補強筋應延伸至牆邊並作錨定，以提升開孔牆之力學性能



- 相關技術可詳NCREE技術報告(TEASPA V5)

林敏郎、邱聰智、鍾立來、涂耀賢、翁元滔、周德光、林皇佐、馬忠駿、魏銪廷、許嘉雯、曾俞傑、楊鈞翔、黃品絜、張季閎、鍾寬勳、黃世建、黃昭勳、歐昱辰、西崚汰、梶原浩一、藤原淳，(2025)，「臺灣結構耐震評估與補強技術手冊(TEASPA V5)」，國家地震工程研究中心報告，NCREE-2025-003，台北，275頁。

林敏郎、邱聰智、鍾立來、許嘉雯、魏銪廷、涂耀賢、翁元滔、周德光、馬忠駿、黃品絜、張季閎、許晉榮、宋隆洧、黃世建，(2025)，「臺灣結構耐震評估與補強程式操作手冊(TEASPA V5)」，國家地震工程研究中心報告，NCREE-2025-010，台北，400頁。

敬請指教！

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