2023.08.25「新版RC設計規範之精進內容與程式介紹」(高雄場)研討會

主辦單位：中華民國結構技師公會全國聯合會、台南市結構工程技師公會
、高雄市結構工程技師公會、台灣省結構工程技師公會

我國鋼筋混凝土梁柱接頭耐震設計細則之修訂

with contribution of NCREE, RuenTex, and YunTech

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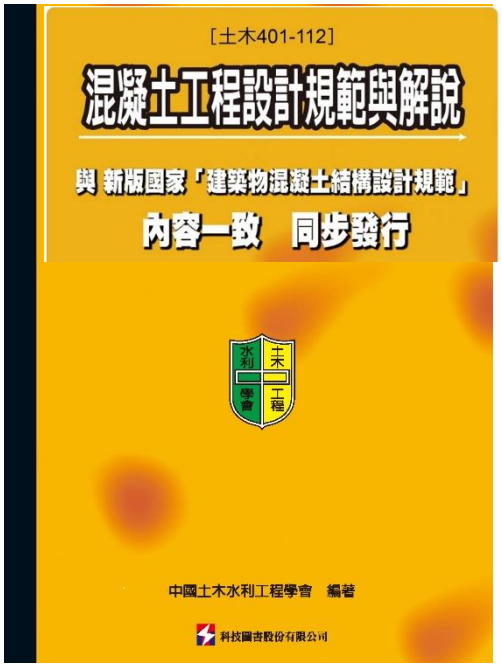
Supported by

NSTC 國家科學及技術委員會 National Science and Technology Council

YunTech 國立雲林科技大學 National Yunlin University of Science & Technology

簡報大綱

- 我國規範修訂之背景說明
- 接頭最小深度 (土木401-112　18.5.2)
- 接頭橫向鋼筋 (土木401-112　18.5.3)
- 接頭剪力強度 (土木401-112　18.5.4)
- 受拉錨定長度 (土木401-112　18.5.5)
- 結語



土木401-110年4月初版

土木401-112年8月
內政部審議後修訂版

中國土木水利工程學會 混凝土工程委員會

混凝土工程設計規範與解說

(土木 401-112)

引用本規範條文為工程契約文件時，應充分瞭解工程與本規範之適用性，針對工程特性妥訂特別條款

設計規範編審小組

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	陳 式 毅		陳 君 弢		陳 清 泉
	彭 康 瑜		黃 然		黃 世 建
	黃 炳 勳		詹 文 宗		詹 穎 雯
	廖 文 正		廖 肇 昌		劉 光 晏
召集人	歐 昱 辰		蔣 啟 恆		鄭 敏 元
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中華民國一一二年八月

目 錄

內政部審議後修訂部頒版112
年下旬公告113.1.1起實施

背景

- 我國RC設計規範以美國ACI 318規範為藍本更新
- 大樓慣用特殊抗彎構架SMF或二元系統
- 地處強震區、地質條件不佳、地震力效應大
- 建築師和消費者偏好較結實、緊湊的梁柱斷面

= RC構造鋼筋密度高導致組裝立和澆置困難，品質=?

美日紐陸續開放490~550 MPa等級鋼筋作為耐震結構用主要鋼筋

規範	耐震構架抵抗軸力、彎矩之縱向鋼筋設計降伏強度最大值 f_y
日本AIJ規範 (2010)	490 MPa (更高強度要特別認可)
紐西蘭NZS規範 (2006)	500 MPa
美國ACI規範 (2019)	550 MPa (2014版前為420 MPa)
我國新版混凝土結構設計規範 (2021→2023)	550 MPa (現行規範為420 MPa)

註：此表是耐震抗彎矩構架梁柱縱向主筋之最高設計降伏強度等級橫向圍束箍筋之降伏強度等級為690 MPa。

簡報大綱

- 我國規範修訂之背景說明
- 接頭最小深度 (土木401-112　18.5.2)
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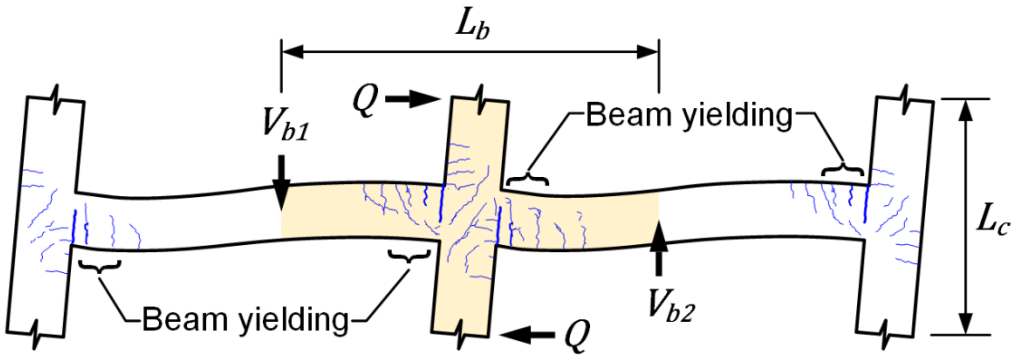
- 結語

特殊抗彎矩構架之接頭

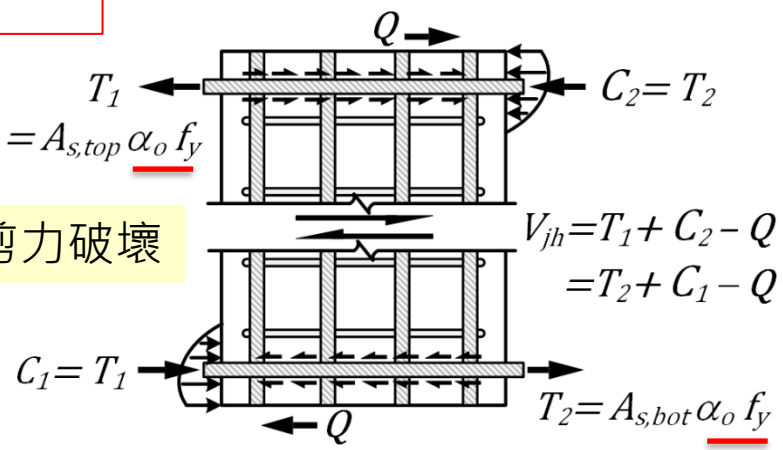
M.C.E. 作用下

@18.5.2.1
梁縱向鋼筋於接頭面之力，應假設撓曲拉力鋼筋應力為 $1.25f_y$ 計算之。



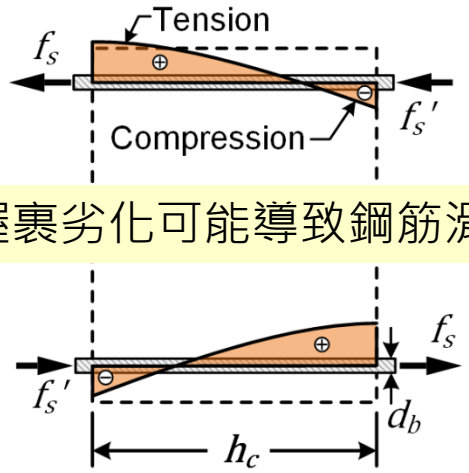
(a) Isolated beam-column unit of a moment-resisting frame

高剪力可能導致剪力破壞



(b) Horizontal forces acting on an interior joint

握裹劣化可能導致鋼筋滑移



(c) Stresses along beam bars

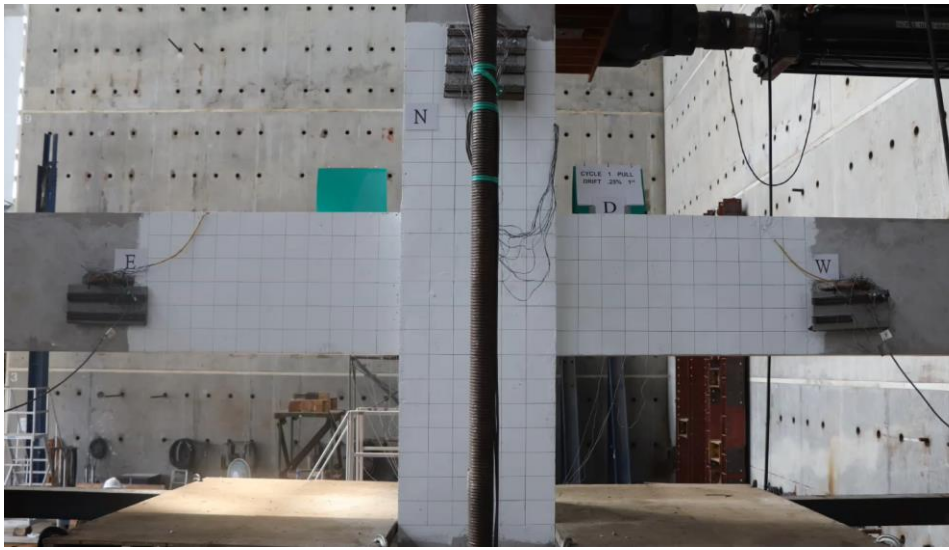
Lee, H.-J.; Lequesne, R.; Lepage, A.; Lin, J.-X.; Wang, J.-C.; Yin, S. Y.-L., Minimum Joint Depth for Special Moment Frames with High-Strength Reinforcement. *ACI Structural Journal* **2023,** 120, (1), 225-239.

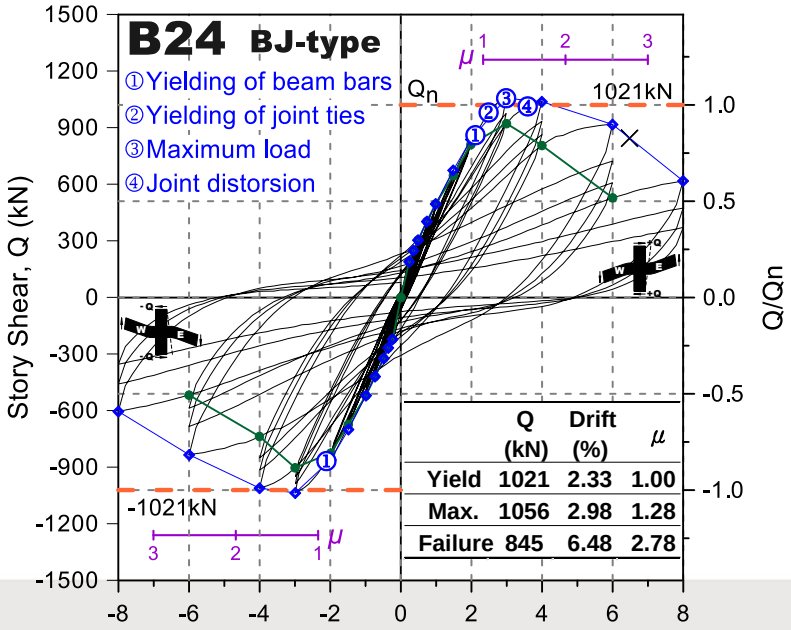
Joint damage in prior earthquakes

1999 Chi-Chi earthquake, Touliu

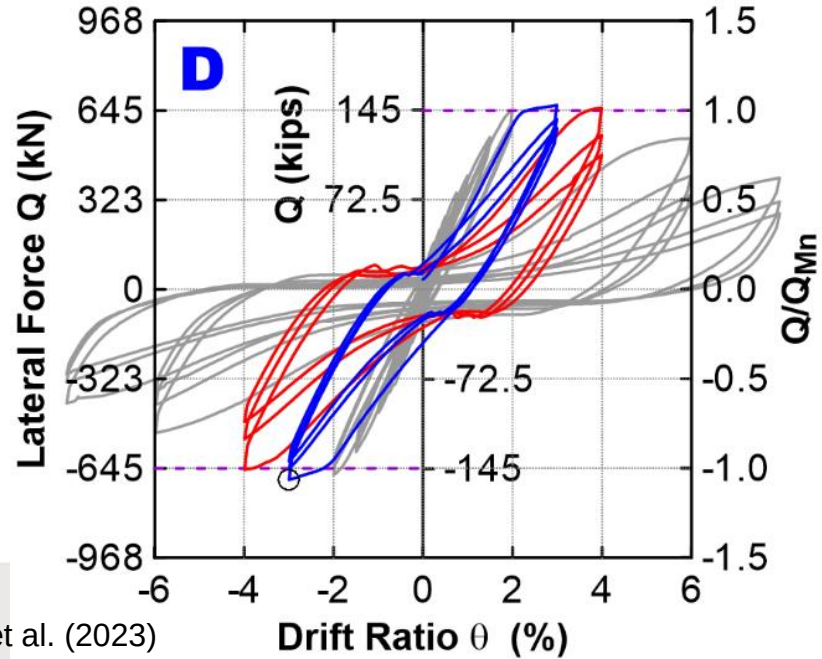
Collapse

Beam yielding with Joint Shear or Bond Failure



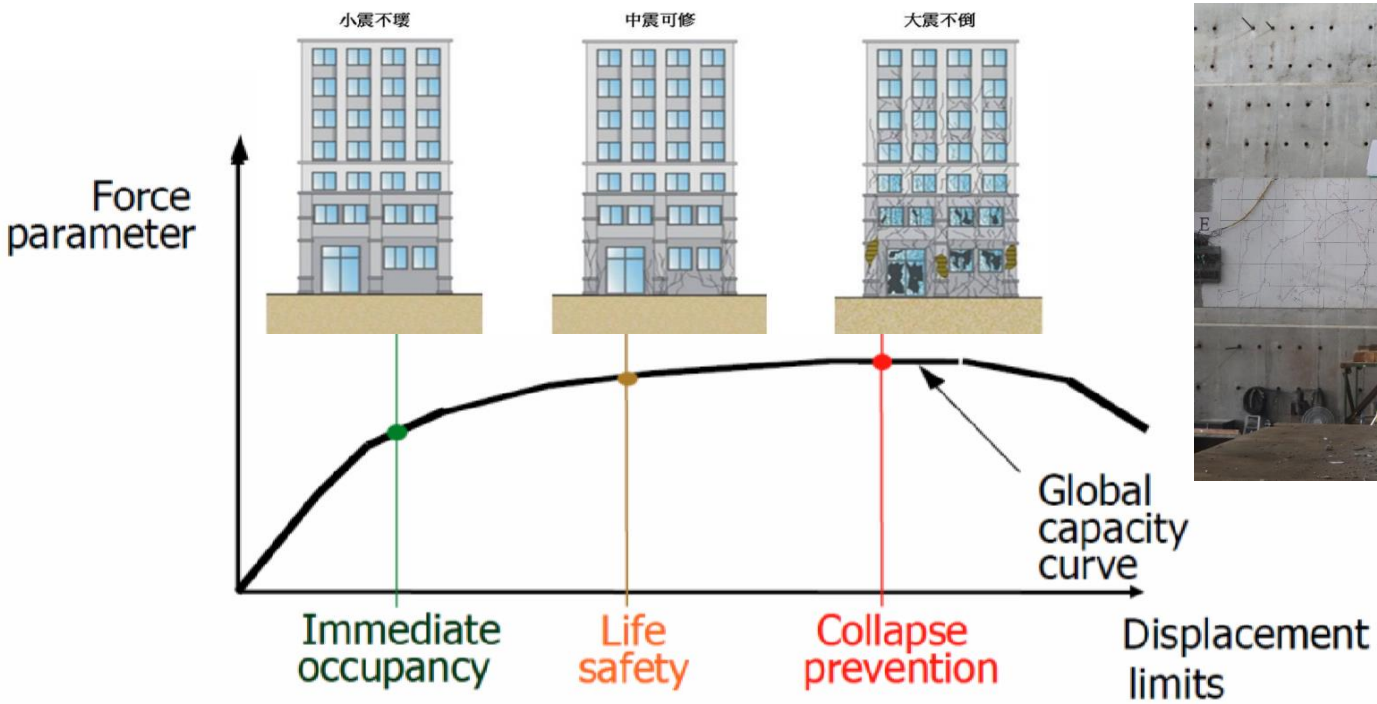


李宏仁等人(2014)

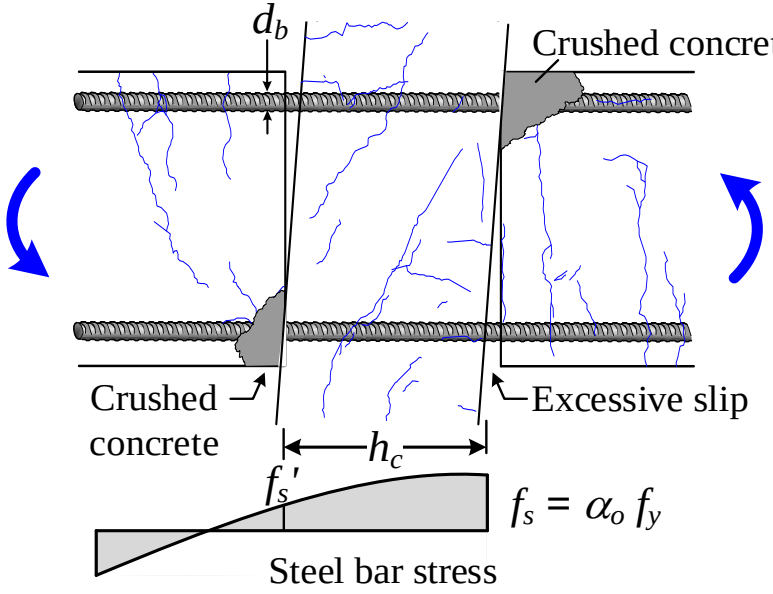


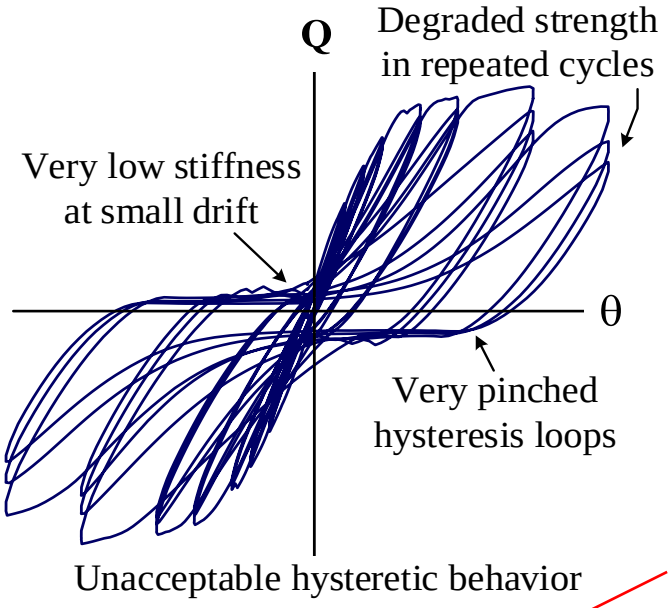
Lee et al. (2023)

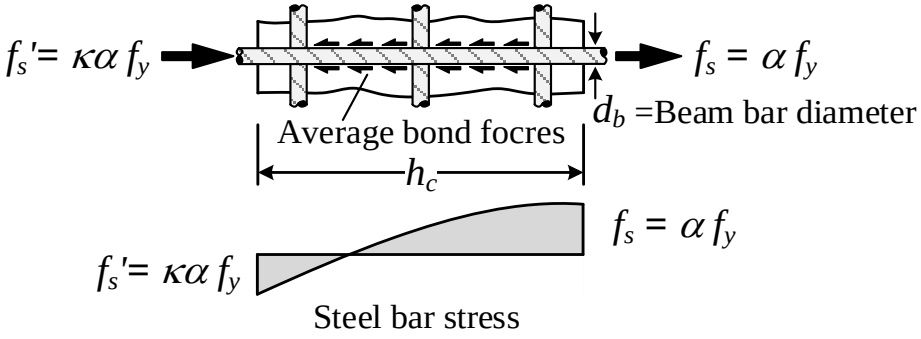
應在2%, 3%, or 4% drift 內避免握裹破壞否?



梁主筋貫穿接頭握裹破壞







Average bond strength

$$\pi d_b h_c * \alpha_p u_b \geq \frac{\pi d_b^2}{4}\left(\alpha f_y + \kappa\alpha f_y\right)$$

移項

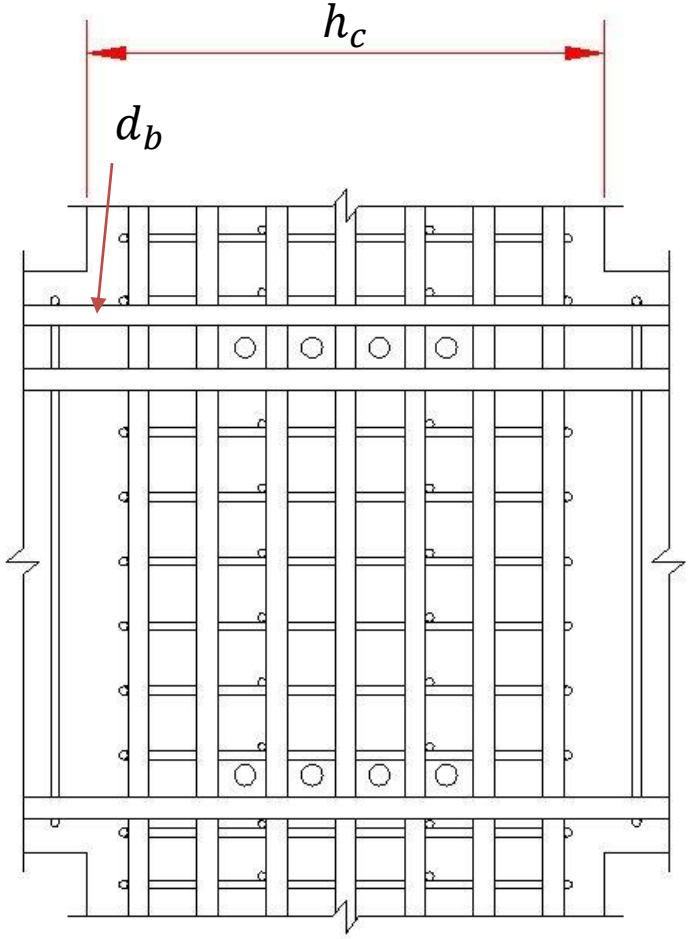
$$h_c \geq \frac{(\alpha + \kappa\alpha)}{4\alpha_p u_b} f_y d_b \approx \alpha_3 \frac{f_y}{\sqrt{f_c'}} d_b$$

係數與混凝土抗張強度、圍束條件有關

SMF梁主筋貫穿接頭之最小柱深

18.5.2.3 當縱向梁鋼筋貫穿梁柱接頭時，若使用常重混凝土，則平行於梁縱向鋼筋方向之接頭深度h應至少為...

$$h_c \geq max \begin{cases} 20d_b(SD420W), 23d_b(SD490W), 26d_b(SD550W) \\ h_b/2 \end{cases}$$



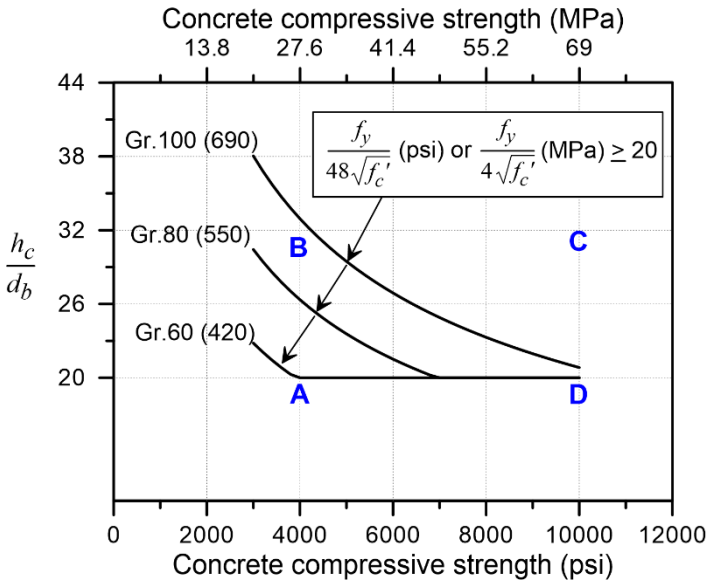
ACI code criterion

f_y psi	f'_c psi	$h_c = \frac{1}{48}\frac{f_y}{\sqrt{f'_c}}d_b$
60,000	4000	20
70,000	4000	23
80,000	4000	26

Since 1992, ACI Code require SMF joint with $h_c \geq 20d_b$
This requirement is aimed at limiting bar slip in joints subjected to interstory drift demands of up to 3% without requiring excessively large and uneconomical columns, but it does not prevent bar slip.

此一準則當初是依據 Zhu and Jirsa (1983) 整理的 60 ksi 級鋼筋和普通強度混凝土接頭試驗資料庫的。
並未考慮軸力及高強度混凝土之效益。

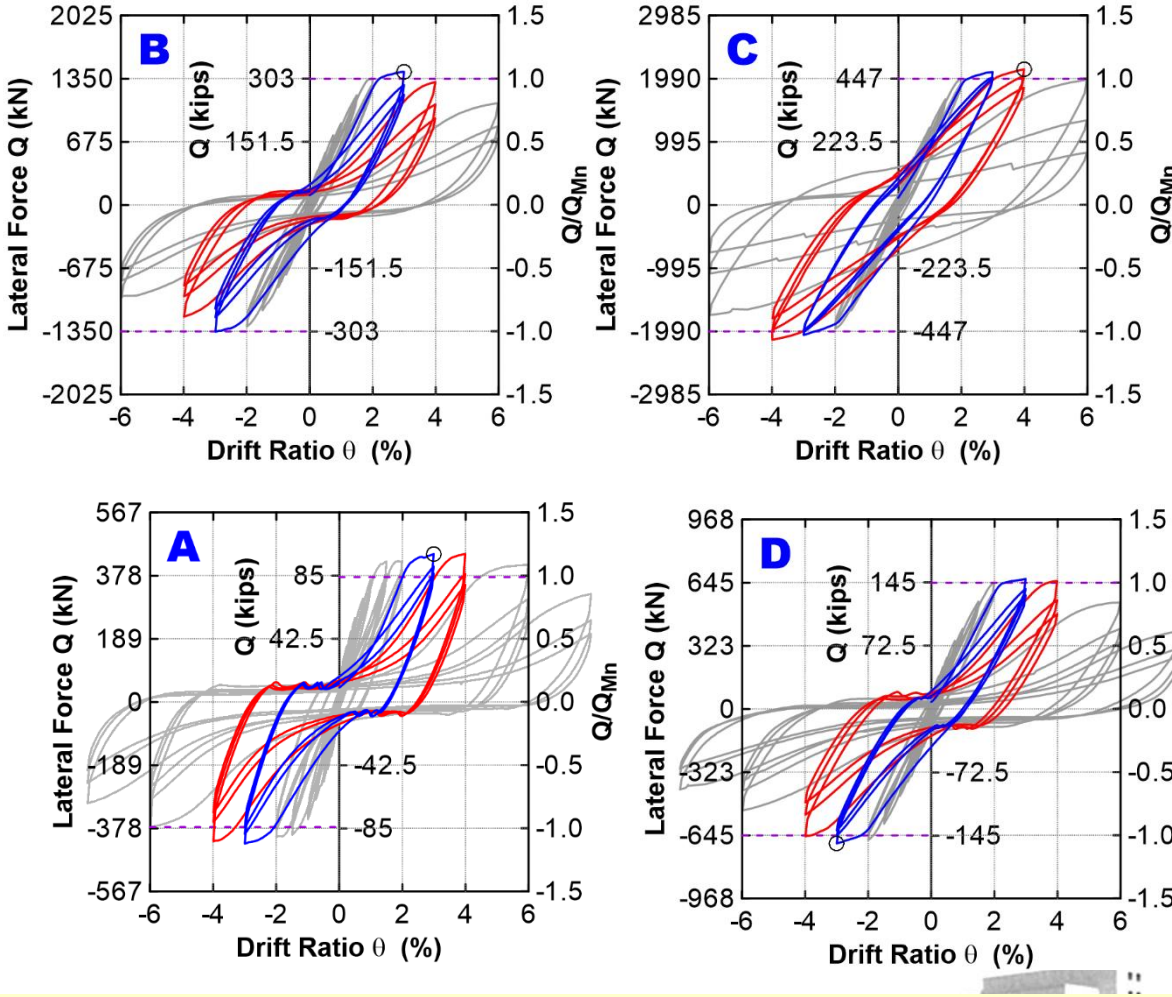
台美合作最新實驗研究之建議



Eq.3

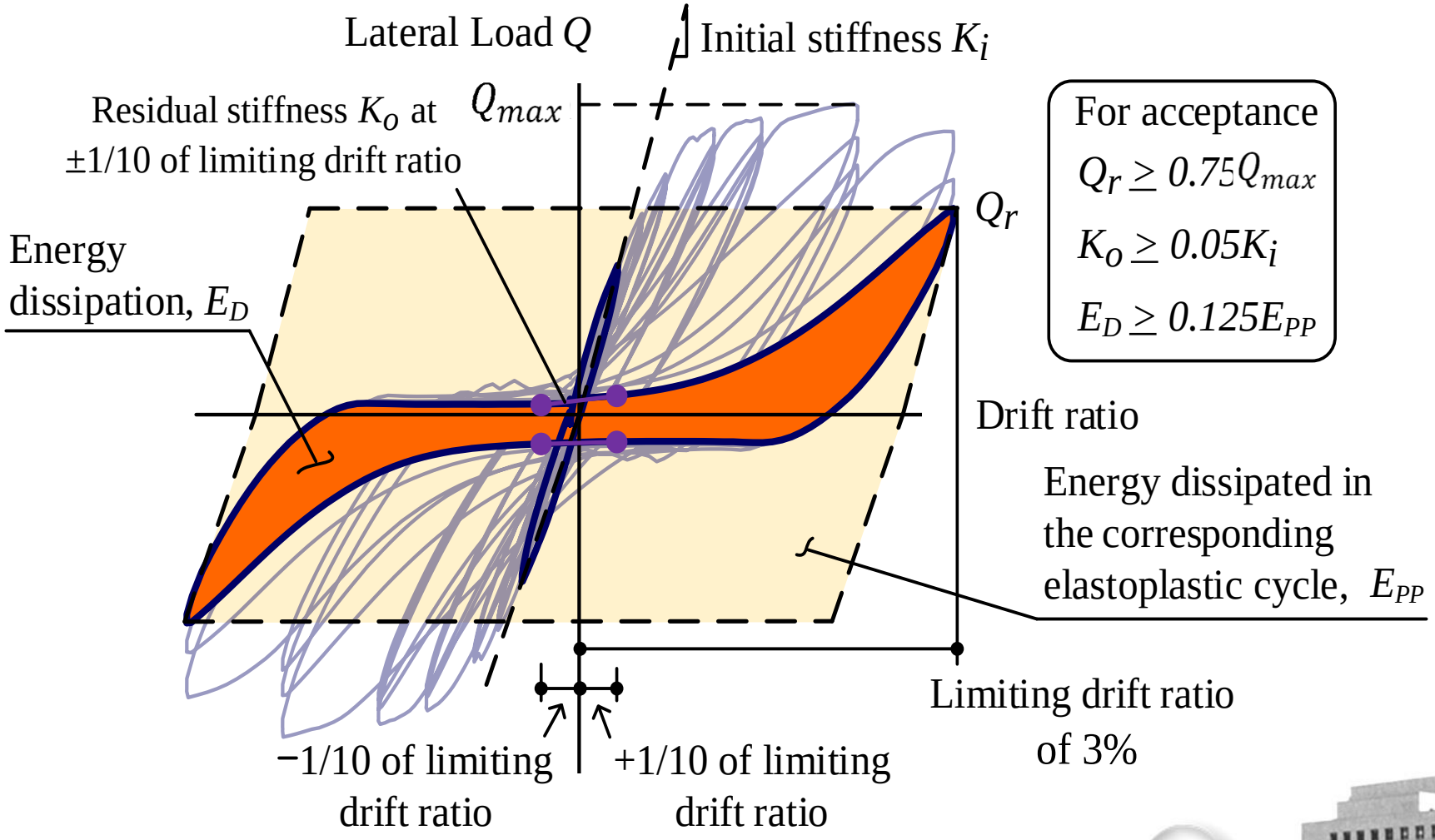
$$h_c = \frac{1}{48}\frac{f_y}{\sqrt{f_c'}}d_b \quad \text{(psi unit)}$$

不能排除3% drift cycle 出現握裹滑移，但何種程度的損壞可接受呢?



Lee, H.-J.; Lequesne, R.; Lepage, A.; Lin, J.-X.; Wang, J.-C.; Yin, S. Y.-L., Minimum Joint Depth for Special Moment Frames with High-Strength Reinforcement. *ACI Structural Journal* **2023,** 120, (1), 225-239.

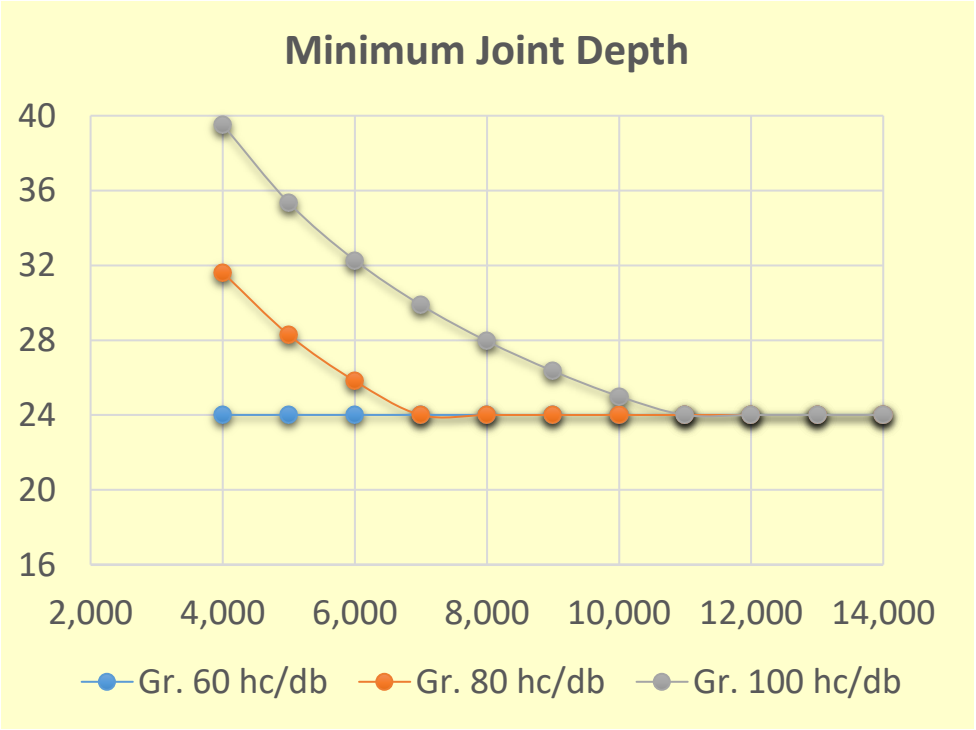
Acceptance criteria for testing components of special moment frames in ACI 374.1-05

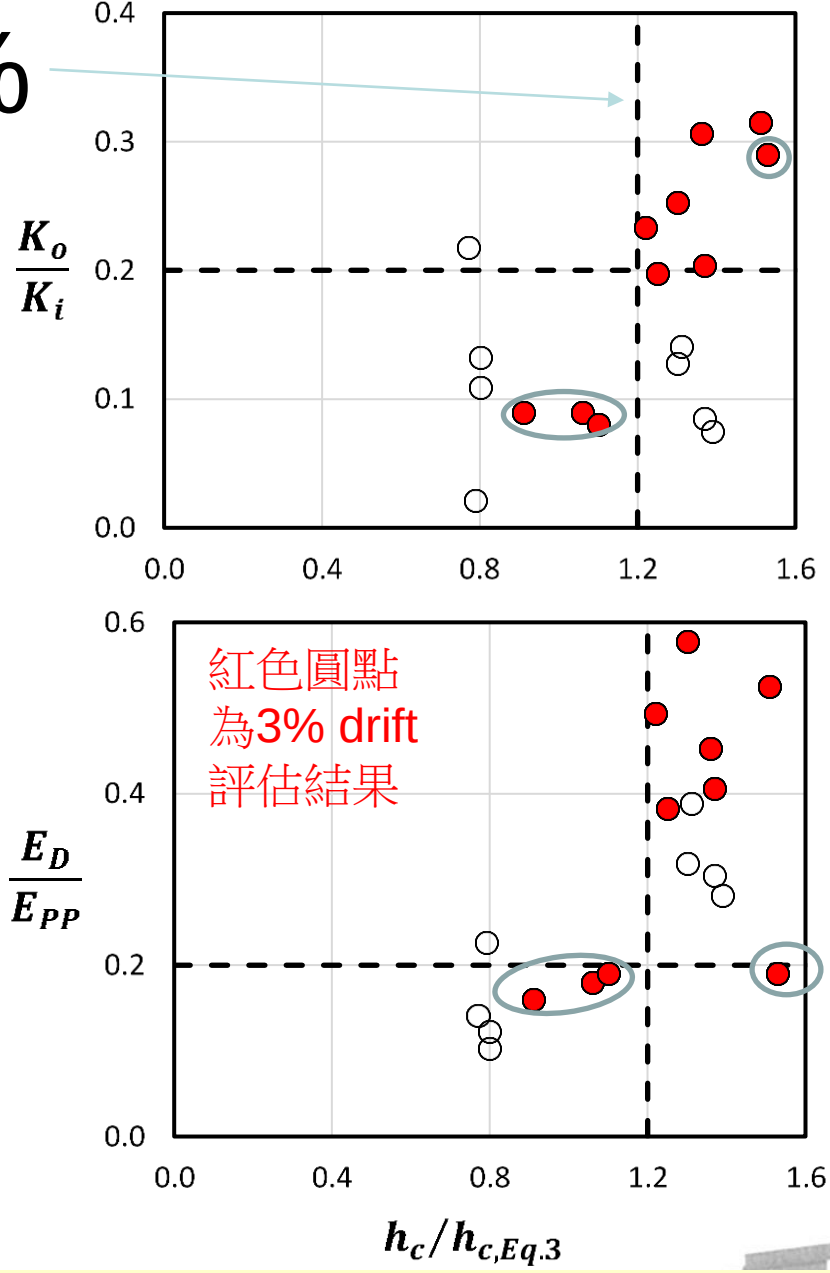


研究建議提高20%

$$h_c = \frac{1}{40}\frac{f_y}{\sqrt{f_c'}} d_b \geq 24 d_b \quad \text{(psi unit)}$$

$$h_c = \frac{1}{10.6}\frac{f_y}{\sqrt{f_c'}} d_b \geq 24 d_b \quad \text{(kgf/cm}^2\text{ unit)}$$





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特殊抗彎矩構架之接頭橫向鋼筋(18.5.3)

18.5.3 橫向鋼筋

18.5.3.1 除第18.5.3.2節之規定所允許外，接頭在最深構入梁深度h之範圍內橫向鋼筋須符合第18.4.5.2節(a)至(e)之規定、第18.4.5.3節、表18.4.5.4(a)(b)或(d)(e)、與第18.4.5.7節之規定。

參閱下三頁投影片，修正部分細則與ACI 318規範不同

ACI 318-19

18.8.3 *Transverse reinforcement*

18.8.3.1 Joint transverse reinforcement shall satisfy 18.7.5.2, 18.7.5.3, 18.7.5.4, and 18.7.5.7, except as permitted in 18.8.3.2.

用量和細節直接比照柱端橫向鋼筋規定，除非四面有橫向梁圍束作用時，用量得減半

SMF 柱橫向鋼筋

@18.4.5.1

$$\ell_o \geq \max \begin{cases} C_1 \\ H_n/6 \\ 45\ \text{cm} \end{cases}$$

@18.4.5.3

$$S_{1,3} \leq \min \begin{cases} C_2/4 \\ 6d_b \text{ or } 5d_b\ (\text{SD550W}) \\ s_o = 10 + \dfrac{(35 - h_x)}{3} \leq 15\ \text{cm} \end{cases}$$

@18.4.5.4

$$\frac{A_{sh}}{sb_c} \geq \max \begin{cases} 0.09\dfrac{f'_c}{f_{yt}} \\ 0.3\dfrac{f'_c}{f_{yt}}\left(\dfrac{A_g}{A_{ch}} - 1\right) \\ 0.2k_f k_n \dfrac{P_u}{f_{yt}A_{ch}} \end{cases}$$

@18.4.5.4(c)

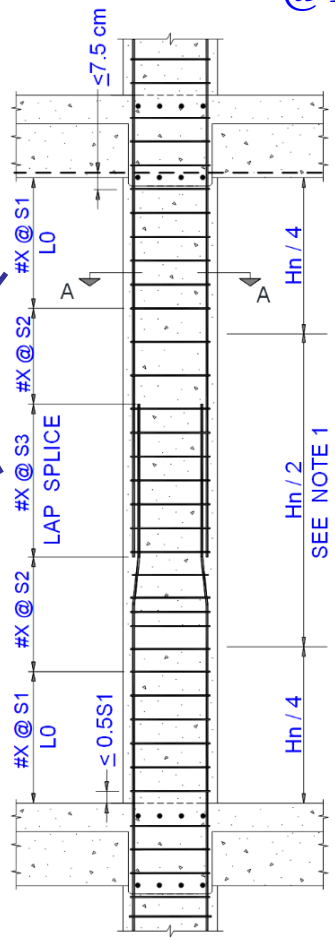
$$k_f = \frac{f'_c}{1750} + 0.6 \geq 1.0 \qquad k_n = \frac{n_l}{n_l - 2}$$

where $f_{yt} \leq 7000\ kgf/cm^2$

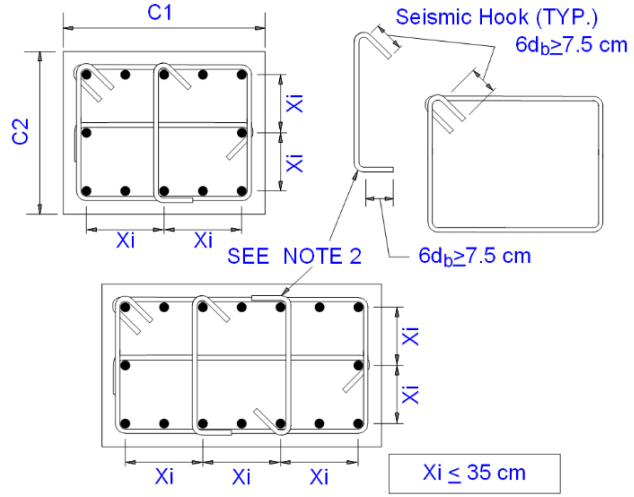
@18.4.5.5

$$S_2 \leq \min \begin{cases} 6d_b \text{ or } 5d_b\ (\text{SD550W}) \\ 15\ cm \end{cases}$$

@18.4.5.2(a)~(e)

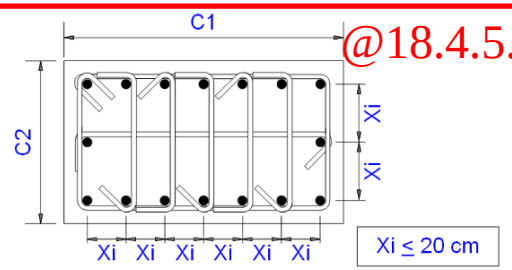


ELEVATION



TIE ARRANGEMENT – TYPE 1

@18.4.5.2(f)



TIE ARRANGEMENT – TYPE 2

SECTION A-A

SPECIAL MOMENT FRAME COLUMN DETAIL

Crossties with one seismic hook and one 90-degree hook are still permitted for $f'_c > 700\ kgf/cm^2$ or $P_u > 0.3A_g f'_c$

塑鉸區橫向鋼筋細節(18.4.5.2)

橫向鋼筋須符合(a)至(f)之規定：

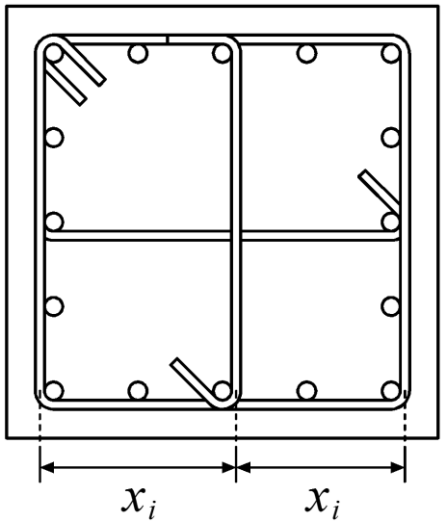
(a) 略

…

(f) 在P_u>0.3$A_g f'_c$ 或f'_c>700 kgf/cm^{2}之直線型閉合箍筋柱，沿柱核心周邊之每一縱向鋼筋或束筋應有閉合箍筋轉角或繫筋彎轉段所提供之側向支撐，且h_x之值應不超過20 cm。P_u應為含E之因數化載重組合所得之最大應力。

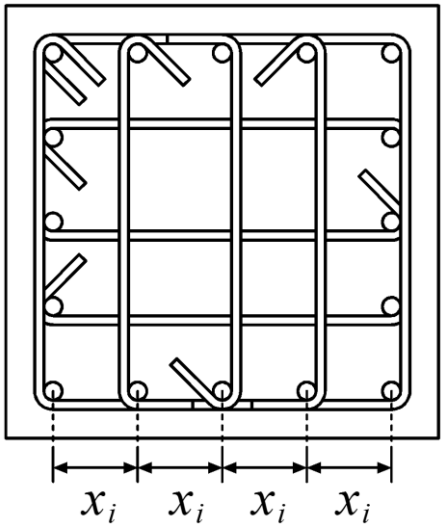
(f)不適用接頭

土木401-100規範



$x_i \le 35$ cm

土木401-110規範



$x_i \le 20$ cm

ACI 318規範對較高軸力或高強度混凝土之柱，採取較嚴格的規定，不僅增加柱箍筋量，並要求以耐震彎鉤圍繞每一根縱向鋼筋。此規定源自於柱塑鉸變形能力至少0.03弧度之性能要求。然而梁柱接頭並無0.03弧度塑性轉角之需求，且以我國施工條件，梁柱接頭難執行ACI 318-19較嚴格之規定。建議回到我國現有規範標準。透過實驗及資料庫驗證，梁柱接頭毋須再提高箍筋量。

塑鉸區橫向鋼筋用量(18.4.5.4)

表18.4.5.4

橫向鋼筋	條件	適用表達式		
直線型閉合箍筋之 A_{sh}/sb_c	$P_u \le 0.3A_g f_c'$ 與 $f_c' \le 700\ \text{kgf/cm}^2$	(a)與(b)之較大值	$0.3\left(\frac{A_g}{A_{ch}}-1\right)\frac{f_c'}{f_{yt}}$	(a)
			$0.09\frac{f_c'}{f_{yt}}$	(b)
	$P_u > 0.3A_g f_c'$ 或 $f_c' > 700\ \text{kgf/cm}^2$	(a)、(b)與(c)之最大值	$0.2k_f k_n \frac{P_u}{f_{yt}A_{ch}}$	(c)
螺箍筋或圓形閉合箍筋 ρ_s	$P_u \le 0.3A_g f_c'$ 與 $f_c' \le 700\ \text{kgf/cm}^2$	(d)與(e)之較大值	$0.45\left(\frac{A_g}{A_{ch}}-1\right)\frac{f_c'}{f_{yt}}$	(d)
			$0.12\frac{f_c'}{f_{yt}}$	(e)
	$P_u > 0.3A_g f_c'$ 或 $f_c' > 700\ \text{kgf/cm}^2$	(d)、(e)與(f)之最大值	$0.35k_f \frac{P_u}{f_{yt}A_{ch}}$	(f)

(c)(f)不適用接頭

混凝土強度係數k_f $\quad k_f = \frac{f_c'}{1750} + 0.6 \ge 1.0$

圍束有效係數k_n $\quad k_n = \frac{n_l}{n_l - 2}$ $\quad n_l =$ 縱向鋼筋受閉合箍筋轉角或耐震彎鉤側向支撐之數目

後面有實驗證明

橫向鋼筋用量與間距可放寬之條件

	土木401-100 規範(15.6.2.2)	土木 401-110 規範(18.5.3.2)
橫向鋼筋用量減半之條件	接頭四面皆有構材構入，且每一構材寬度最少為柱寬度之3/4，則柱在接頭處最淺構材之深度範圍內，可配置較少之橫向鋼筋，惟其量至少應為第15.5.4.1節規定量之半。上述之深度範圍內，第15.5.4.2節規定之間距得增至15 cm。	若構入接頭的梁連續或符合15.2.7節，且梁寬度至少為柱寬度之3/4，則在該構入梁接頭兩側較淺梁深度*h*之範圍內依表18.4.5.4 (a)(b)或(d)(e)規定平行該構入梁方向之鋼筋量應可減半，且依第18.4.5.3節規定之間距允許增至15 cm 。(18.5.3.2)

接頭一雙對面受圍束時，經實驗證明平行該梁方向之箍筋量可減半，不減損其耐震性能。建議將箍筋量減半分成XY兩方向獨立考慮，這是基於我方之研究成果。

ACI 318-19

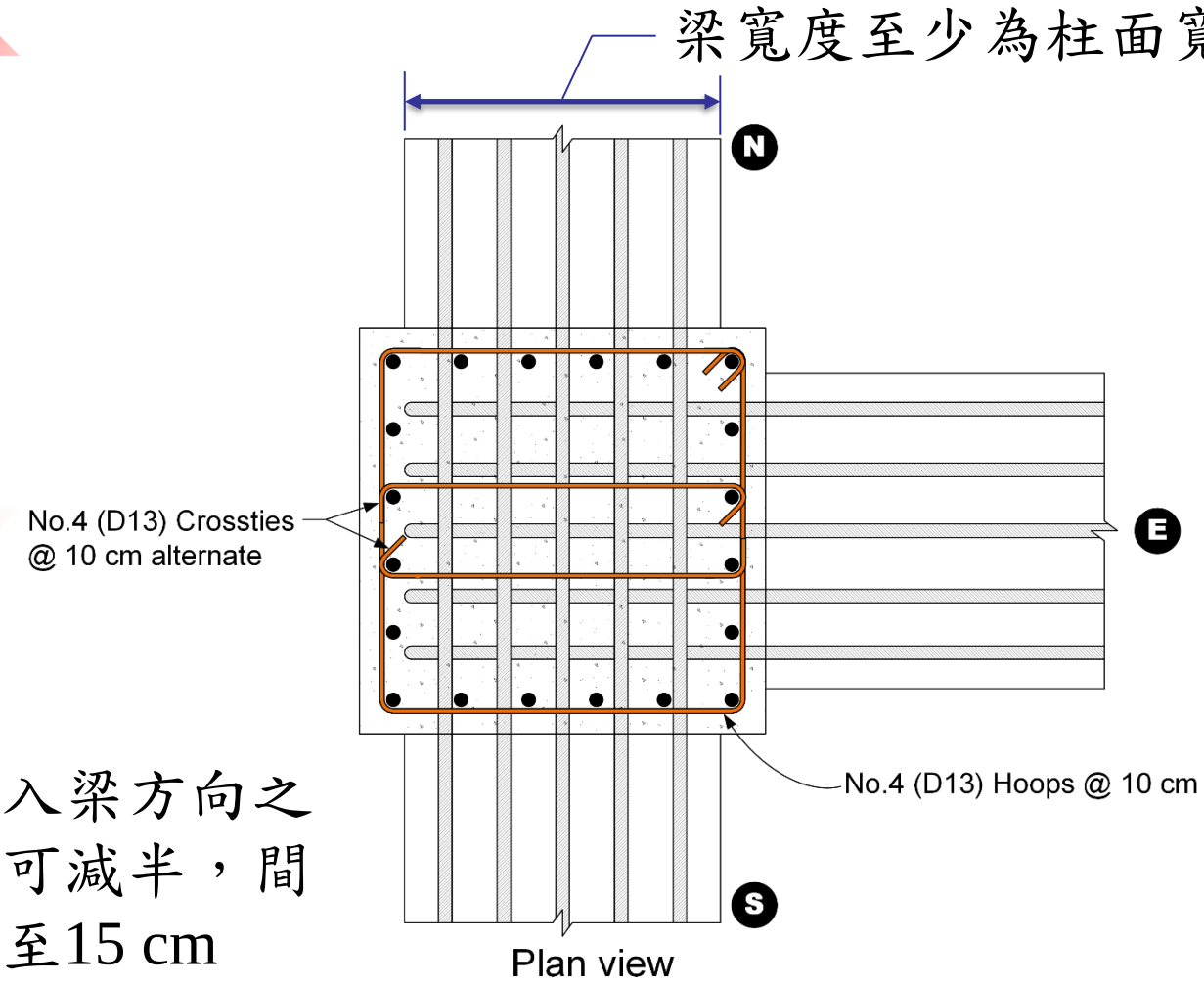
18.8.3 *Transverse reinforcement*

…

18.8.3.2 Where beams frame into all four sides of the joint and where each beam width is at least three-fourths the column width, the amount of reinforcement required by 18.7.5.4 shall be permitted to be reduced by one-half, and the spacing required by 18.7.5.3 shall be permitted to be increased to 6 in. within the overall depth h of the shallowest framing beam.

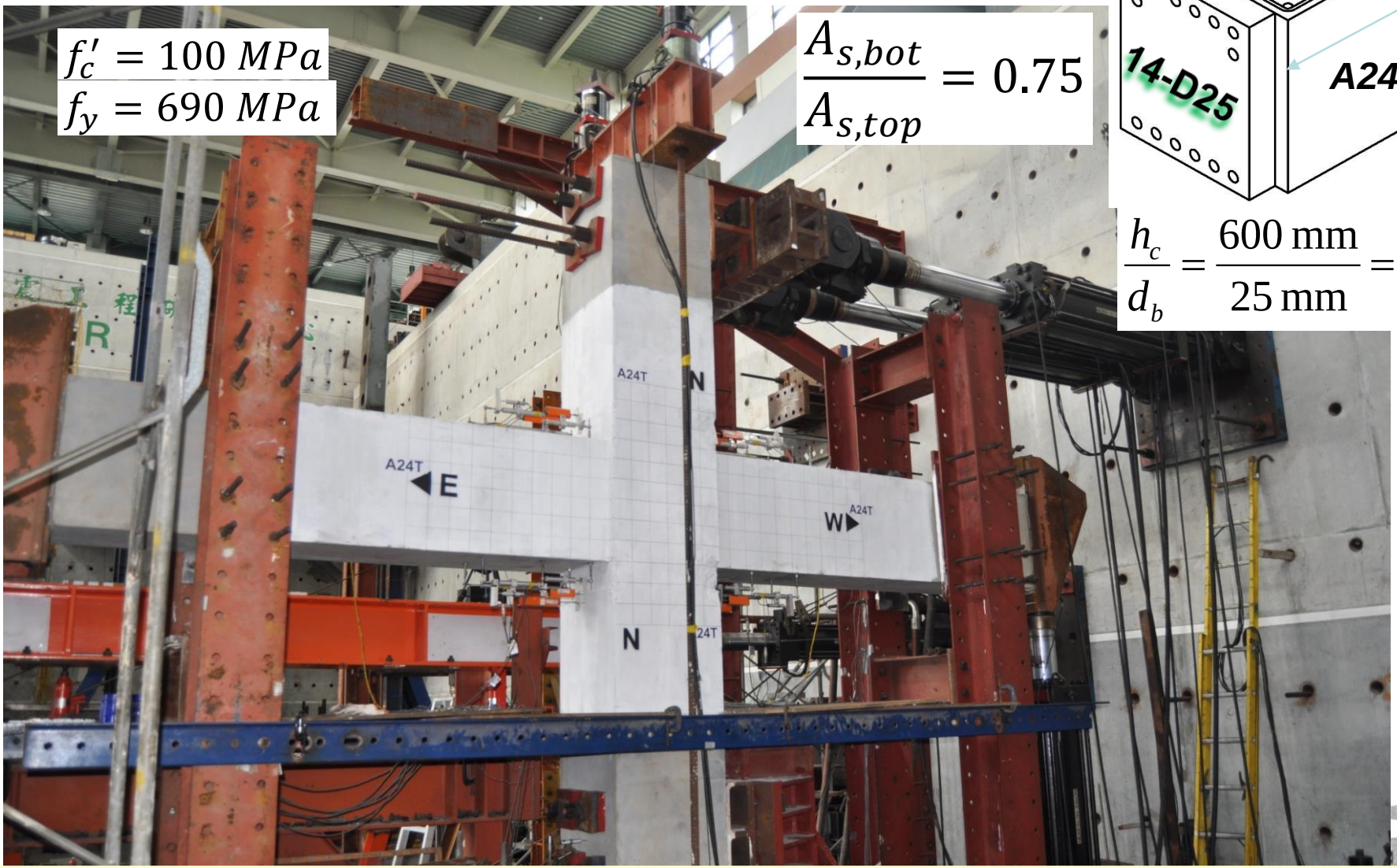
橫向鋼筋用量減半之示意圖

構入接頭的梁連續



平行該構入梁方向之鋼筋量應可減半，間距可放寬至15 cm

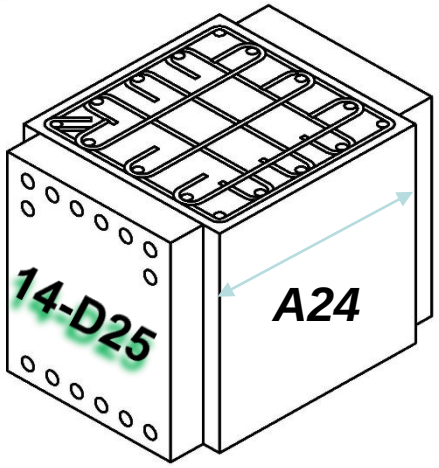
Seismic Testing for Interior Beam-Column Joints



$f_c' = 100\ MPa$

$f_y = 690\ MPa$

$$\frac{A_{s,bot}}{A_{s,top}} = 0.75$$



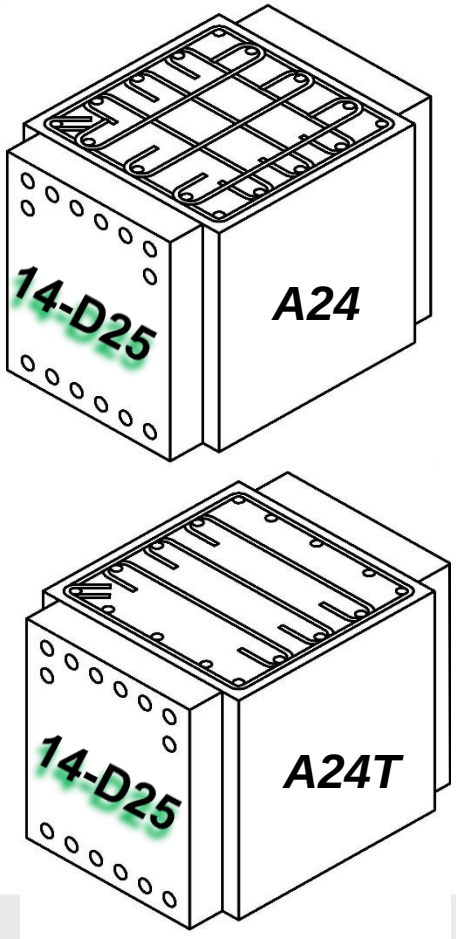
$$\frac{h_c}{d_b} = \frac{600\ \text{mm}}{25\ \text{mm}} = 24$$

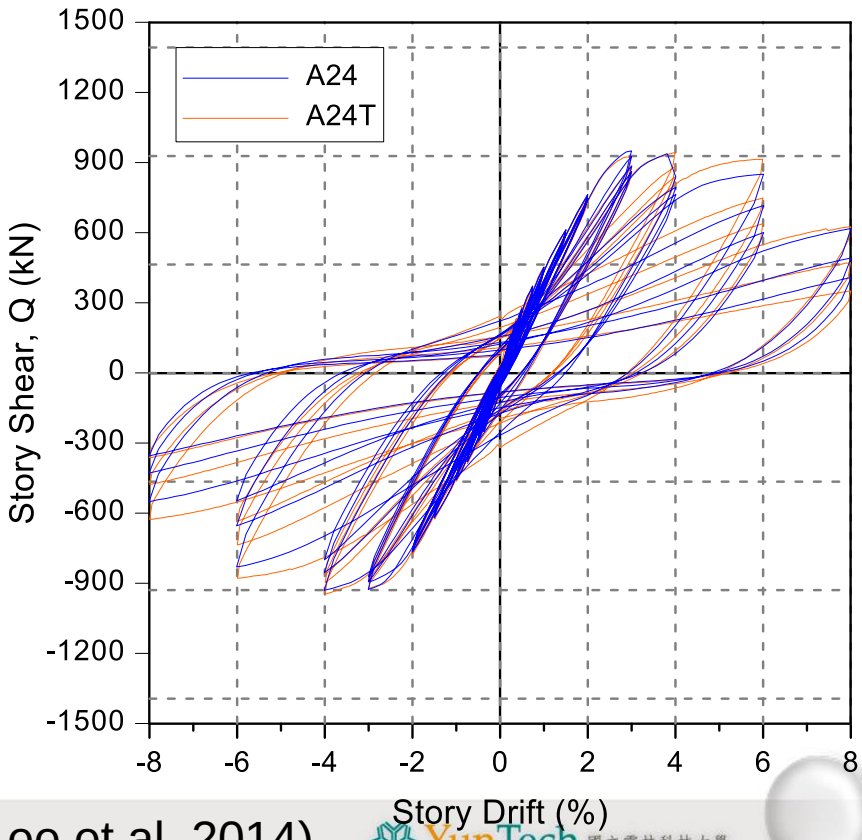
李宏仁; 張又仁; 黃世建, 高強度鋼筋混凝土梁柱接頭耐震性能研究及資料庫建構. *中國土木水利工程學刊* **2014,** 26, (3), 257-263.

18.5.3.2 若構入接頭的梁連續或符合15.2.7節，且梁寬度至少為柱寬度之3/4，則在該構入梁接頭兩側較淺梁深度h之範圍內，依表18.4.5.4 (a)(b)或(d)(e)規定平行該構入梁方向之鋼筋量應可減半，且依第18.4.5.3節規定之間距允許增至 15 cm。

解說：

若具適當尺寸之梁在接頭一雙對面構入時，可降低平行該梁方向之圍束鋼筋用量 (李宏仁等人 2014)，且可增加其間距。





(Lee et al. 2014)

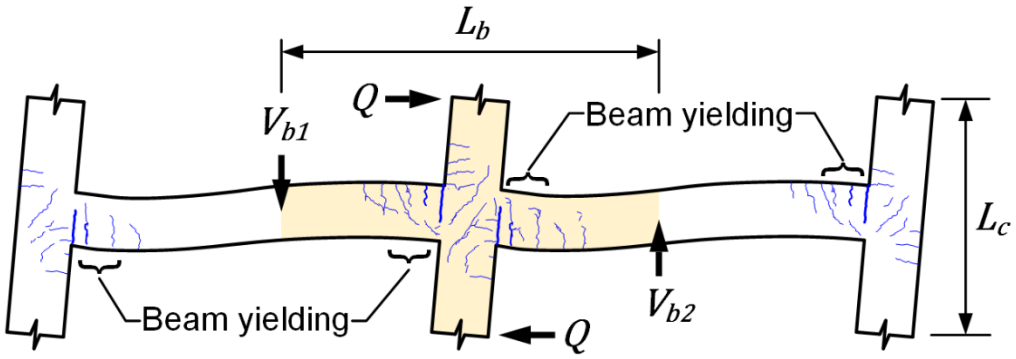
簡報大綱

- 我國規範修訂之背景說明
- 接頭最小深度 (土木401-112　18.5.2)
- 接頭橫向鋼筋 (土木401-112　18.5.3)
- 接頭剪力強度 (土木401-112　18.5.4)
- 受拉錨定長度 (土木401-112　18.5.5)
- 結語

特殊抗彎矩構架之接頭

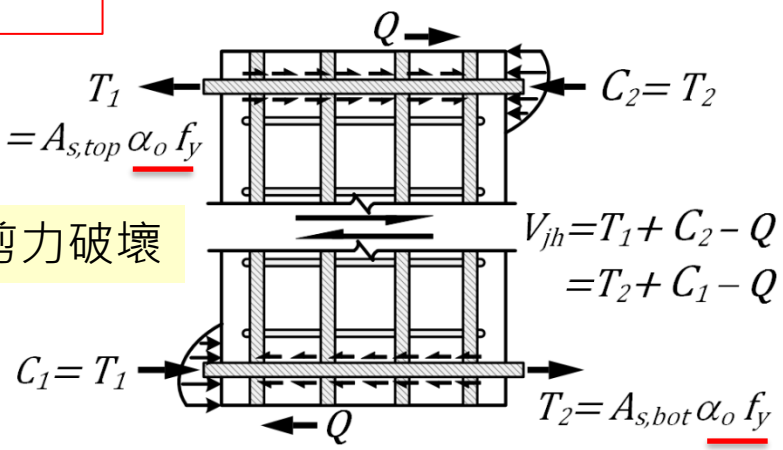
M.C.E. 作用下

@18.5.2.1
梁縱向鋼筋於接頭面之力，應假設撓曲拉力鋼筋應力為 $1.25f_y$ 計算之。



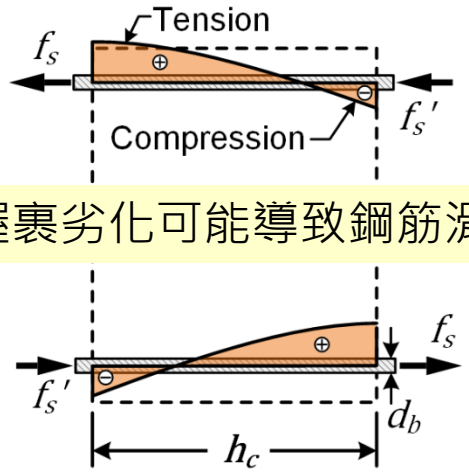
(a) Isolated beam-column unit of a moment-resisting frame

高剪力可能導致剪力破壞



(b) Horizontal forces acting on an interior joint

握裹劣化可能導致鋼筋滑移



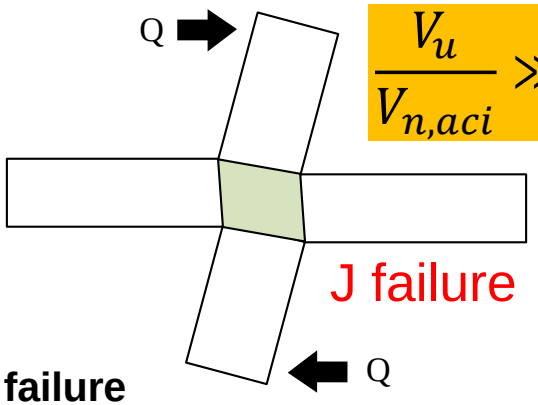
(c) Stresses along beam bars

Lee, H.-J.; Lequesne, R.; Lepage, A.; Lin, J.-X.; Wang, J.-C.; Yin, S. Y.-L., Minimum Joint Depth for Special Moment Frames with High-Strength Reinforcement. *ACI Structural Journal* **2023,** 120, (1), 225-239.

接頭設計剪力與標稱強度之比值

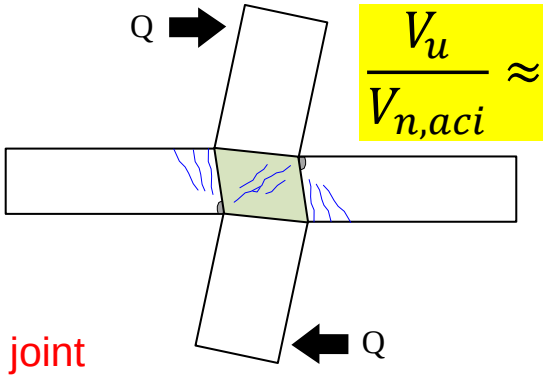
Joint shear demand to capacity ratio

$$V_u \le \phi V_n$$



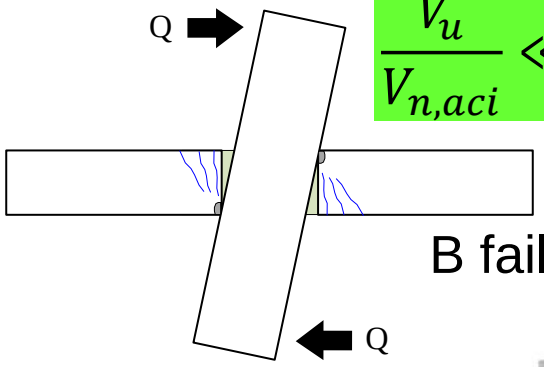
$$\frac{V_u}{V_{n,aci}} \gg 1.0$$

J failure



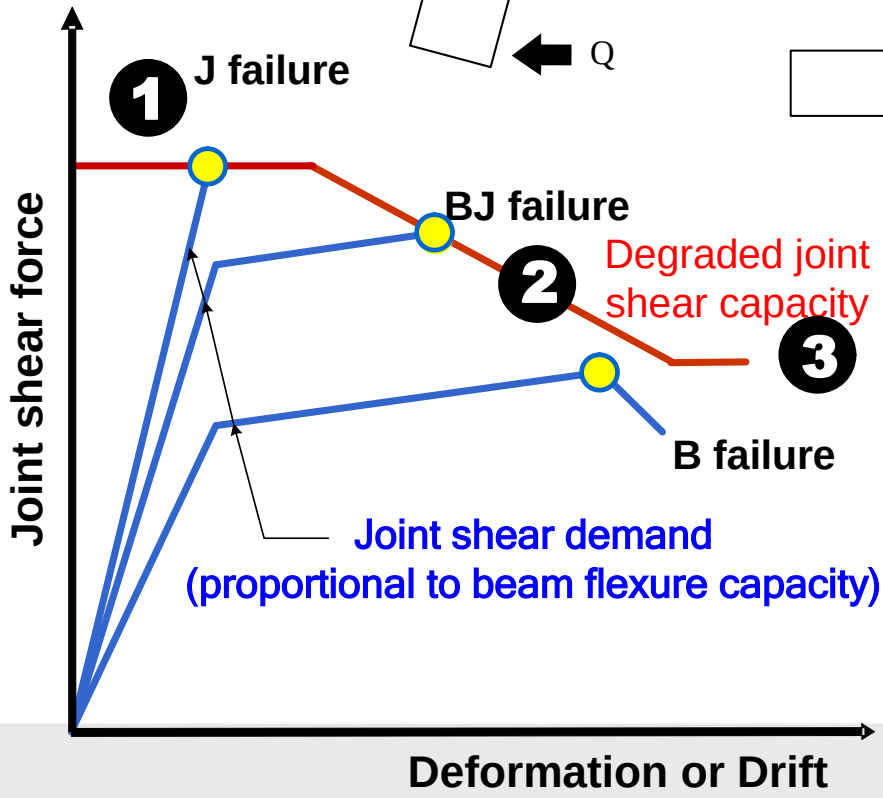
$$\frac{V_u}{V_{n,aci}} \approx 1.0$$

BJ failure



$$\frac{V_u}{V_{n,aci}} \ll 1.0$$

B failure



@18.5.4

標稱接頭剪力強度 V_n

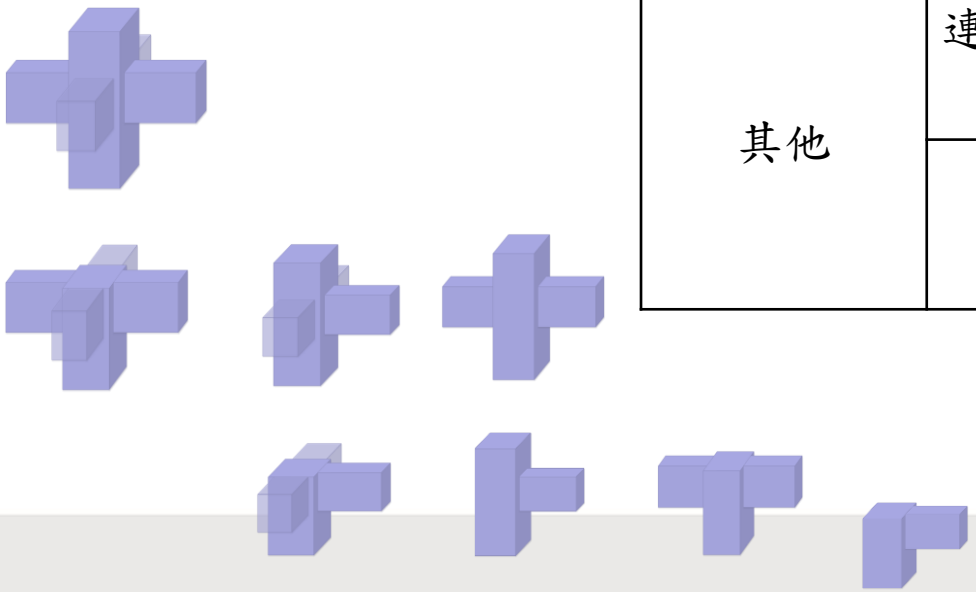
$\phi V_n \geq V_u$

土木401-100規範(表 15.6.3.1)

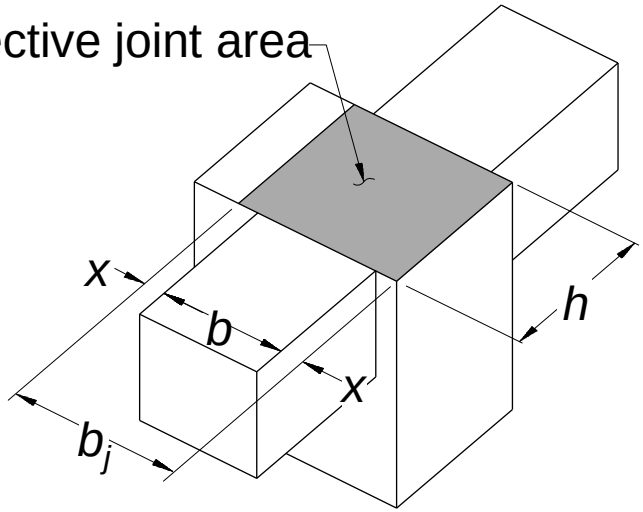
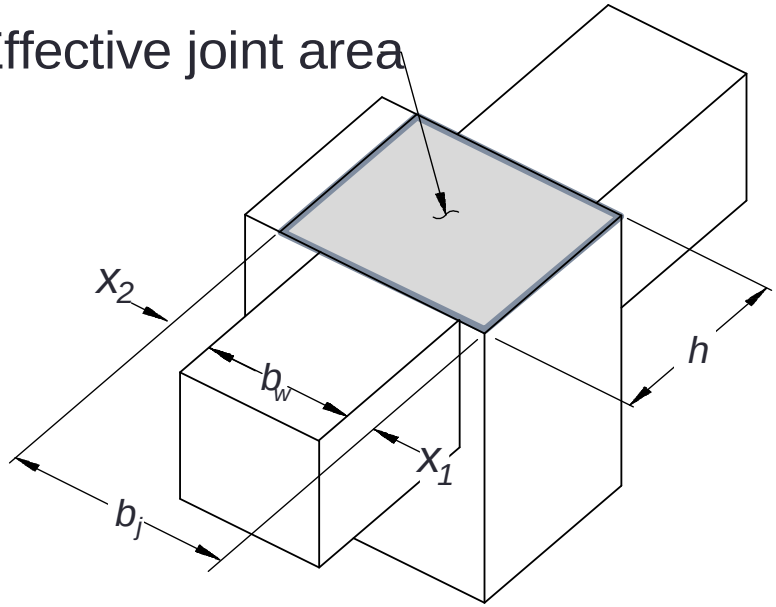
接頭形式	V_n
接頭四面皆受圍束	$5.3\sqrt{f'_c}A_j$
三面或一雙對面受圍束	$3.9\sqrt{f'_c}A_j$
其他	$3.2\sqrt{f'_c}A_j$

土木401-110 規範 (表 18.5.4.3) (kgf/cm^2 unit)

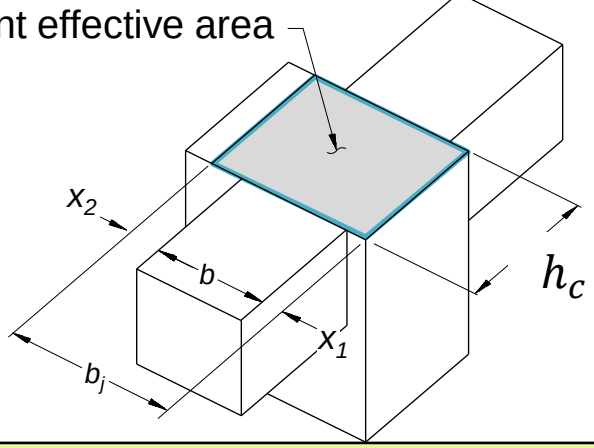
柱	V_u方向的梁	符合第15.2.8節之橫向梁圍束	V_n
連續或符合第15.2.6節	連續或符合第15.2.7節	有	$5.3\lambda\sqrt{f'_c}A_j$
		無	$3.9\lambda\sqrt{f'_c}A_j$
	其他	有	$3.9\lambda\sqrt{f'_c}A_j$
		無	$3.2\lambda\sqrt{f'_c}A_j$
其他	連續或符合第15.2.7節	有	$3.9\lambda\sqrt{f'_c}A_j$
		無	$3.2\lambda\sqrt{f'_c}A_j$
	其他	有	$3.2\lambda\sqrt{f'_c}A_j$
		無	$2.1\lambda\sqrt{f'_c}A_j$



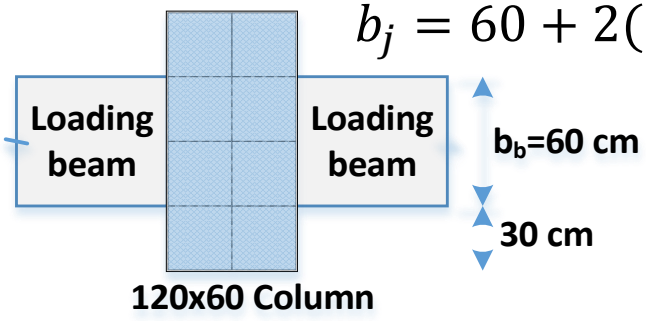
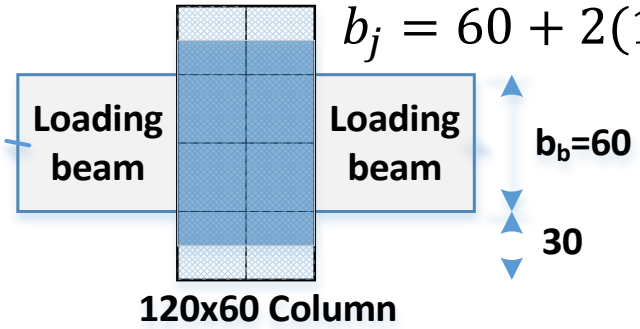
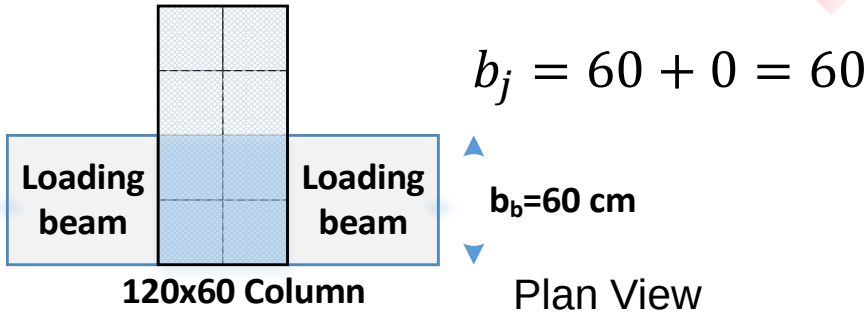
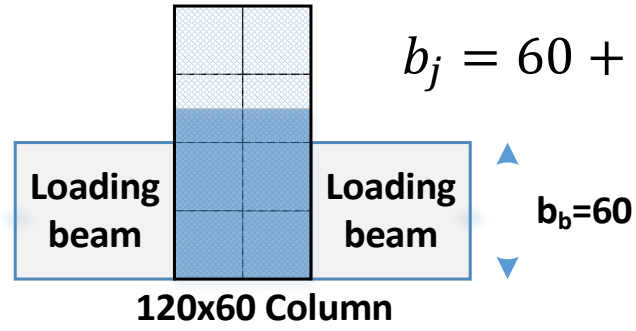
接頭內有效斷面積$A_j = b_j h$

土木 401-100 規範(15.6.3.1)	新規範 (15.4.2.4)
有效接頭寬度 b_j 不超過(a)與(b)之較小值 (a)梁寬加沿剪力方向之柱全深。 (b)梁中心線至兩柱邊取小值的兩倍。	有效接頭寬度b_j不超過梁腹寬度b_w兩側各加(a)與(b)之較小值： (a)接頭深度之1/4。 (b)梁腹側面至柱邊之距離。
 $b_j = b + h \leq b + 2x$	 $b_j = b_w + x_1 + x_2 \leq b_{col}$ $x_1 \; and \; x_2 \; \leq h/4$

接頭內有效斷面積$A_j = b_j h$



由(黃世建等人 2014)檢討梁柱接頭試驗資料庫，建議之有效接頭寬度$b_j = b_w + x_1 + x_2$，對於寬柱-窄梁接頭及偏心梁柱接頭可以獲致較合理的剪力計算強度。

ACI 318-19 或土木401-100	土木401-110 新版規範
$b_j = b + 2x \leq b + h_c$	$b_j = b_w + x_1 + x_2 \leq b_{col}$ x_1 and $x_2 \leq h_c/4$
$b_j = 60 + 2(30) = 120$ 	$b_j = 60 + 2(15) = 90$ 
$b_j = 60 + 0 = 60$ 	$b_j = 60 + 15 = 75$ 

某建案之梁柱接頭剪力強度檢討(I)

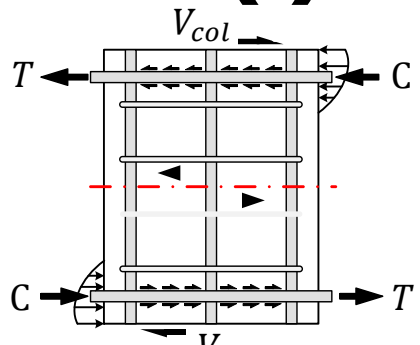
柱	V_u方向的梁	橫向梁圍束	γ
連續	連續	有	5.3
		無	3.9
	不連續	有	3.9
		無	3.2

X向：柱連續、梁不連續、有橫向梁圍束，$\gamma = 3.9$

Y向：柱連續、梁連續、無橫向梁圍束，$\gamma = 3.9$

$$V_u = T + C - V_{col}$$

$$V_n = \gamma\sqrt{f'_c}A_j$$

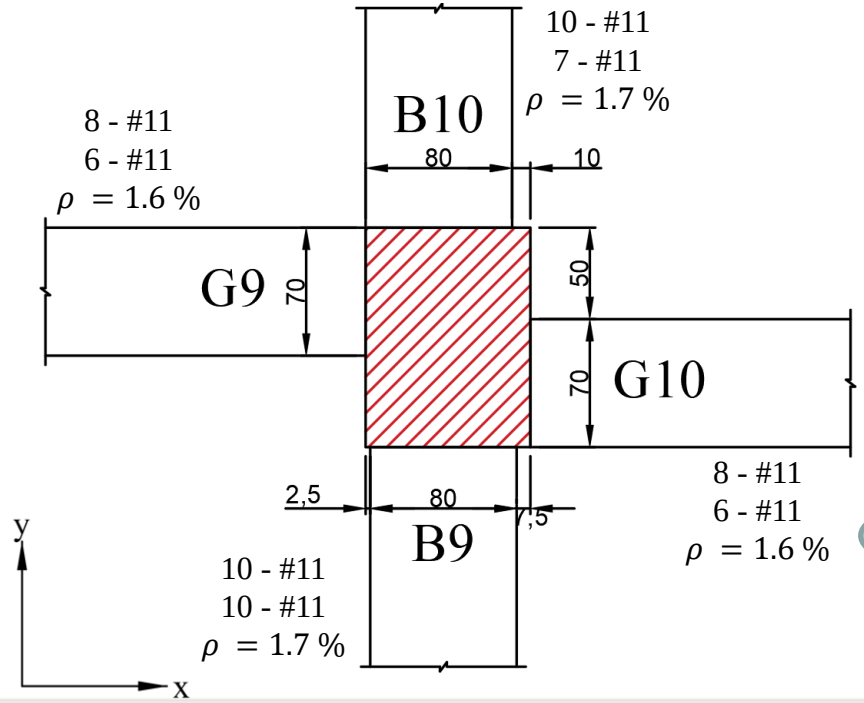


$f'_c = 350\ kgf/cm^2$

$f_y = 5000\ kgf/cm^2$

方向	V_u (tf)	γ	b_j (cm)	h_c (cm)	V_n (tf)	$\frac{\Phi V_n}{V_u}$
X	708	3.9	92.5	90	607	0.73

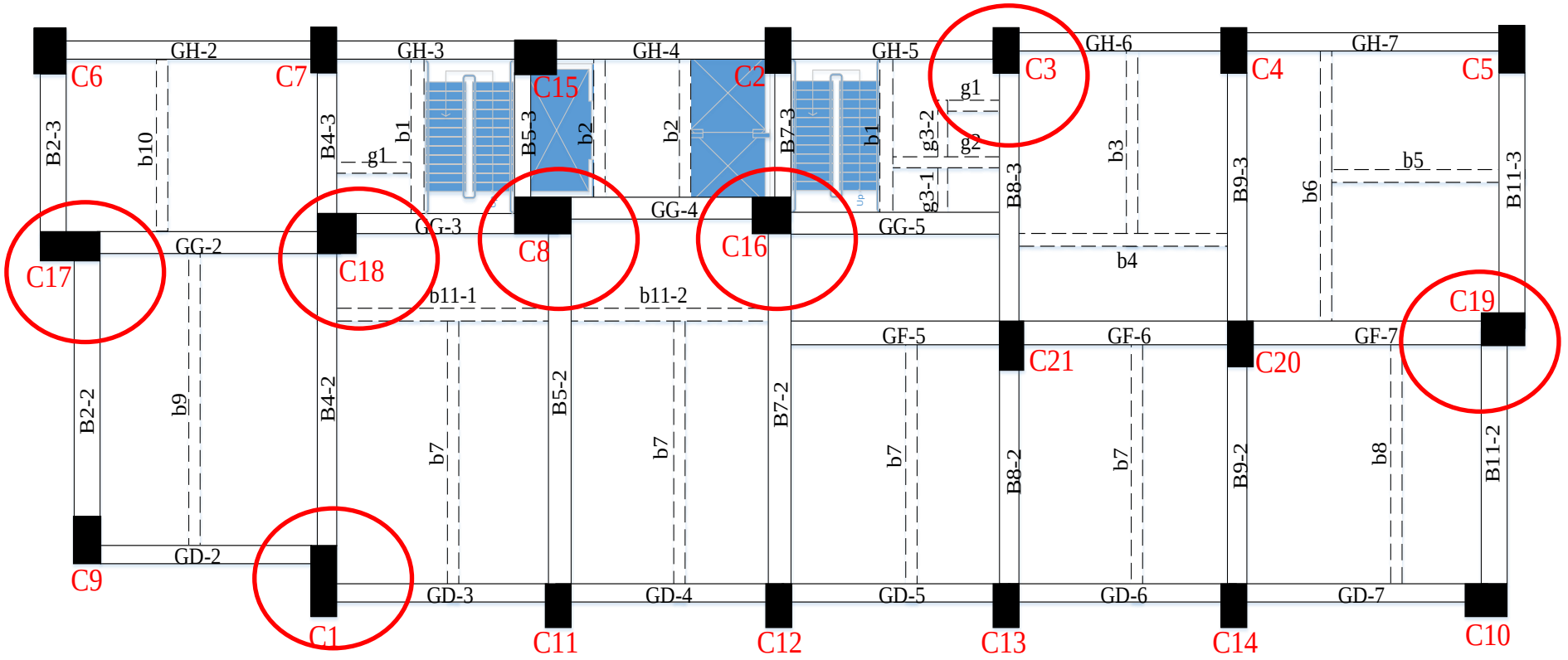
接頭強度不足



錯位偏心梁柱接頭之有效寬度
如何計算?
規範並未說明

Unit：$cm\ , kgf/cm^2$　　$\rho = A_S/bd$　　$\Phi = 0.85$

李宏仁等人(2019)，鋼筋混凝土柱梁偏心接合之耐震抗剪強度檢討，內政部建築研究所委託研究報告，台北



寬柱-窄梁 接合

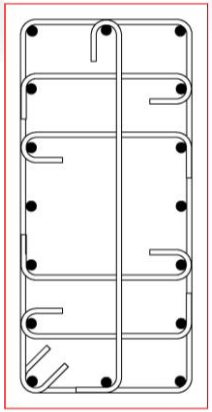
偏心梁柱接頭比例極高
錯位偏心接頭!

系列I試體接頭設計參數

2
1

柱

100×50



柱主筋: 16-#8
箍筋: #4@10
繫筋: #4@10

$f_c' = 350\ (kgf/cm^2)$

$f_y = 4200\ (kgf/cm^2)$

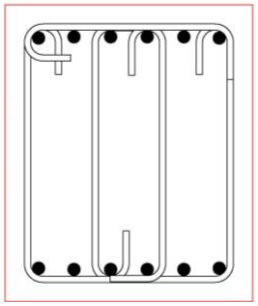
$f_{yt} = 4200\ (kgf/cm^2)$

$A_{sh,ratio1} = 1.10$

$A_{sh,ratio2} = 1.21$

$\frac{V_u}{A_g\sqrt{f_c'}} = 2.84$

$M_R = \frac{M_{nc}}{M_{nb}} = \frac{100.93}{63.18} = 1.59$

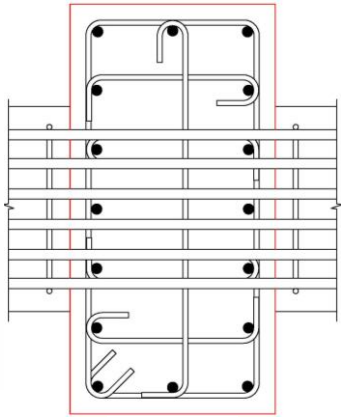


梁

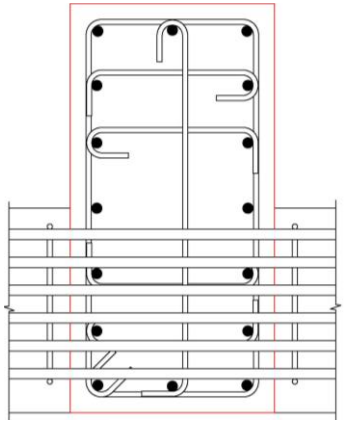
50×60

梁主筋: 6-#8 , 6-#8
箍筋: #4@15
繫筋: #4@15

$\frac{h_c}{d_b} = \frac{50\ cm}{2.5\ cm} = 20$

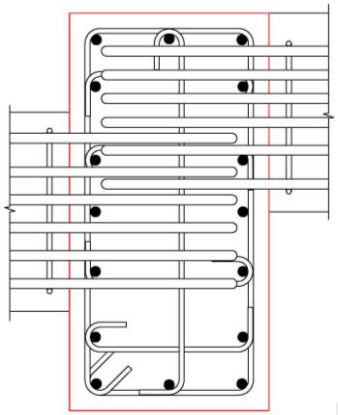


1A

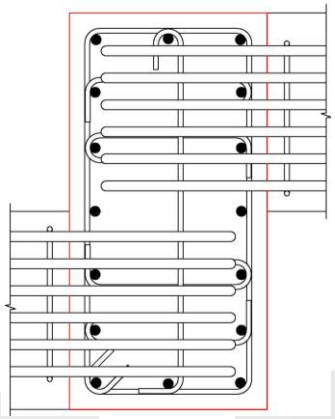


1B

$\frac{l_{dh,provided}}{l_{dh,required}} = \frac{40.3\ cm}{34.2\ cm} = 1.18$



1C



1D

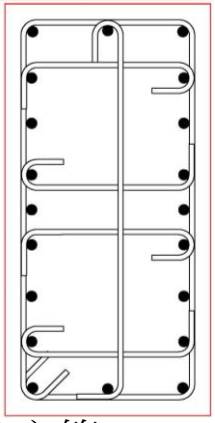
系列II試體接頭設計參數

2

1

柱

100×50



柱主筋: 20-#8
箍筋: #4@10
繫筋: #4@10

$f_c' = 350\ (kgf/cm^2)$

$f_y = 4200\ (kgf/cm^2)$

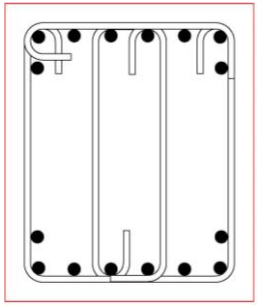
$f_{yt} = 4200\ (kgf/cm^2)$

$A_{sh,ratio1} = 1.10$

$A_{sh,ratio2} = 1.21$

$\frac{V_u}{A_g\sqrt{f_c'}} = 3.93$

$M_R = \frac{M_{nc}}{M_{nb}} = \frac{116.3}{81.1} = 1.43$

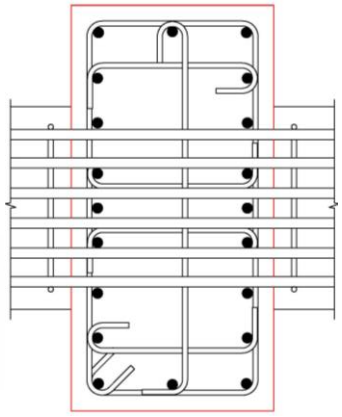


梁

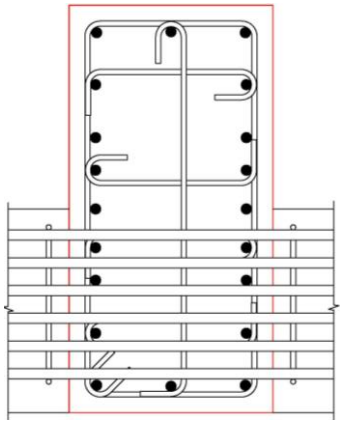
50×60

梁主筋: 8-#8 , 8-#8
箍筋: #4@15
繫筋: #4@15

$\frac{h_c}{d_b} = \frac{50\ cm}{2.5\ cm} = 20$

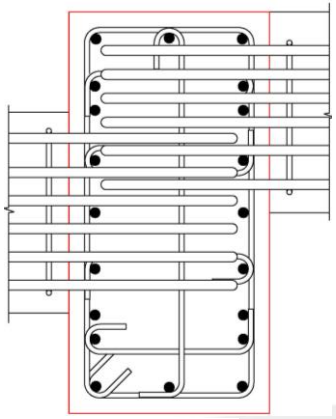


2A

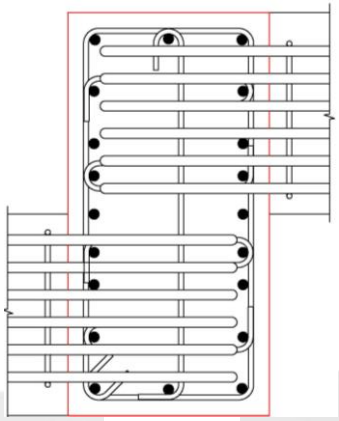


2B

$\frac{l_{dh,provided}}{l_{dh,required}} = \frac{35\ cm}{34.2\ cm} = 1.02$



2C



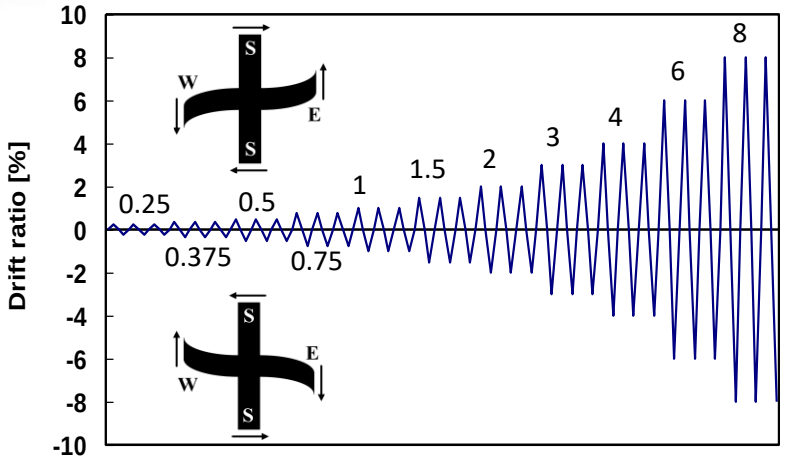
2D

材料試驗

批次	試體編號	梁柱接頭 試驗齡期 (日)	圓柱抗壓 試驗齡期 (日)	f'_c (kgf/cm^2)	MPa
第一批次	1A	49	45	282	27.6
	2A	55			
	1B	59	57	305	28.9
	2B	65			
第二批次	1C	46	46	317	31.1
	2C	49			
	1D	69	68	328	32.1
	2D	74			

鋼筋規格 SD420W	降伏點 MPa		抗拉強度 MPa		伸長率%		拉降比	
稱號	試驗值	平均值	試驗值	平均值	試驗值	平均值	試驗值	平均值
D13-1	487	488	659	659	18	20	1.35	1.35
D13-2	489		657		22		1.34	
D13-3	489		660		20		1.35	
D25-1	447	450	657	658	15	20	1.47	1.46
D25-2	453		661		23		1.46	
D25-3	449		656		21		1.46	

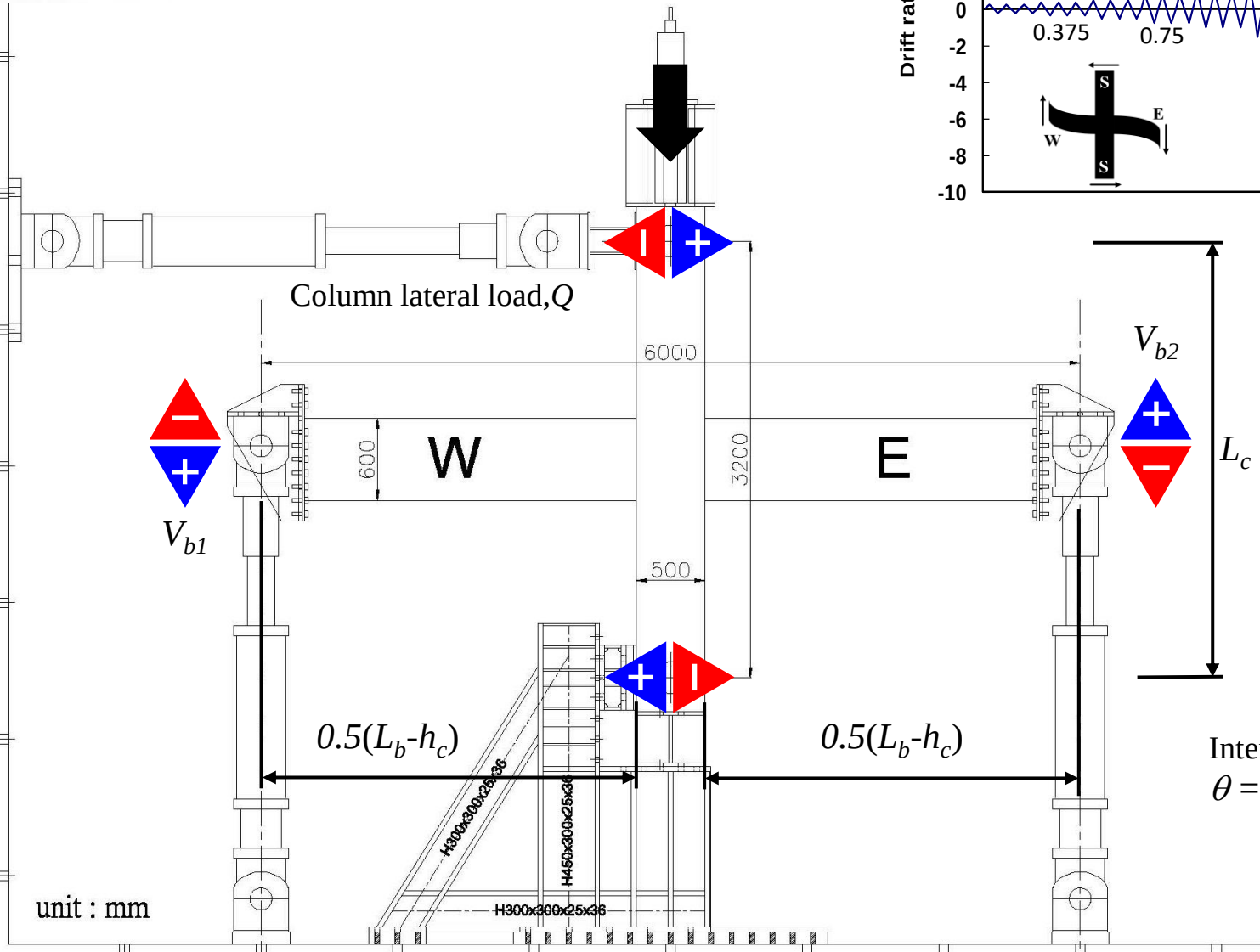
試驗佈設



$0.1A_g f_c'$

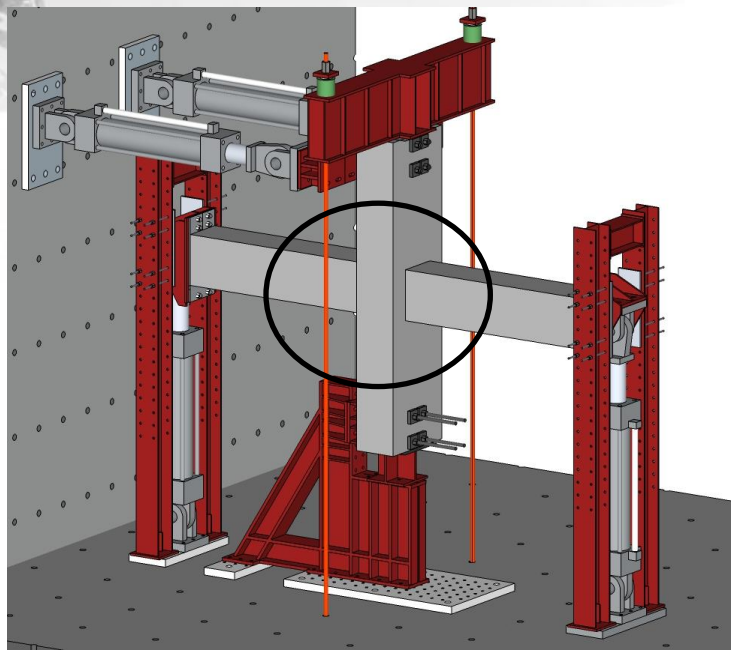
Column axial load,P

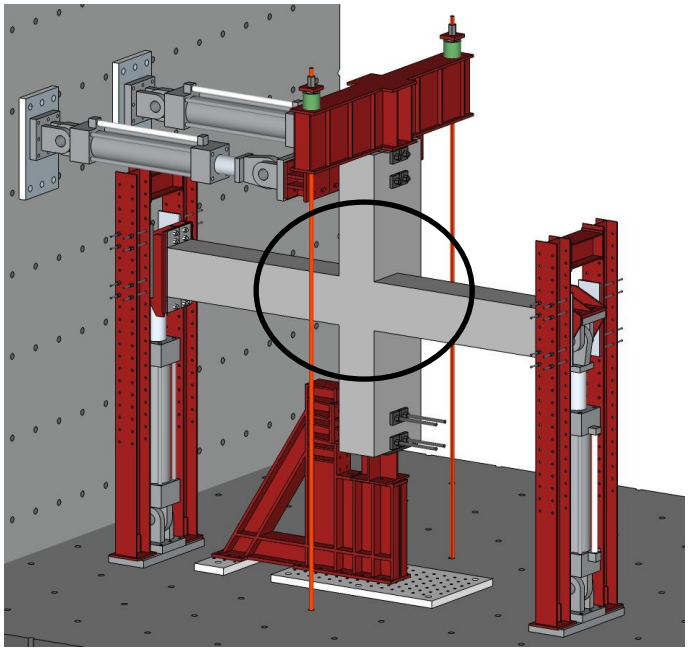
Column lateral load,Q

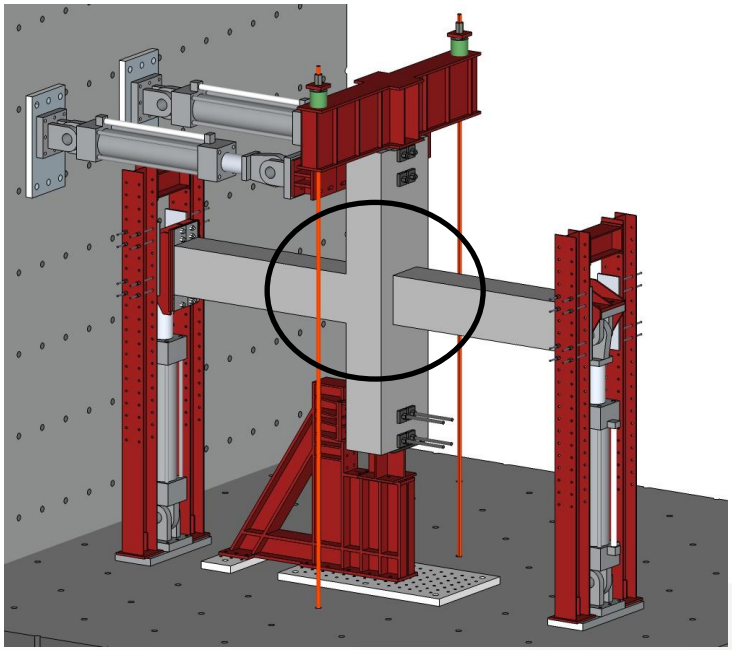


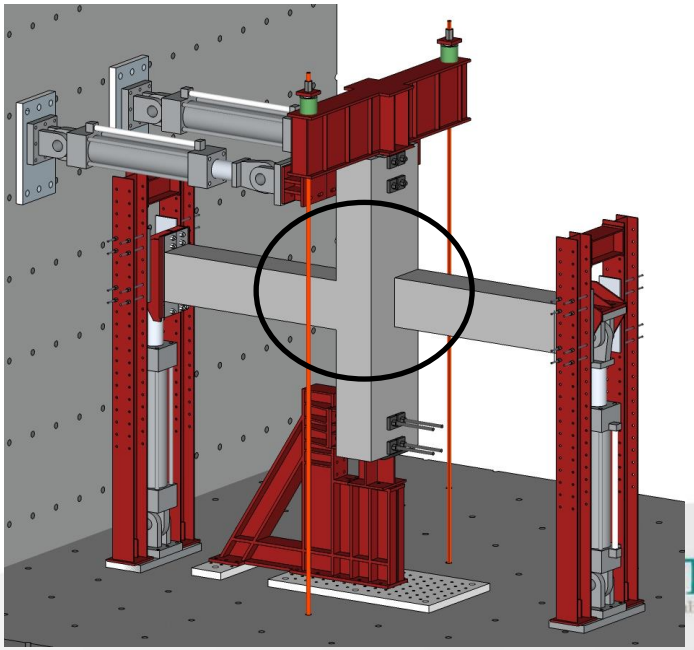
Interstory drift load ratio

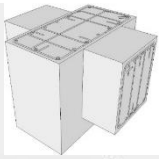
$\theta = (\delta_1+\delta_2)/L_b=\Delta/L_c$

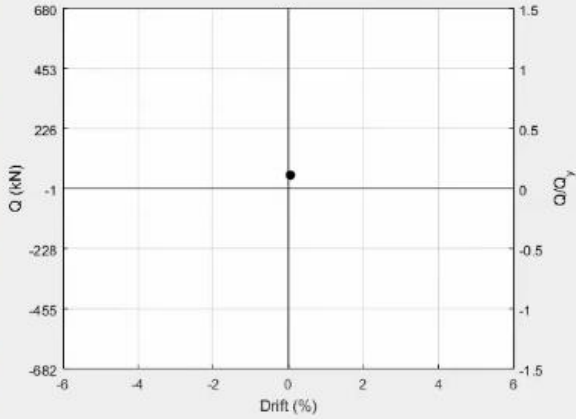




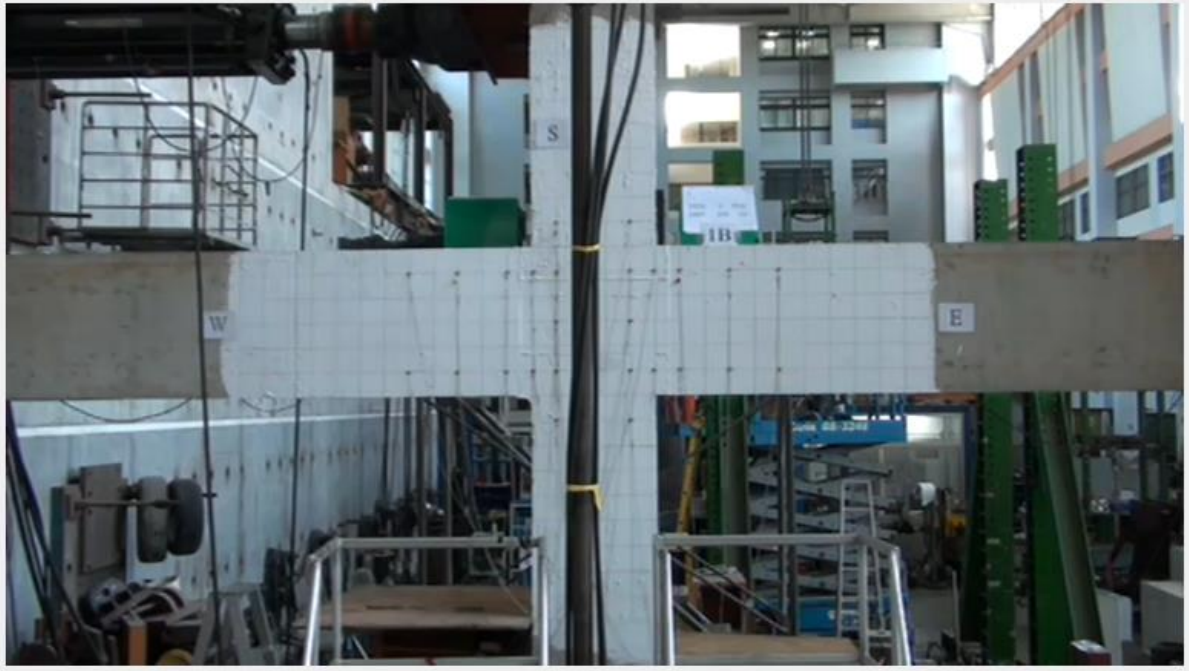


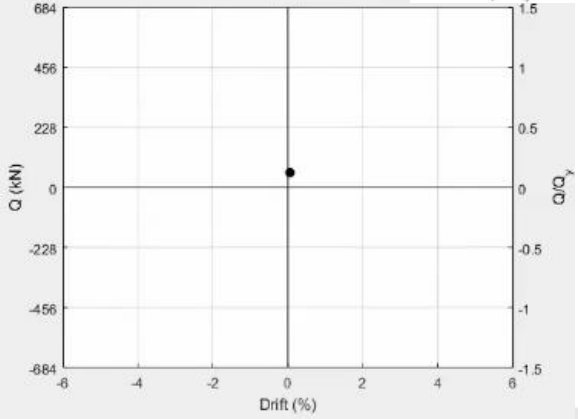




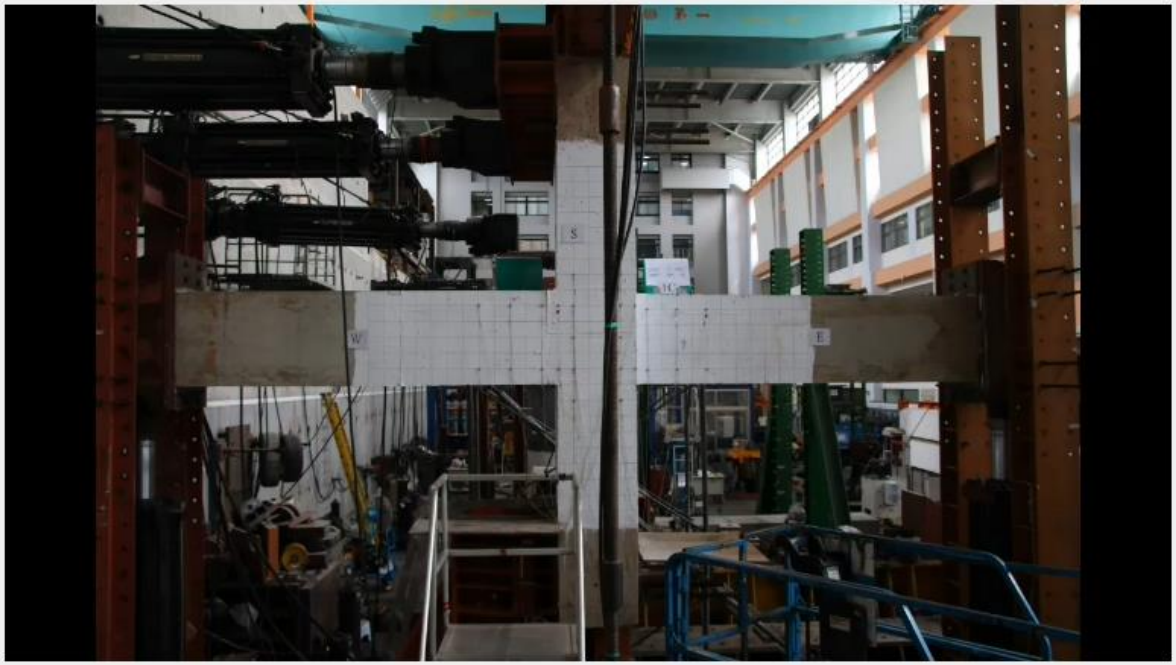


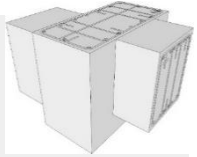
Q_{max}=467 kN



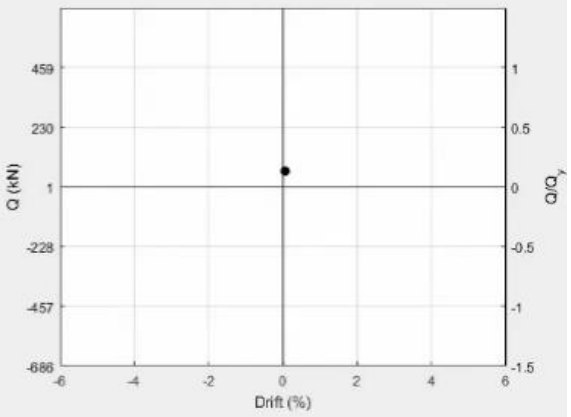


Q_{max}=468 kN

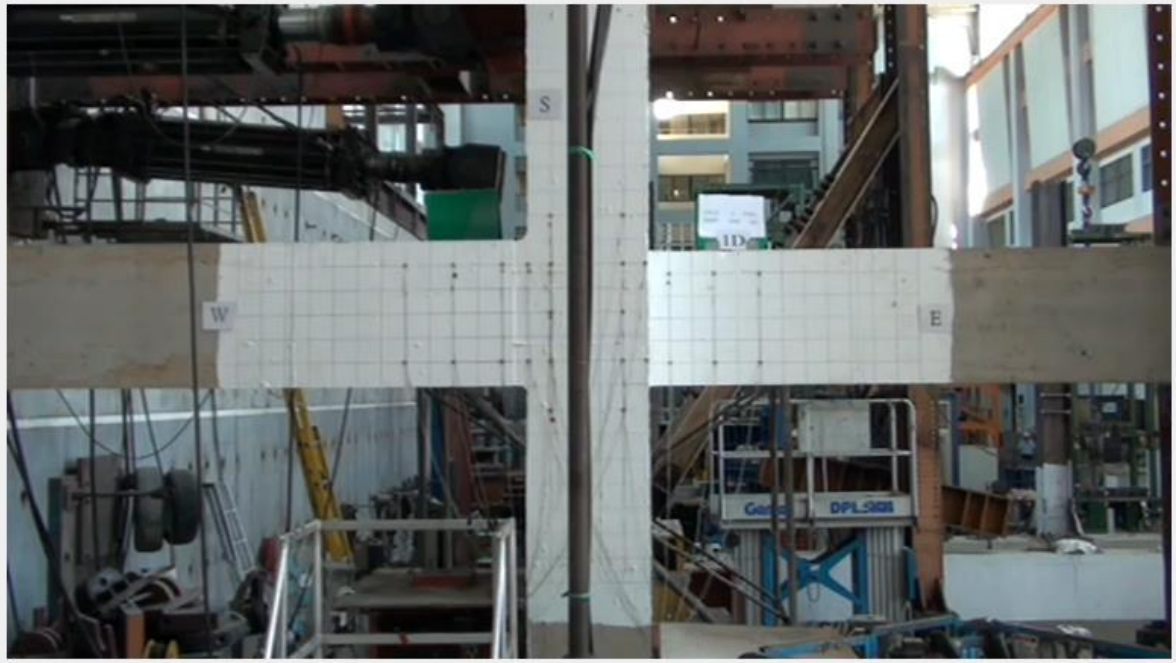


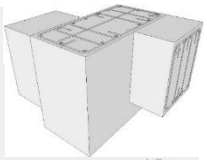


1C

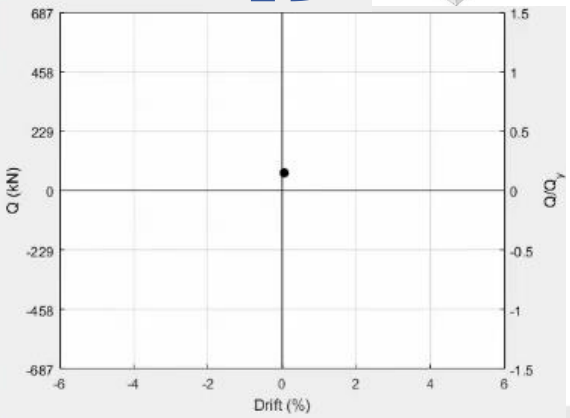


Q_{max}=497 kN

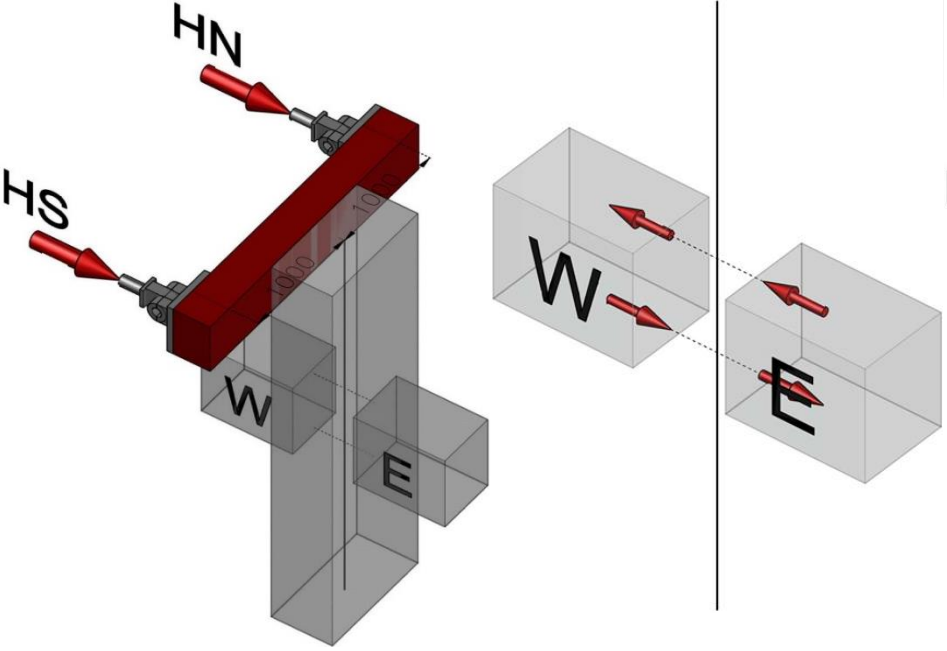


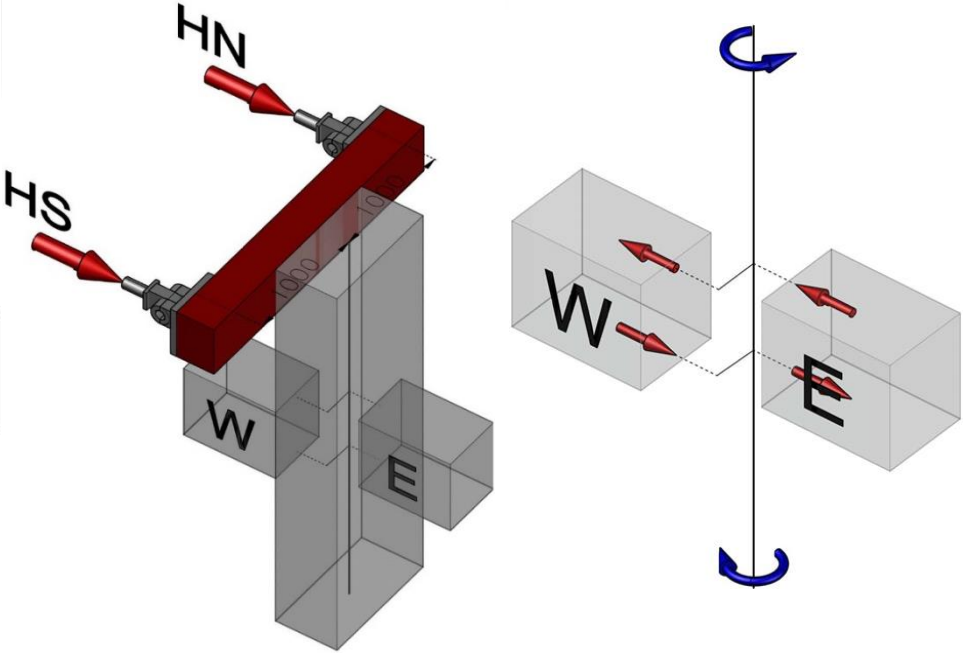


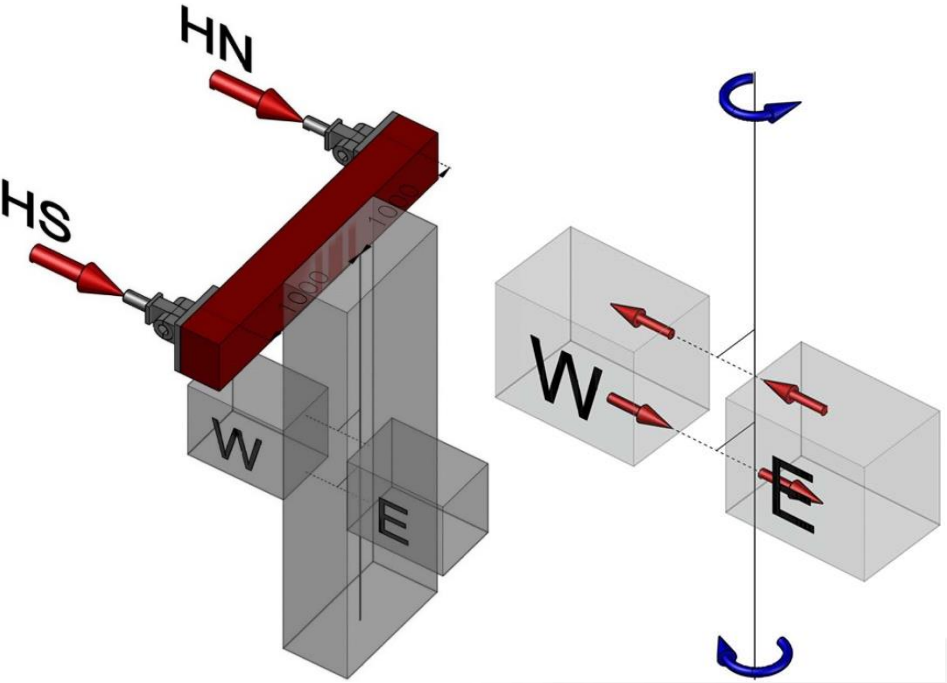
1D

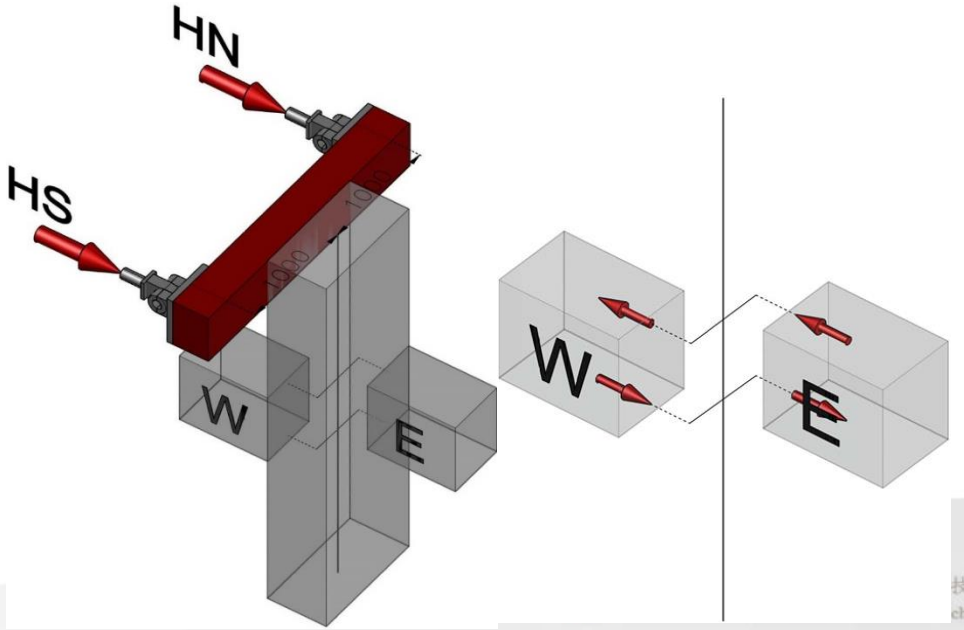


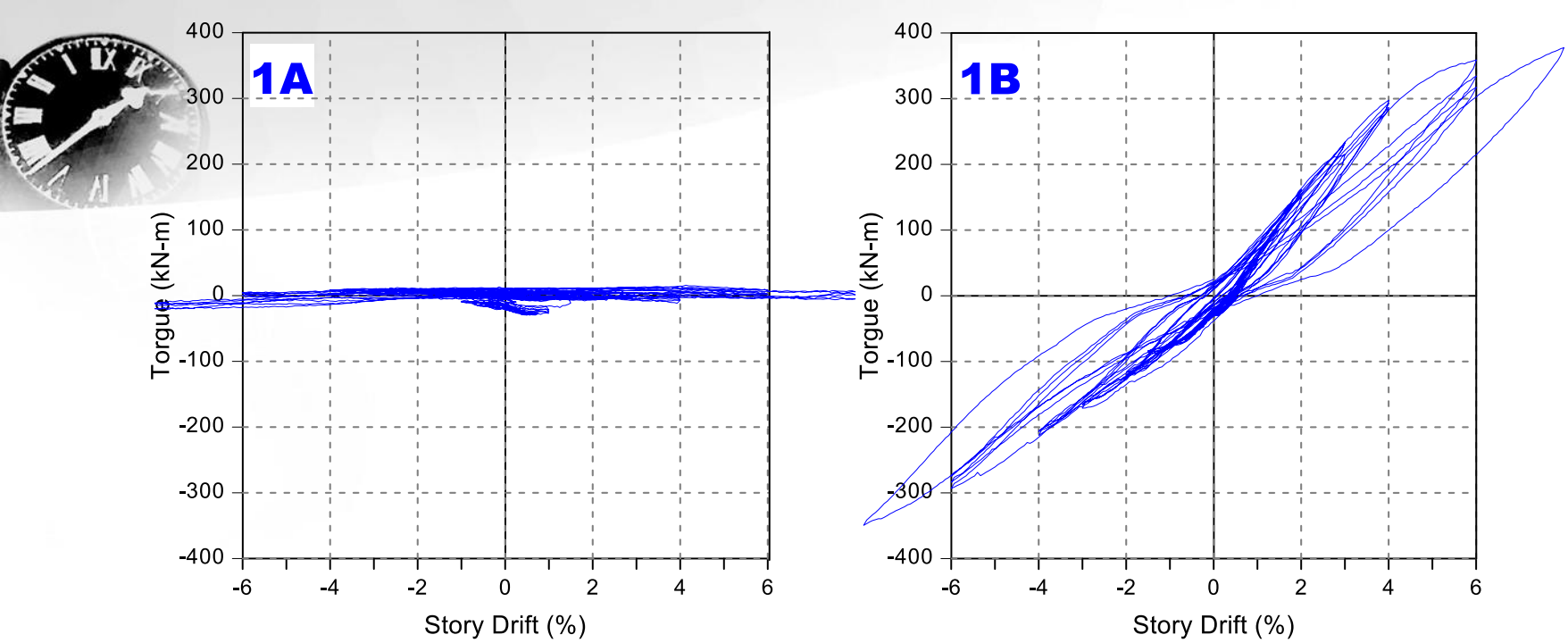
Q_{max}=488 kN


HN
HS
W
E
W
E

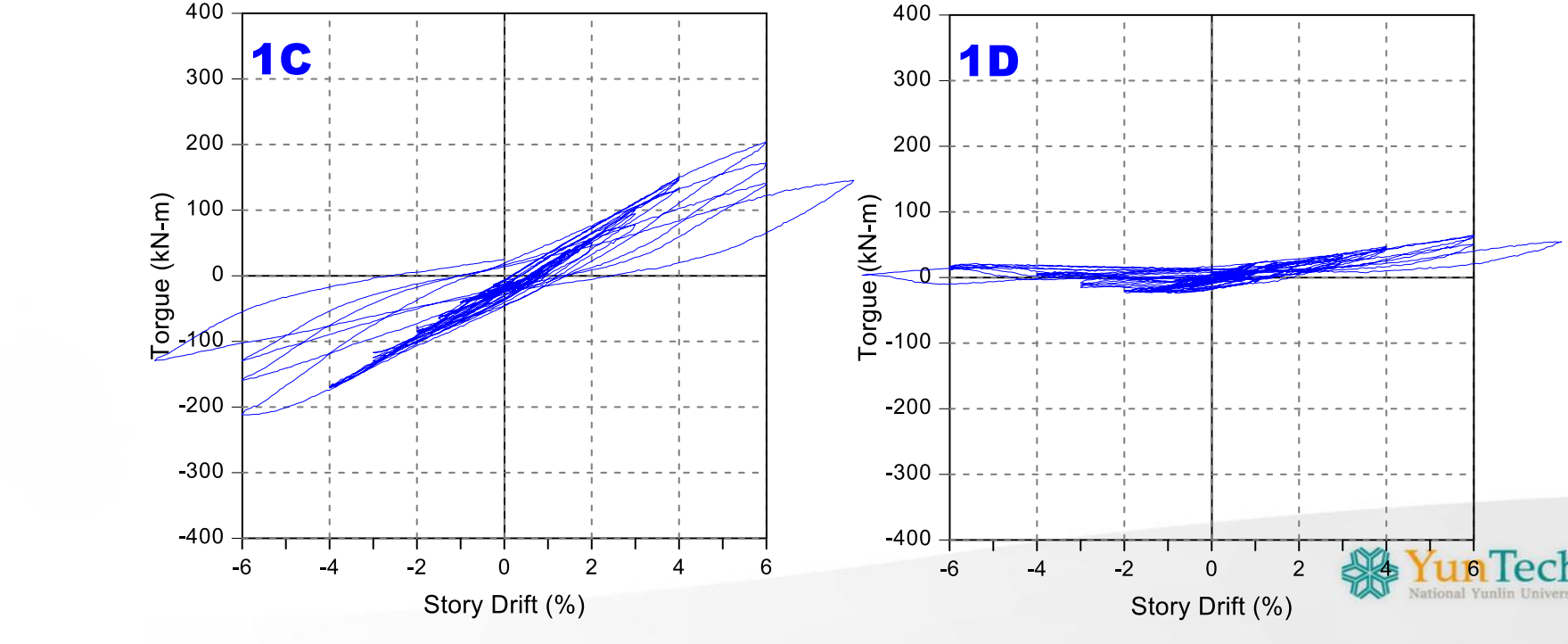

HN
HS
W
E
W
E


HN
HS
W
E
W
E

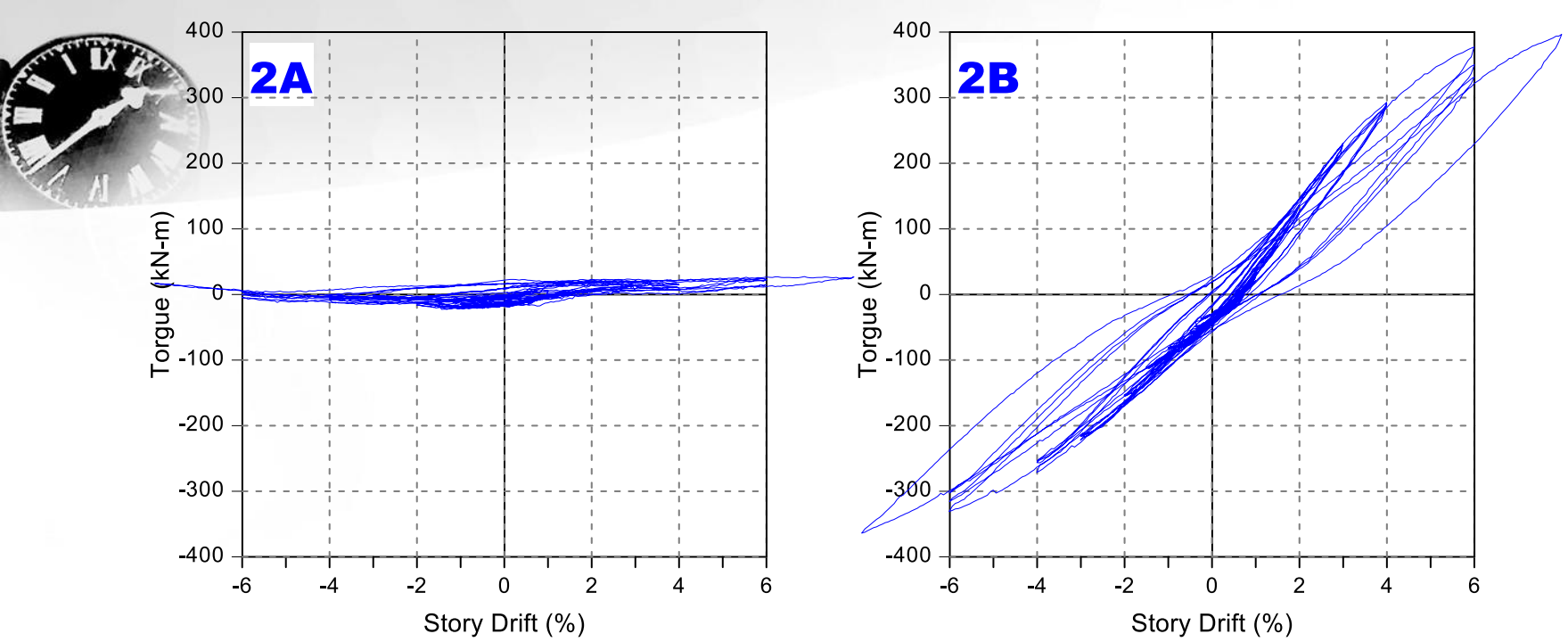

HN
HS
W
E
W
E


1A
Torgue (kN-m)
Story Drift (%)

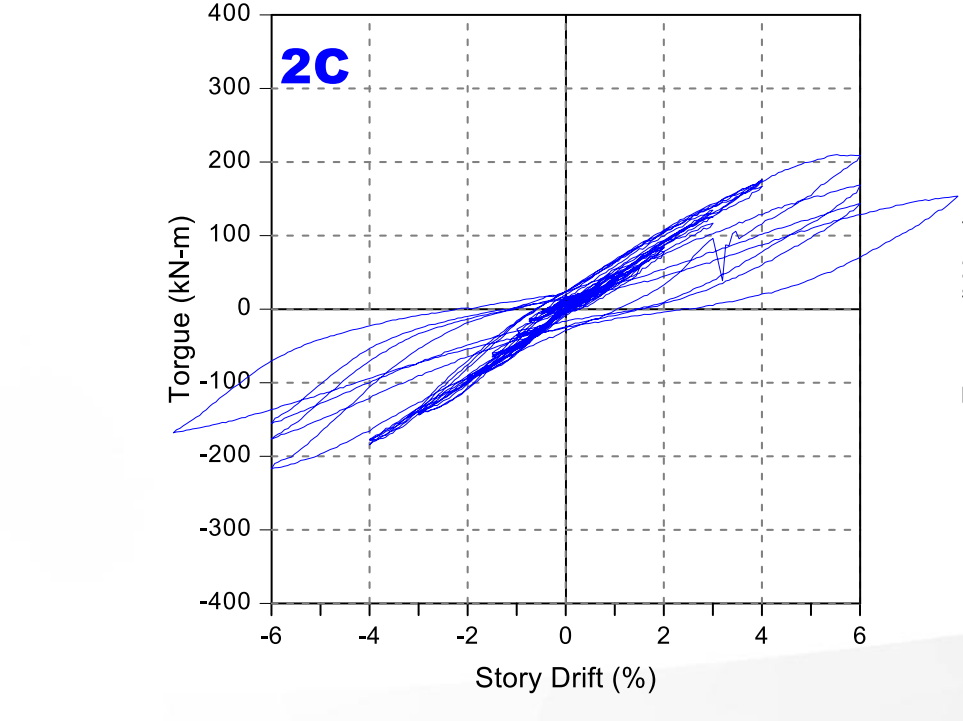
1B
Torgue (kN-m)
Story Drift (%)

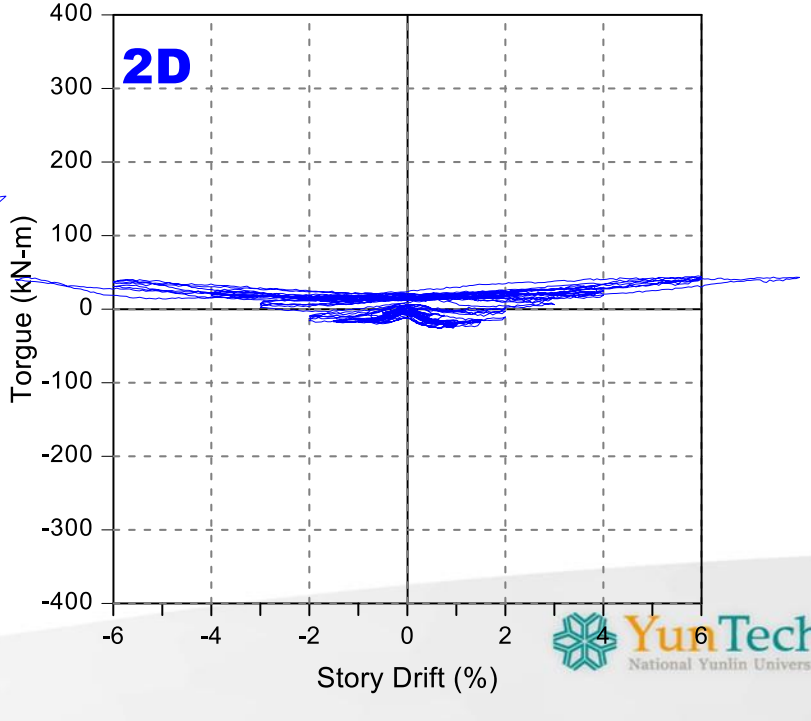

1C
Torgue (kN-m)
Story Drift (%)

1D
Torgue (kN-m)
Story Drift (%)

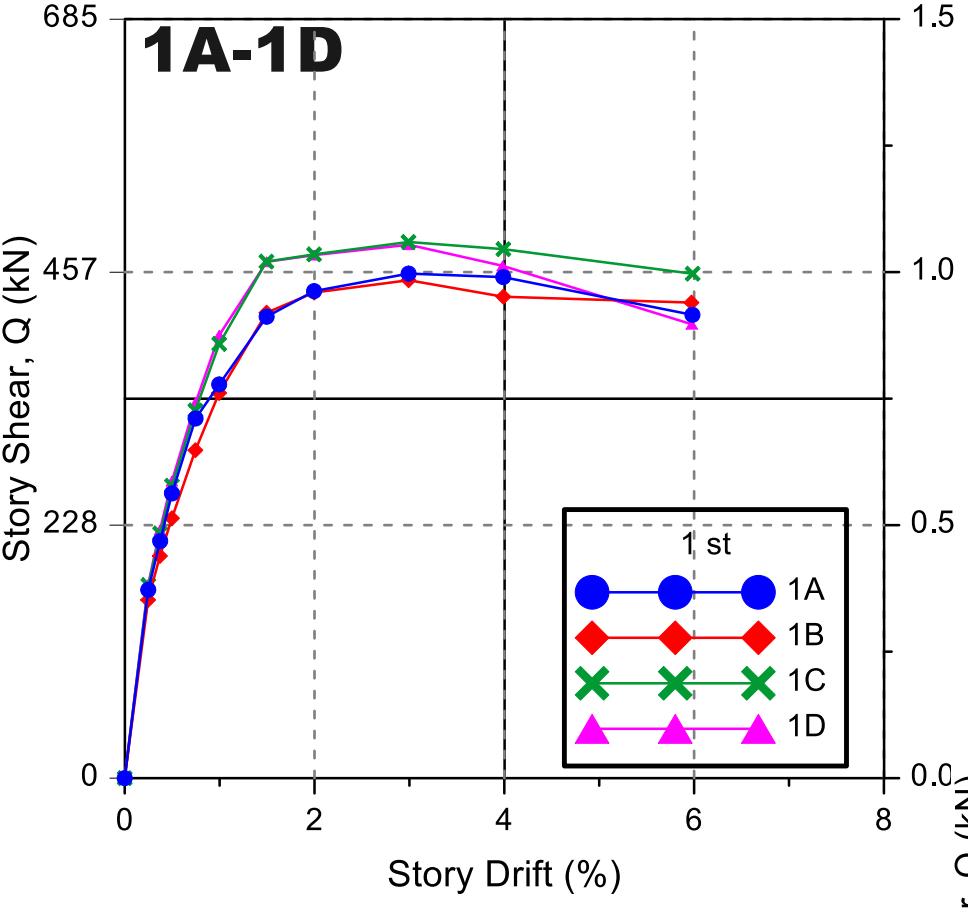

2A
Torgue (kN-m)
Story Drift (%)
400
300
200
100
0
-100
-200
-300
-400
-6
-4
-2
0
2
4
6

2B
Torgue (kN-m)
Story Drift (%)
400
300
200
100
0
-100
-200
-300
-400
-6
-4
-2
0
2
4
6

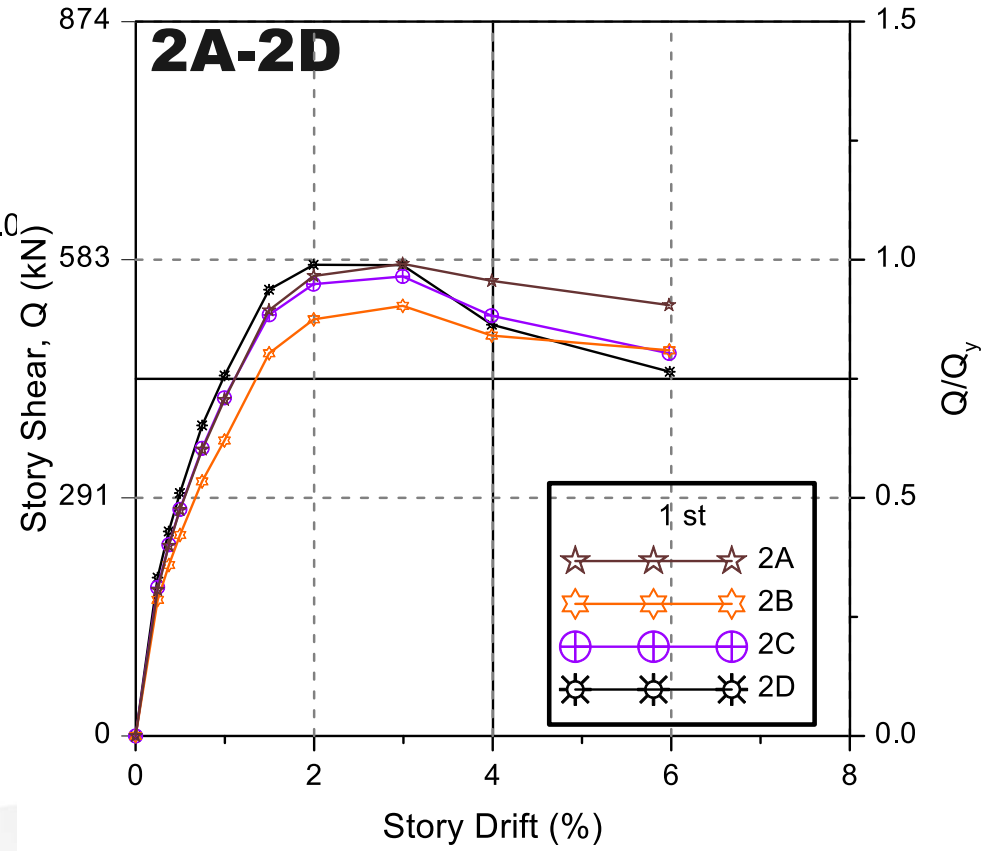

2C
Torgue (kN-m)
Story Drift (%)
400
300
200
100
0
-100
-200
-300
-400
-6
-4
-2
0
2
4
6


2D
Torgue (kN-m)
Story Drift (%)
400
300
200
100
0
-100
-200
-300
-400
-6
-4
-2
0
2
4
6

包絡線比較

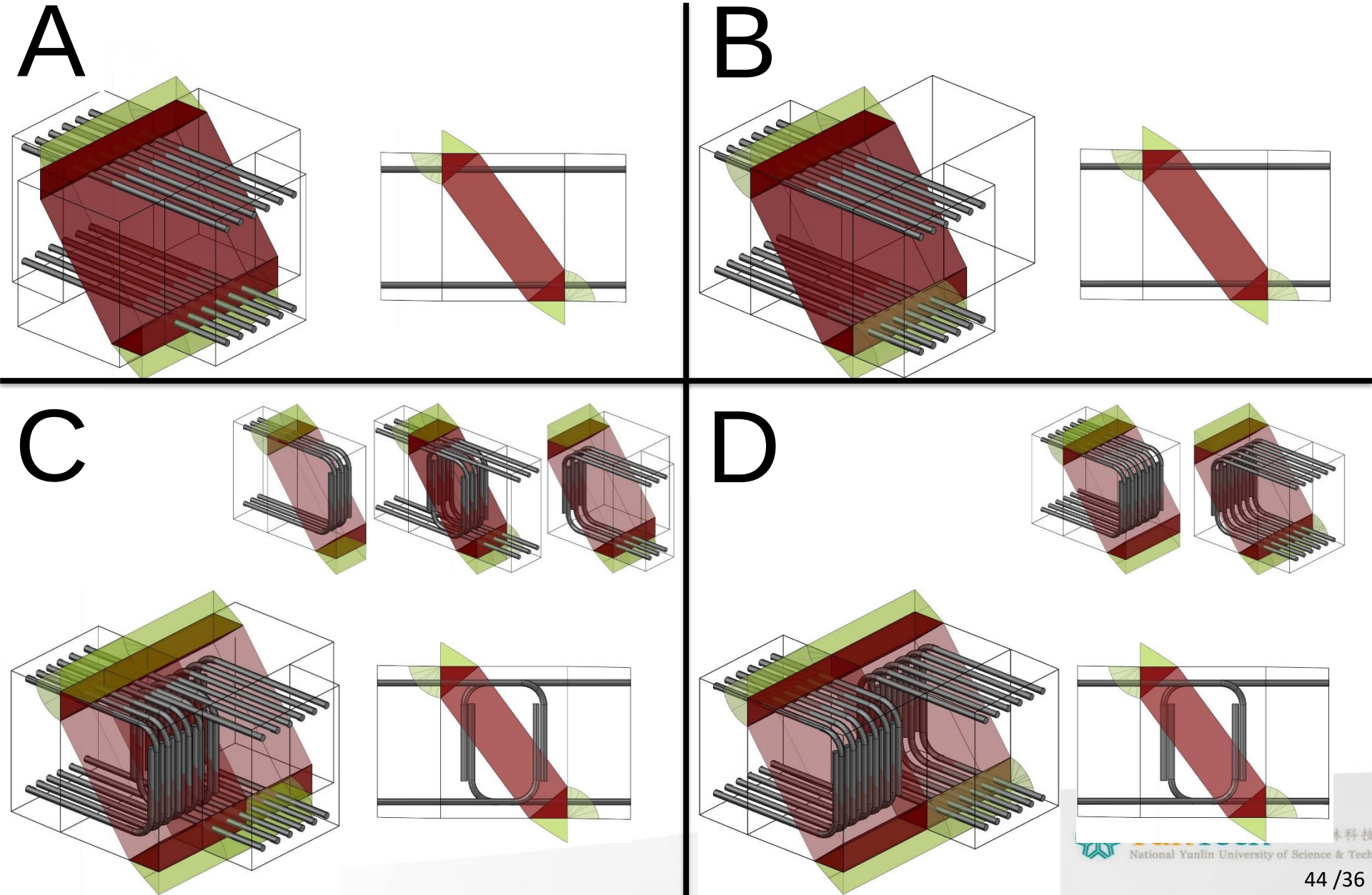


注意試體C、D在3% drift 達到最大強度後的衰減較為明顯



錯位偏心接合
強度並不差

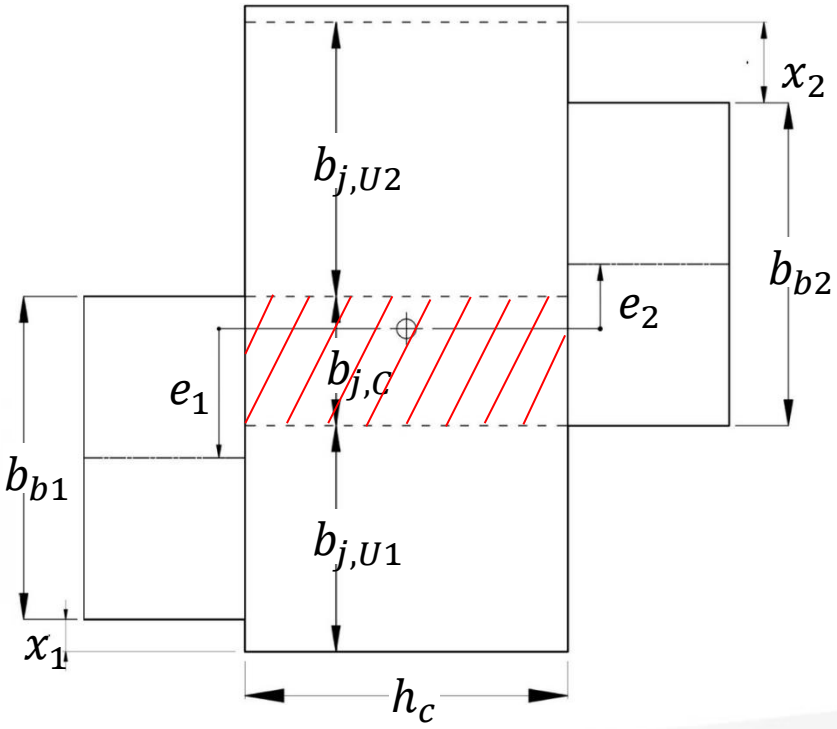
對角壓桿傳遞機制



本研究建議(未來規範修正方案)

$$V_{n,pro} = \gamma_C \sqrt{f'c} \times b_{j,C} \times h_c + \gamma_U \sqrt{f'c} \times b_{j,U} \times h_c$$

交集區　　延伸區



$$b_{j,C} = \frac{b_{b1} + b_{b2}}{2} - \Delta e \quad \geq 0$$

$$\Delta e = \left| \vec{e}_1 - \vec{e}_2 \right|$$

$$b_{j,U} = b_{b1} + b_{b2} + x_1 + x_2 - 2b_{j,C}$$

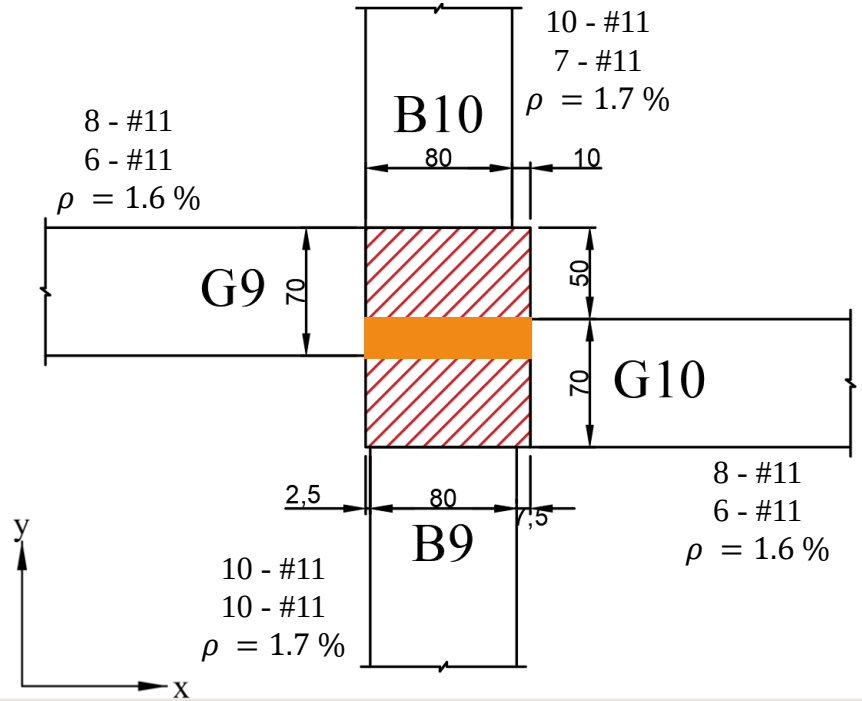
x_1與$x_2 \leq h/4$

某建案之梁柱接頭剪力強度檢討(修)

柱	V_u方向的梁	橫向梁圍束	γ
連續	連續	有	5.3
		無	3.9
	不連續	有	3.9
		無	3.2

X向：柱連續、梁不連續、有橫向梁圍束，$\gamma = 3.9$

Y向：柱連續、梁連續、無橫向梁圍束，$\gamma = 3.9$



$$V_u = T + C - V_{col}$$

$$V_{n,pro} = \gamma_C \sqrt{f'c} \times b_{j,C} \times h_c + \gamma_U \sqrt{f'c} \times b_{j,U} \times h_c$$

$f_y = 5000\ kgf/cm^2$　　$f_c' = 350\ kgf/cm^2$

方向	區塊	γ	b_j (cm)	h_c (cm)	V_n (tf)
X	交集區	5.3	20	90	178.5
X	延伸區	3.9	50+50	90	459.7
				合計	835.2

$$V_u = 709\ \text{tf} < \phi V_n = 710\ \text{tf}\quad \text{OK.}$$

Unit：$cm, kgf/cm^2$　　$\rho = A_S/bd$　　$\Phi = 0.85$

簡報大綱

- 我國規範修訂之背景說明
- 接頭最小深度 (土木401-112　18.5.2)
- 接頭橫向鋼筋 (土木401-112　18.5.3)
- 接頭剪力強度 (土木401-112　18.5.4)
- 受拉錨定長度 (土木401-112　18.5.5)
- 結語

ACI 規範18章SMF接頭錨定長度的變革

規範	ACI 318-14規範	ACI 318-19規範
第18章 接頭內梁主筋彎鉤伸展長度ℓ_{dh}	$\left(\frac{0.06 f_y \psi_e}{\sqrt{f_c'}}\right) d_b$	$\left(\frac{0.06 f_y \psi_e}{\sqrt{f_c'}}\right) d_b$
第18章 接頭內梁主筋擴頭伸展長度ℓ_{dt}	$\left(\frac{0.06 f_y \psi_e}{\sqrt{f_c'}}\right) d_b$	$\left(\frac{1.25 f_y \psi_e \psi_p \psi_o \psi_c}{32\sqrt{f_c'}}\right) d_b^{1.5}$

彎鉤沒有變

擴頭鋼筋錨定長度依圍束條件計算

(kgf/cm^2 unit)

UT Austin

James O. Jirsa

Kansas U

David Darwin

表25.4.4.3 擴頭竹節鋼筋受拉伸展長度之修正因數

修正係數	條件	因數值
環氧樹脂 ψ_e	環氧樹脂塗布或鋅與環氧樹脂雙層塗布鋼筋	1.2
	無塗布或鋅塗布 (鍍鋅) 鋼筋	1.0
平行肋筋 ψ_p	圍束D36以下鋼筋之肋筋 $A_{tt} \geq 0.3A_{hs}$ 或 S[1] $\geq 6d_b$ [2, 3]	1.0
	其他	1.6
位置 ψ_o	擴頭竹節鋼筋在 (1) 柱核心內終止且側面保護層 ≥ 6.5cm，或 (2) 側面保護層 $\geq 6d_b$	1.0
	其他	1.25
混凝土強度 ψ_c	f'_c < 420 kgf/cm² [42 MPa]	$(f'_c/1050) + 0.6$ [$(f'_c/105) + 0.6$]
	f'_c ≥ 420 kgf/cm² [42 MPa]	1.0

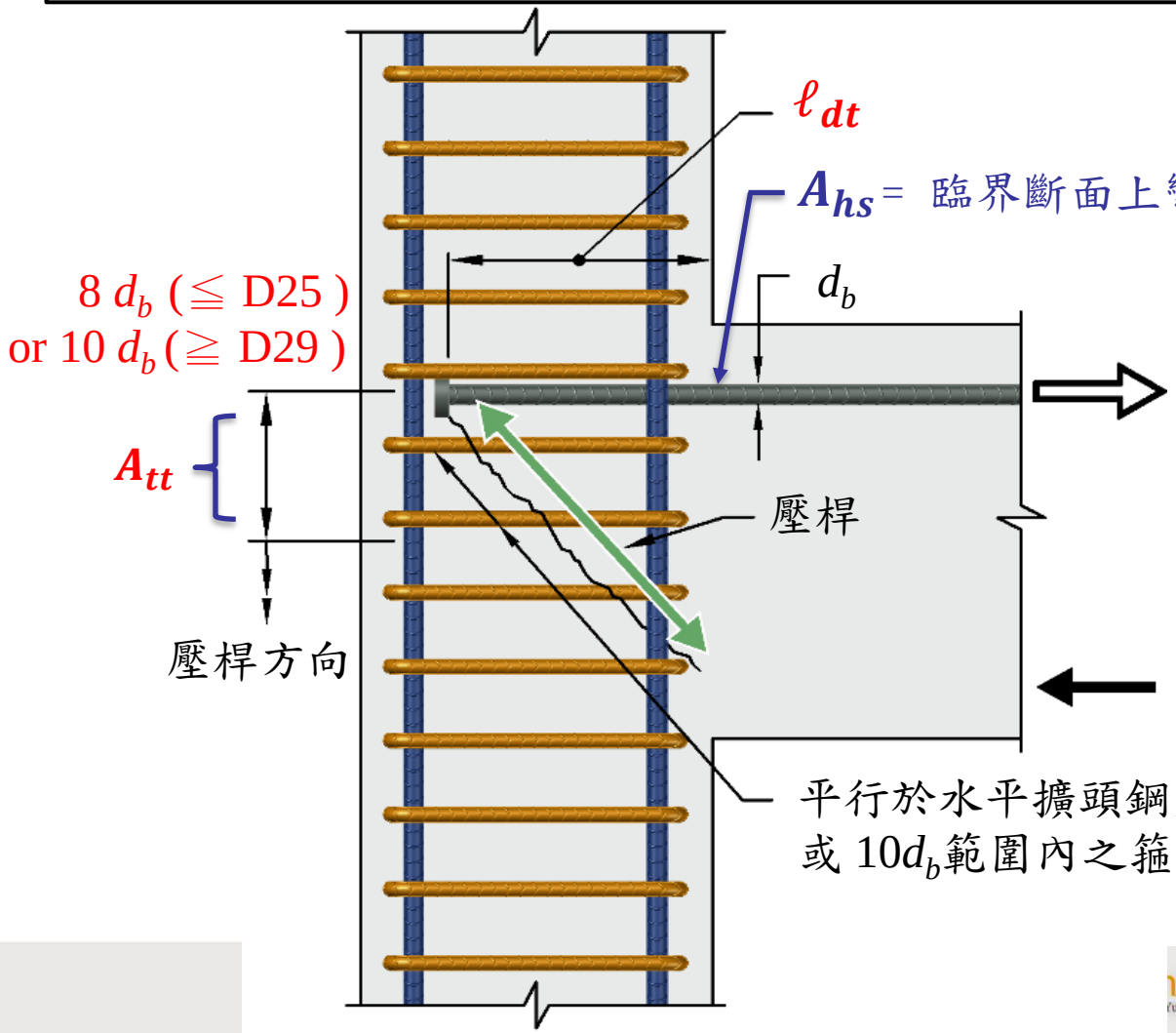
[1] s ＝ 擴頭竹節鋼筋之最小中心距。

[2] d_b ＝ 擴頭竹節鋼筋之標稱直徑。

[3] 參見第25.4.4.5節。

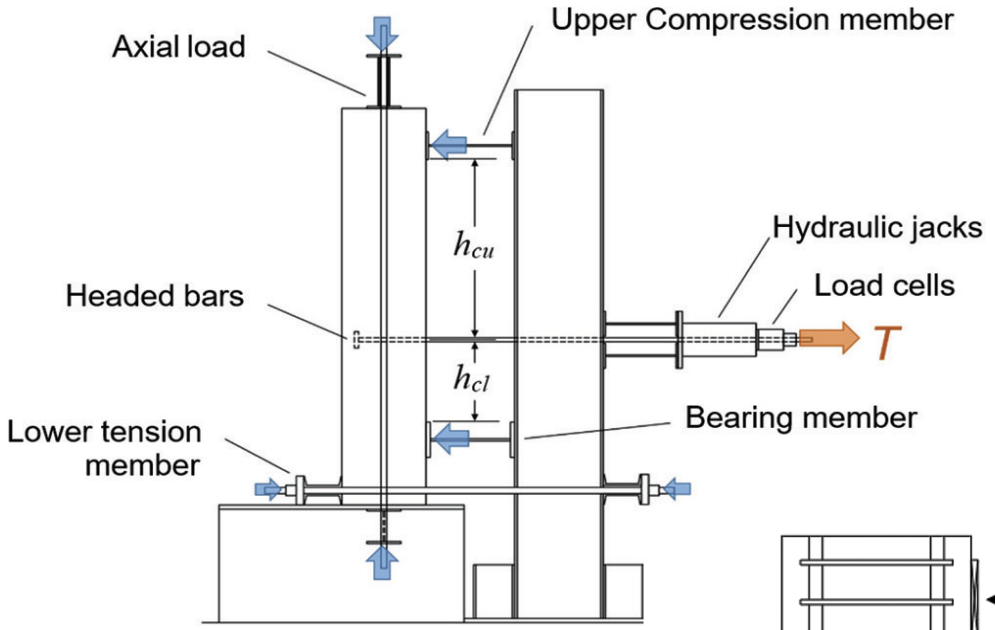
f'_c	ψ_c
210	0.80
280	0.87
350	0.93

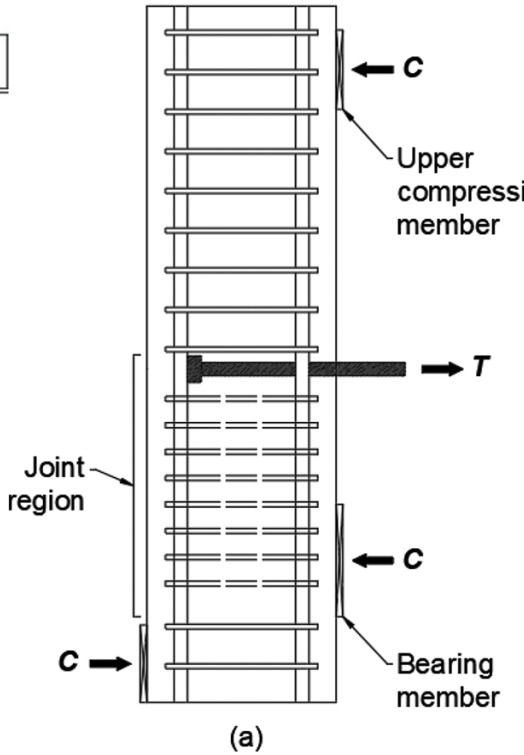
25.4.4.4 梁柱接頭區平行肋筋之總斷面積A_{tt}應包含平行於ℓ_{dt}之肋筋或箍筋，對D25或較小之擴頭竹節鋼筋而言，前述計入A_{tt}之肋筋或箍筋須配置於從擴頭竹節鋼筋中心起向接頭中心$8d_b$的範圍，對D29或較大之擴頭鋼筋而言，則允許該範圍擴大為$10d_b$，其中d_b為擴頭竹節鋼筋之標稱直徑。

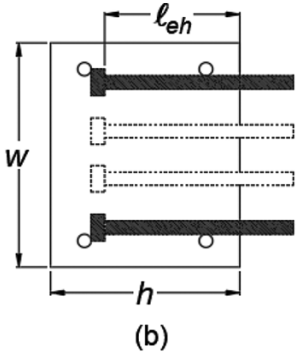


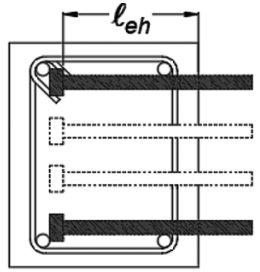
平行肋筋 ψ_p	圍束D36以下鋼筋之肋筋 $A_{tt} \geq 0.3A_{hs}$ 或 $S^{[1]} \geq 6d_b$ [2, 3]	1.0
	其他	1.6

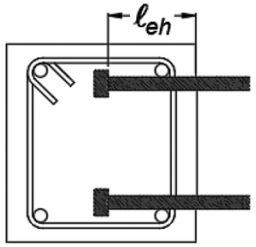
Monotonic tests in U of Kansas

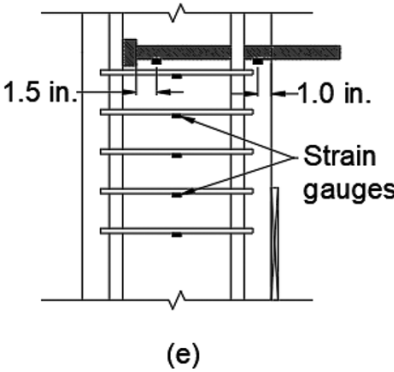












Ghimire, K. P.; Shao, Y.; Darwin, D.; and O'Reilly, M., "Conventional and High-Strength Headed Bars-Part 1: Anchorage Tests," ACI Structural Journal, 116(3), 2019, 255-264.

Monotonic tests in U of Kansas

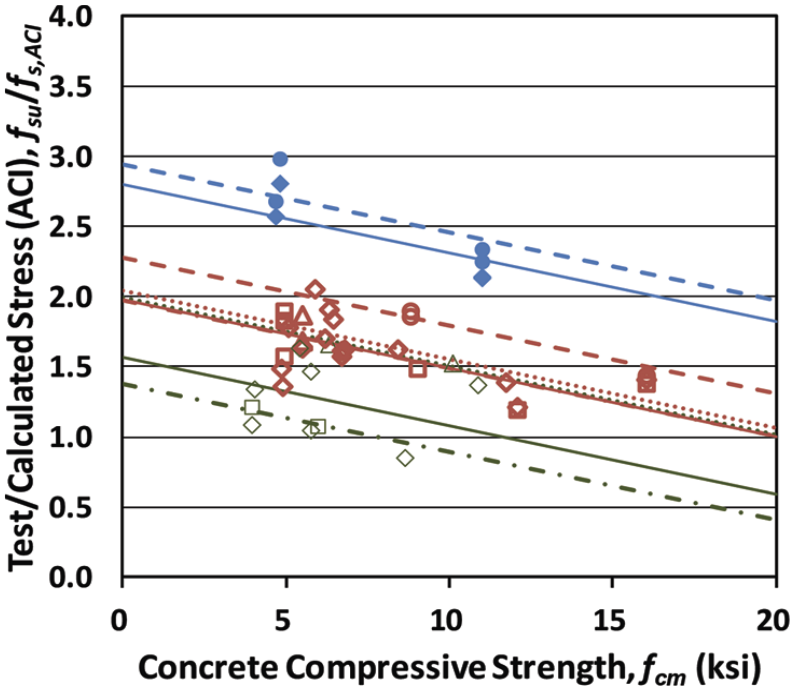


Fig. 8—Ratio of test-to-calculated stress $f_{su}/f_{s,ACI}$ *versus measured concrete compressive strength* f_{cm} *for specimens without confining reinforcement in joint region.*

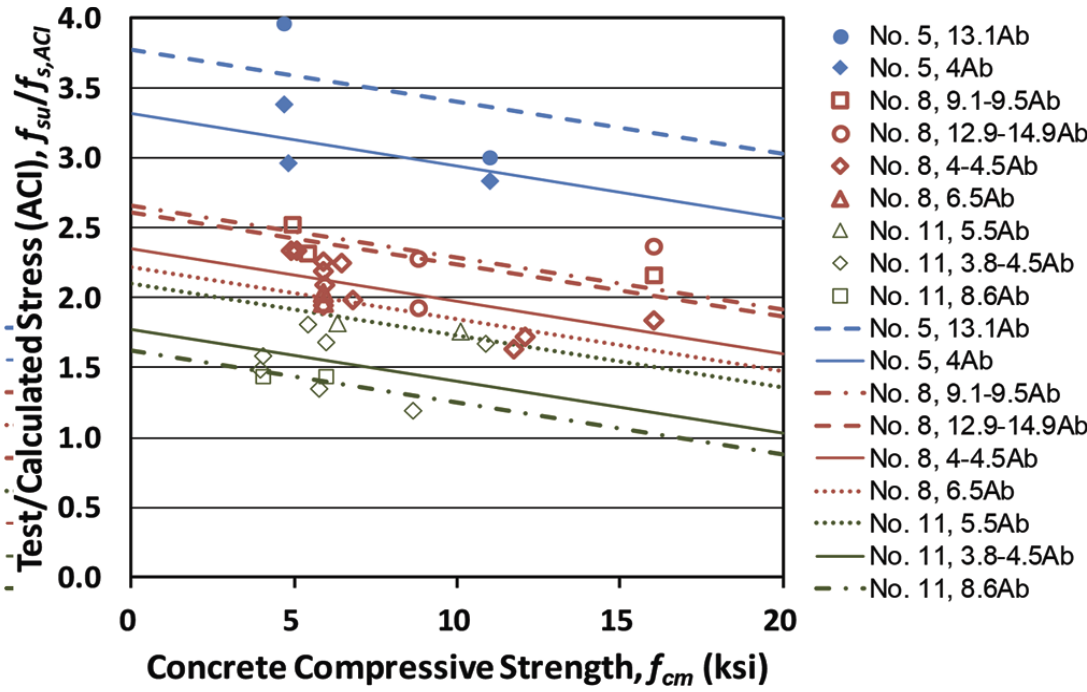


Fig. 10—Ratio of test-to-calculated stress $f_{su}/f_{s,ACI}$ *versus measured concrete compressive strength* f_{cm} *for specimens with No. 3 hoops spaced at* $3d_b$ *in joint region.*

評判 ACI 318-14的 ℓ_{dt} 對#5鋼筋過保守、#8鋼筋OK、#11不夠保守

Ghimire, K. P.; Shao, Y.; Darwin, D.; and O'Reilly, M., "Conventional and High-Strength Headed Bars-Part 1: Anchorage Tests," ACI Structural Journal, 116(3), 2019, 255-264.

Recent studies in U of Kansas

David Darwin

對 202 類接頭錨定試體作迴歸分析，建議2個經驗公式
(ACI 318-19錨定長度之基準)

$$T_h = 781 f'^{\,0.24}_{c,m}\, \ell^{\,1.03}_{eh}\, d_b^{\,0.35} \left(0.0836 \frac{c_{ch}}{d_b} + 0.344\right) \quad \text{(in.-lb)} \quad (1a)$$

$$T_h = 132 f'^{\,0.24}_{c,m}\, \ell^{\,1.03}_{eh}\, d_b^{\,0.35} \left(0.0836 \frac{c_{ch}}{d_b} + 0.344\right) \quad \text{(SI)} \quad (1b)$$

with $0.0836 \frac{c_{ch}}{d_b} + 0.344 \le 1.0$

$$T_h = \left(781 f'^{\,0.24}_{c,m}\, \ell^{\,1.03}_{eh}\, d_b^{\,0.35} + 48800 \frac{A_{tt}}{n} d_b^{\,0.88}\right)\left(0.0622 \frac{c_{ch}}{d_b} + 0.543\right) \quad \text{(in.-lb)} \quad (2a)$$

$$T_h = \left(132 f'^{\,0.24}_{c,m}\, \ell^{\,1.03}_{eh}\, d_b^{\,0.35} + 19.5 \frac{A_{tt}}{n} d_b^{\,0.88}\right)\left(0.0622 \frac{c_{ch}}{d_b} + 0.543\right) \quad \text{(SI)} \quad (2b)$$

with $0.0622 \frac{c_{ch}}{d_b} + 0.543 \le 1.0$ and $\frac{A_{tt}}{n} \le 0.3 A_b$

Recent studies in U of Kansas

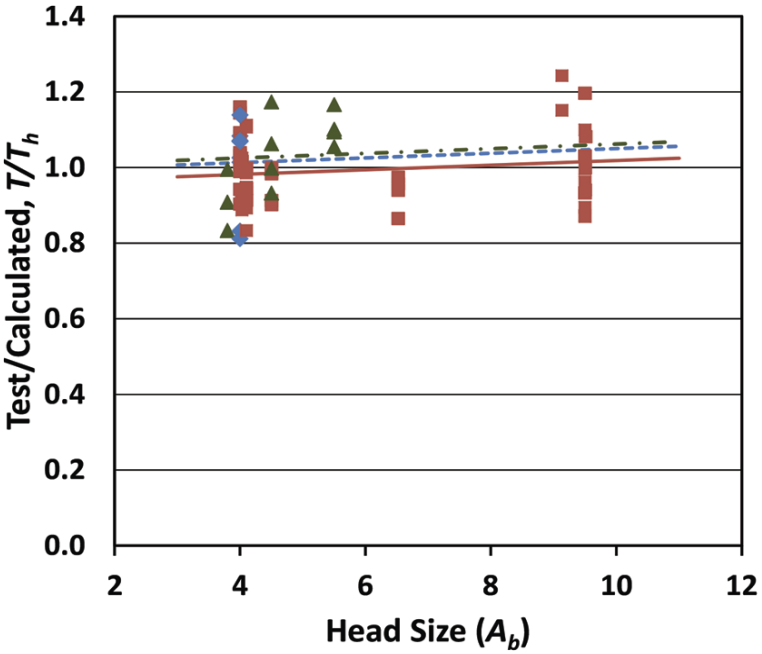


Fig. 10—Ratio of test-to-calculated failure load T/T$_h$ *versus head size for specimens with confining reinforcement.*

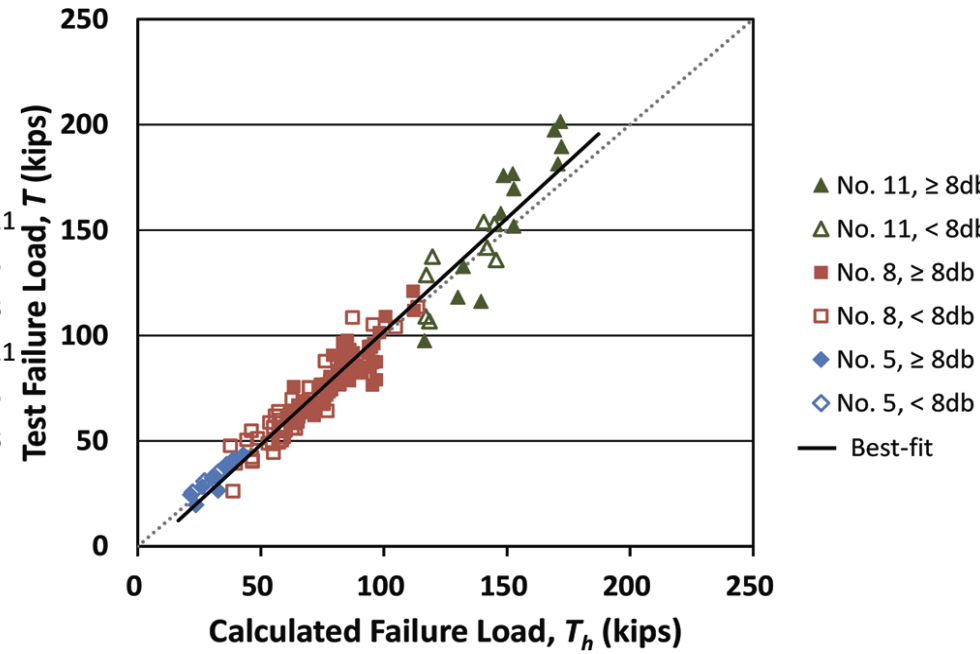
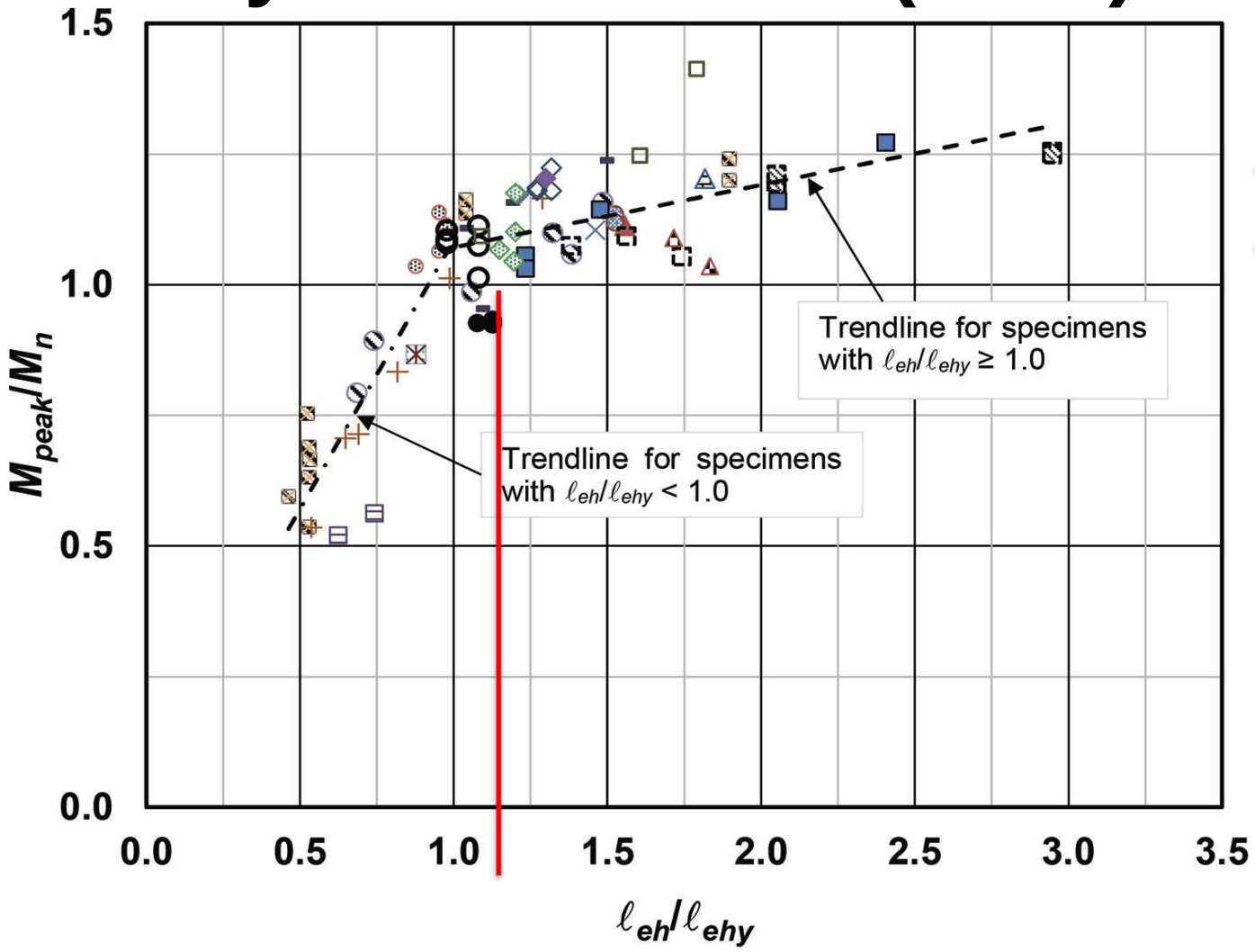


Fig. 11—Test failure load T *versus calculated failure load* T$_h$ *for specimens without and with confining reinforcement.* T$_h$ *is calculated based on Eq. (4) and (8). (Note: 1 kip = 4.448 kN.)*

Ghimire, K. P.; Shao, Y.; Darwin, D.; and O'Reilly, M., "Conventional and High-Strength Headed Bars-Part 2: Data Analysis," ACI Structural Journal, 116(3), 2019, 265-272..

Review 23 studies of BCJs by Ghimire et al. (2021)

這些是反復載重測試之梁柱接頭試體，但是…



- Bashandy (1996)
- Wallace et al. (1998)
- Nakazawa et al. (2000)
- Yoshida et al. (2000)
- Ishibashi et al. (2003)
- Kiyohara et al. (2004)
- Kato (2005)
- Adachi and Masuo (2007)
- Tazaki et al. (2007)
- Kang et al. (2012)
- Dhake et al. (2015)
- Linear ($\ell_{eh}/\ell_{ehy} \geq 1.0$)

- Murakami et al. (1998)
- Matsushima et al. (2000)
- Tasai et al. (2000)
- Takeuchi et al. (2001)
- Ishibashi and Inokuchi (2004)
- Kiyohara et al. (2005)
- Masuo et al. (2006)
- Chun et al. (2007)
- Lee and Yu (2009)
- Chun and Shin (2014)
- Linear ($\ell_{eh}/\ell_{ehy} < 1.0$)

Ghimire, K. P.; Darwin, D.; and Lepage, A., "Headed Bars in Beam-Column Joints Subjected to Reversed Cyclic Loading," ACI Structural Journal, 118(3), 2021, pp. 27-33.

以 Yoshida 的試體No.1-3為例，Ash遠低於ACI規範要求

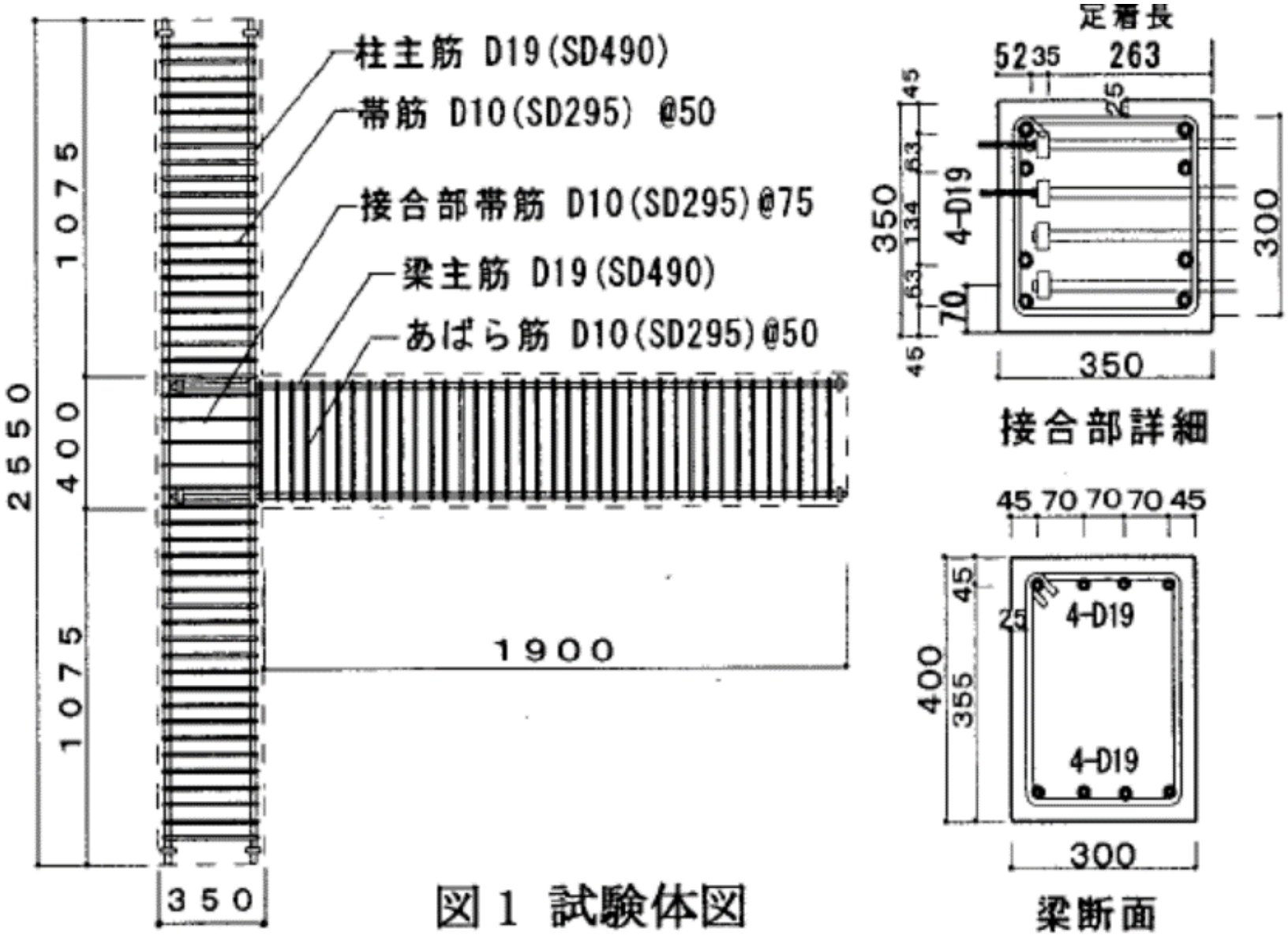
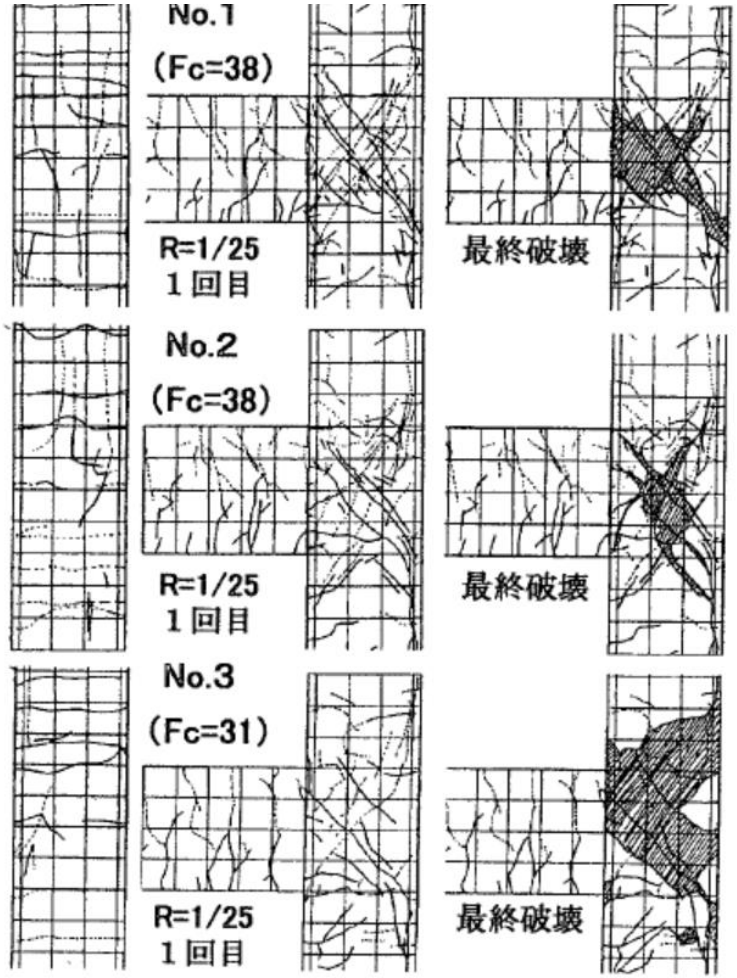


図 1 試験体図

吉田純子, 石橋一彦, 中村一彦, “23318 外柱梁接合部におけるナット定着板の性能に関する実験的研究：その1 試験体と実験概要,” 日本建築学會学術講演梗概集 2000年 9月



背面　　図4 ひび割れ状況

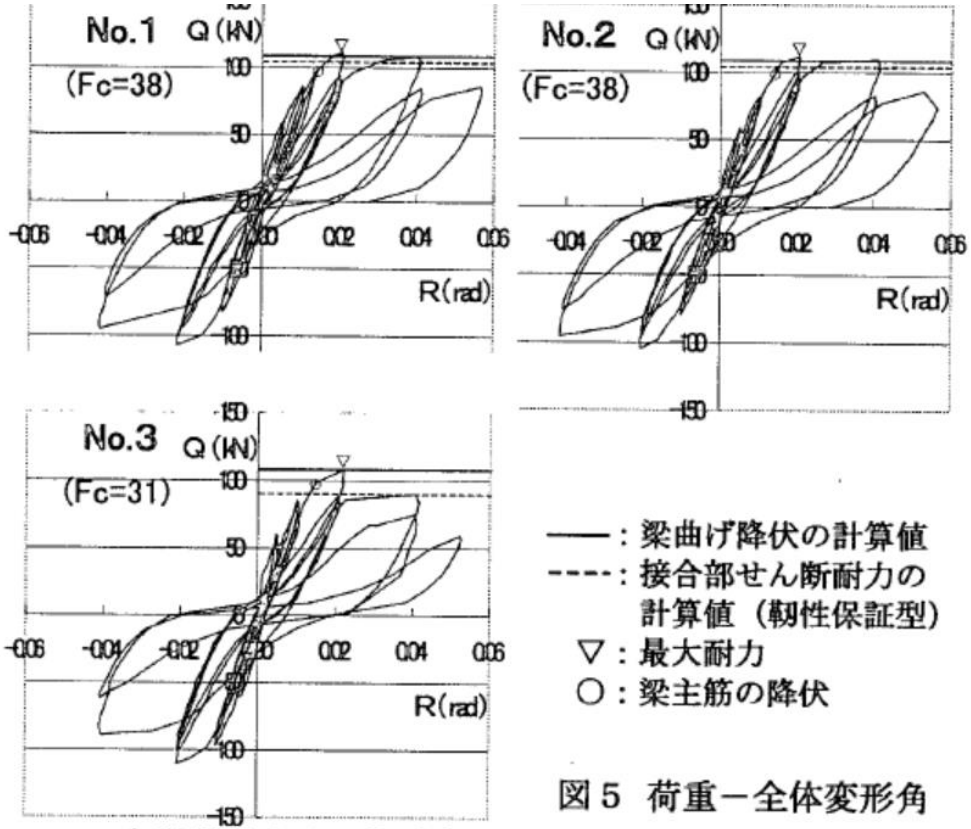
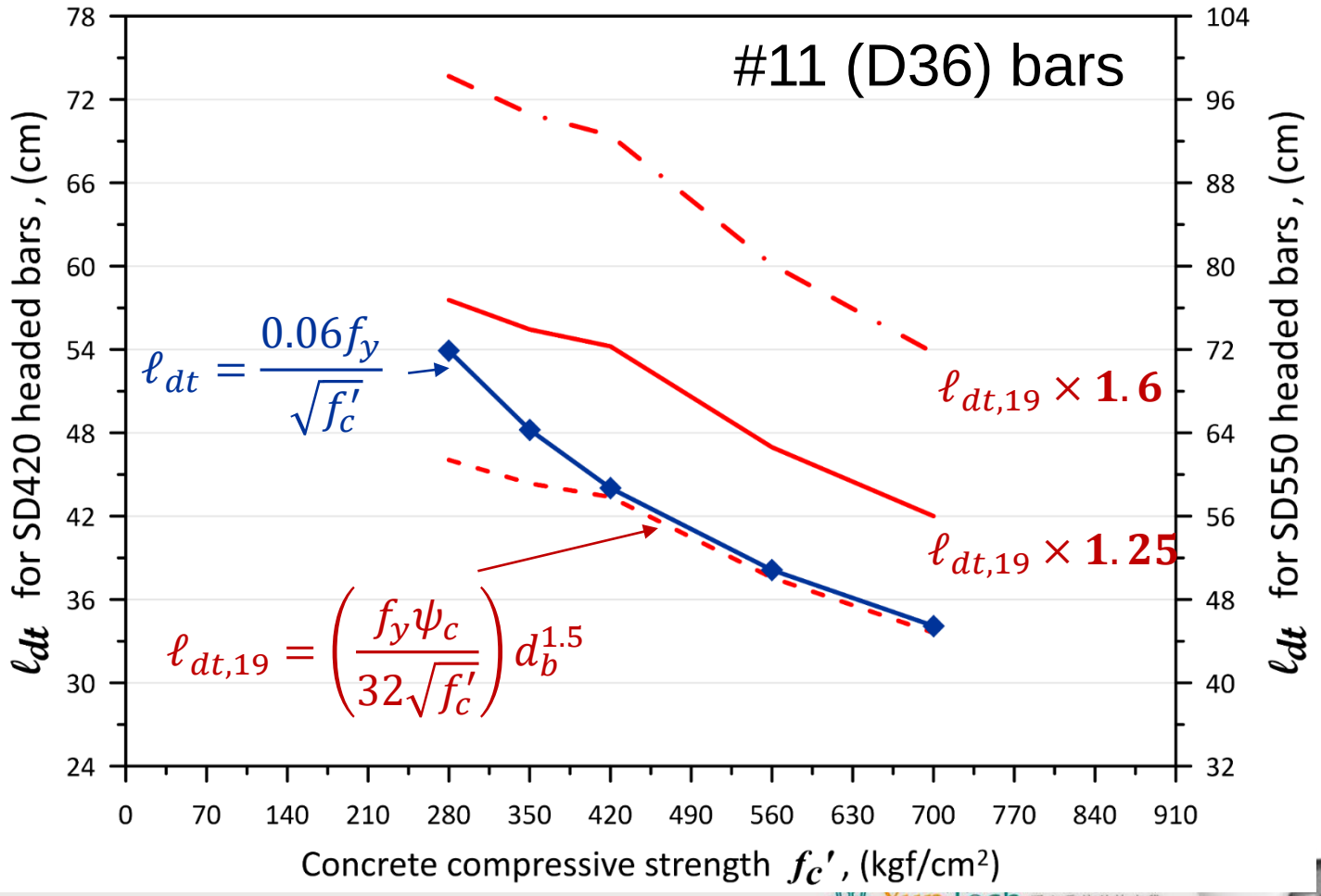


図5 荷重－全体変形角

各試験体とも、曲げ降伏耐力計算値に至るまでに剛性が低下し始めた。その剛性の低下は、No. 1、No. 2、No. 3の順に大きい。特に、コンクリート強度が低かったNo. 3で顕著である。これは、接合部せん断変形が他に

ACI 318-19 第18章 (土木401-110第18章)

18.5.5.2 符合第20.2.1.6節規定之擴頭竹節鋼筋，其**受拉伸展應符合第25.4.4節**之規定，惟梁縱向鋼筋應以**1.25f_y**取代 f_y 進行計算…



我方最新研究之接頭設計參數

Parameter / Specimen	Materials				Beam	Joint	Col. Reinf.		Anchorage		
	f_y ksi (MPa)	f_{yt} ksi (MPa)	f'_c psi (MPa)	$\sqrt{f'_c}$ psi	A_s	$\frac{V_u}{V_n}$	$A_{st,col}$	$\frac{A_{tt}}{A_{hs}}$	$l_{dt,14}$ (d_b)	$l_{dt,19}$ (d_b)	ℓ_{eh} (d_b)
N4	60 (420)	40 (280)	4,000 (28)	63	4 #11	0.81	12 #8	0.39	15.2	16.3	16.2
N5					5 #11	1.01	12 #10	0.32		16.3	
N6					6 #11	1.23	12 #11	0.26		26.0	
M4	80 (550)	60 (420)	6,000 (42)	77	4 #11	0.86	12 #8	0.39	16.5	20.4	
M5					5 #11	1.08	12 #10	0.32		20.4	
M6					6 #11	1.31	12 #11	0.26		32.7	
H4	100 (690)	100 (690)	10,000 (69)	100	4 #11	0.81	12 #8	0.39	16.0	19.8	
H5					5 #11	1.01	12 #10	0.32		19.8	
H6					6 #11	1.23	12 #11	0.26		31.7	

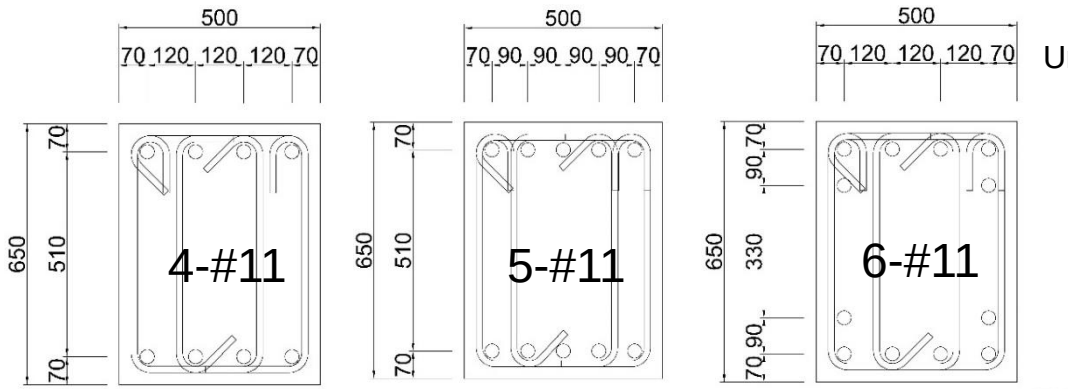
$$\ell_{dt,14} = \frac{0.016 f_y}{\sqrt{f'_c}} d_b[\text{英}] = \frac{0.192 f_y}{\sqrt{f'_c}} d_b[\text{SI}] = \frac{0.06 f_y}{\sqrt{f'_c}} d_b\ [\text{公}]$$

4-#11 ($\frac{V_u}{V_n} \approx 0.8$)

5-#11 ($\frac{V_u}{V_n} \approx 1.0$)

6-#11 ($\frac{V_u}{V_n} \approx 1.2$)

$M_R = 2.3 \sim 2.8$

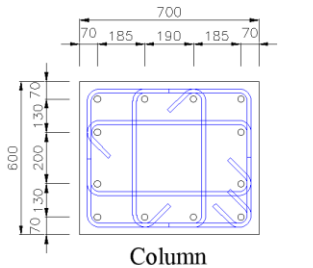


120/36=3.33d_b 90/36=2.5d_b 120/36=3.33d_b

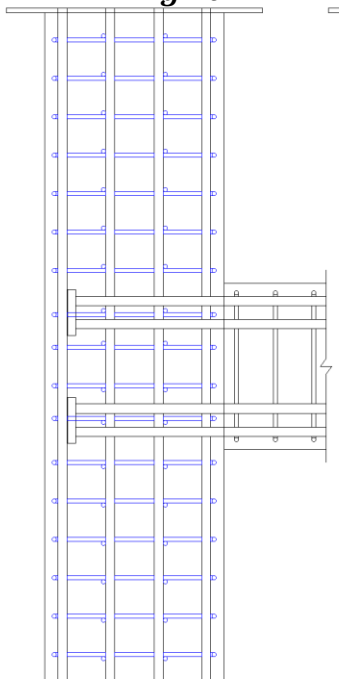
ACI 要求間距至少3d_b

Design parameters for confinement

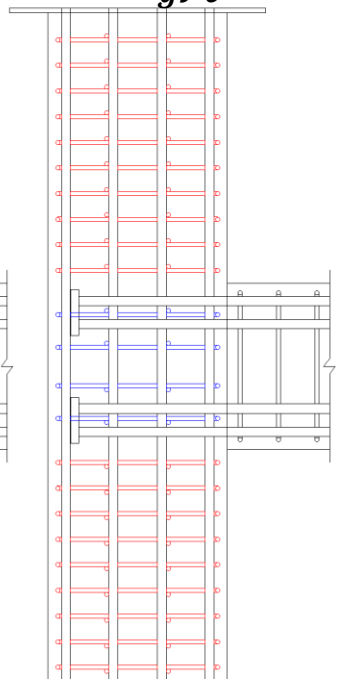
Group	f_{yt} (psi) [MPa]	f_c' (psi) [MPa]	A_{sh}/sb_c				
			$P = 0.10A_g f_c'$		$P = 0.45A_g f_c'$		
			Code	Col. & Joint	Code	Col.	Joint
N	40 [280]	4,000 [28]	0.009	0.010	N.A.		
M	60 [420]	6,000 [42]			0.015	0.015	0.010
H	100 [690]	10,000 [69]					

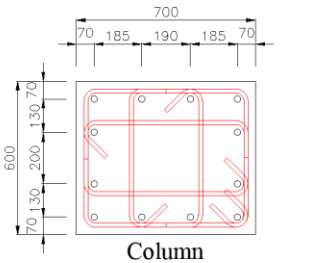


$0.10A_g f_c'$



$0.45A_g f_c'$





$$\frac{A_{sh}}{sb_c} \geq 0.3\frac{f_c'}{f_{yt}}\left(\frac{A_g}{A_{ch}} - 1\right) \quad \text{(a)}$$

$$\frac{A_{sh}}{sb_c} \geq 0.09\frac{f_c'}{f_{yt}} \quad \text{(b)}$$

$$\frac{A_{sh}}{sb_c} \geq 0.2k_f k_n \times \frac{P_u}{f_{yt}A_{ch}} \quad \text{(c)}$$

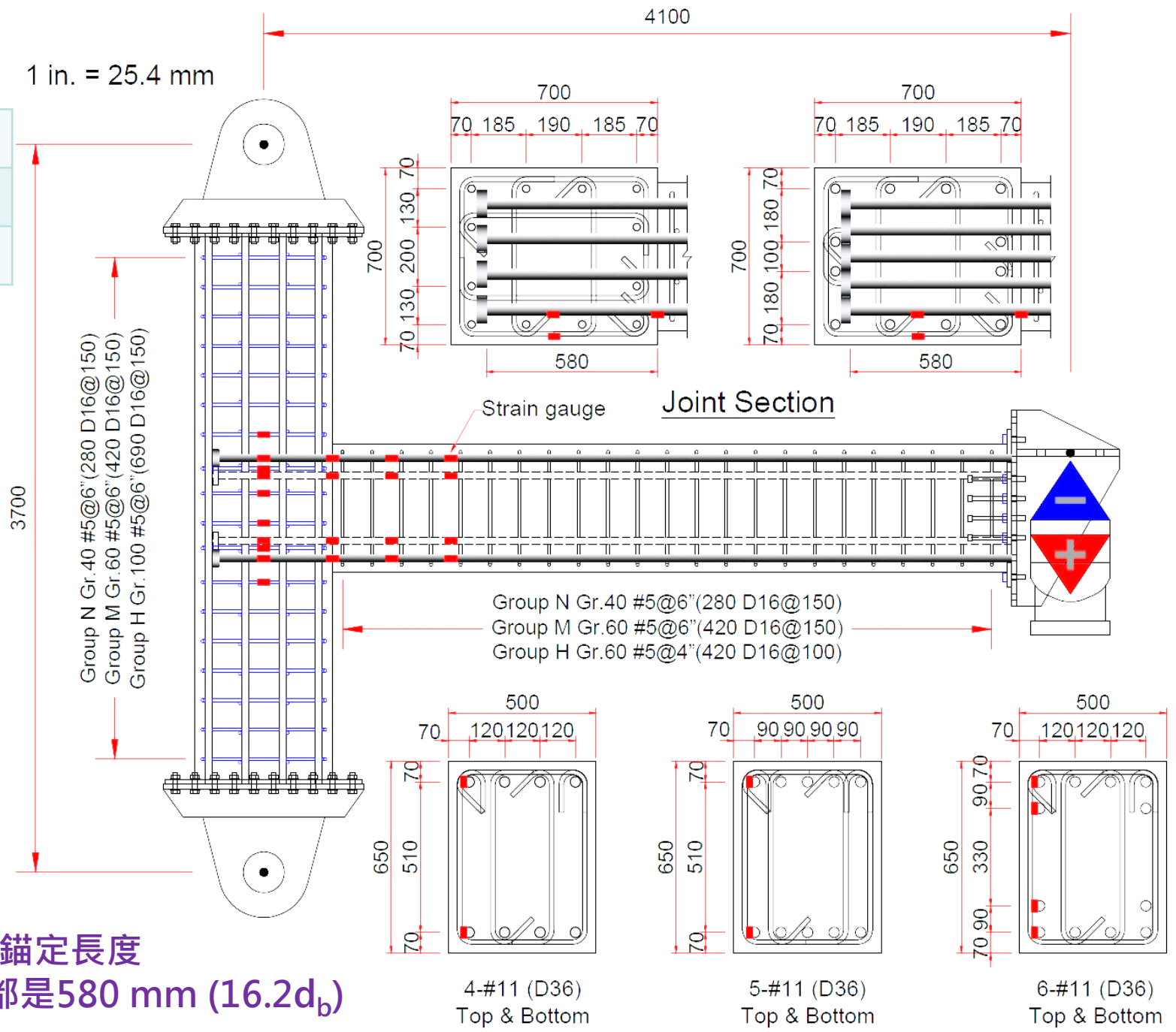
土木401-110 18.5.3.1
(c)式不適用接頭

N、M Group

H Group

N4、N5、N6
M4、M5、M6
H4、H5、H6

$$P = 0.1A_g f_c'$$



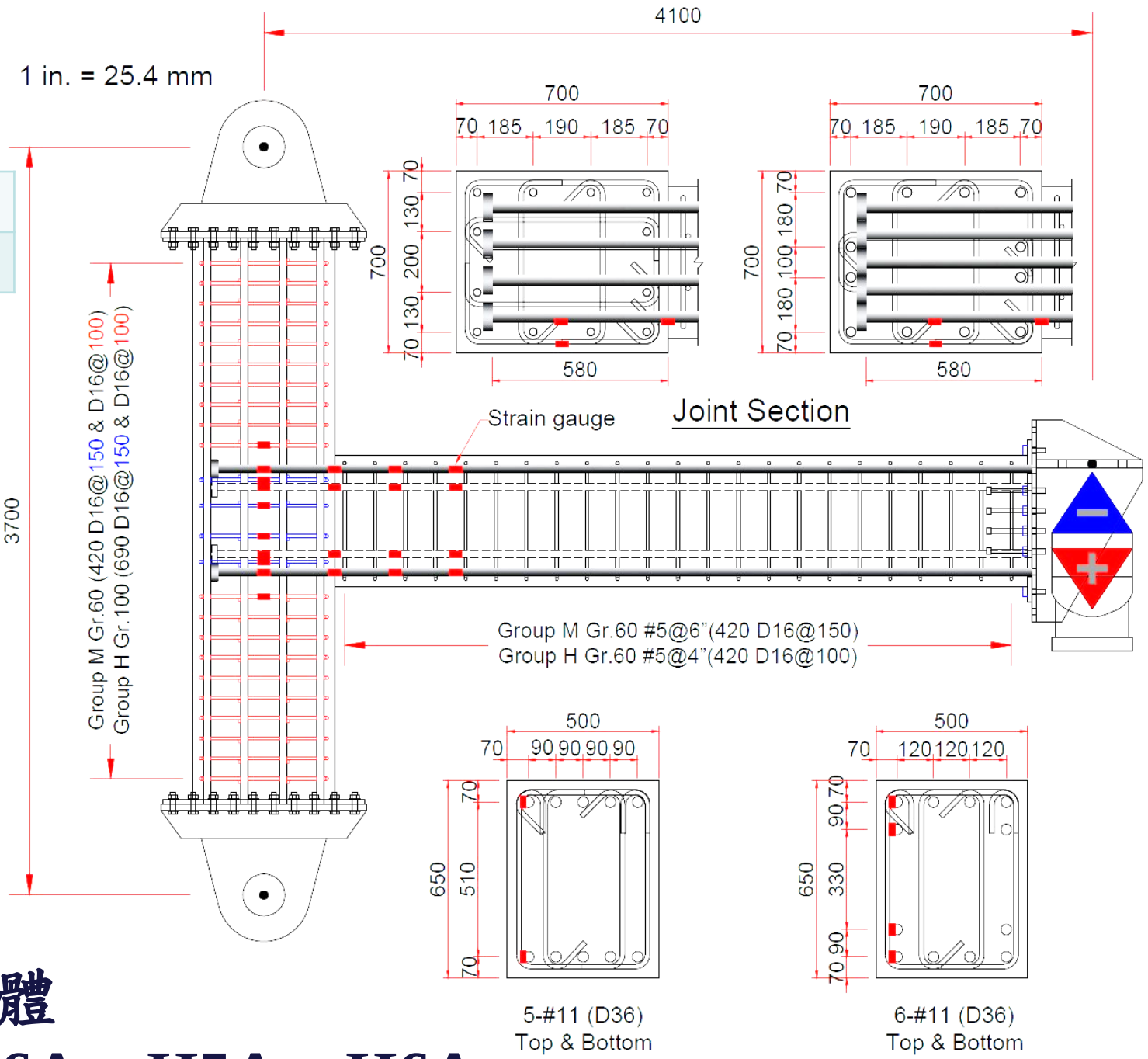
注意擴頭鋼筋錨定長度

N、M、H組都是580 mm ($16.2d_b$)

M5A、M6A

H5A、H6A

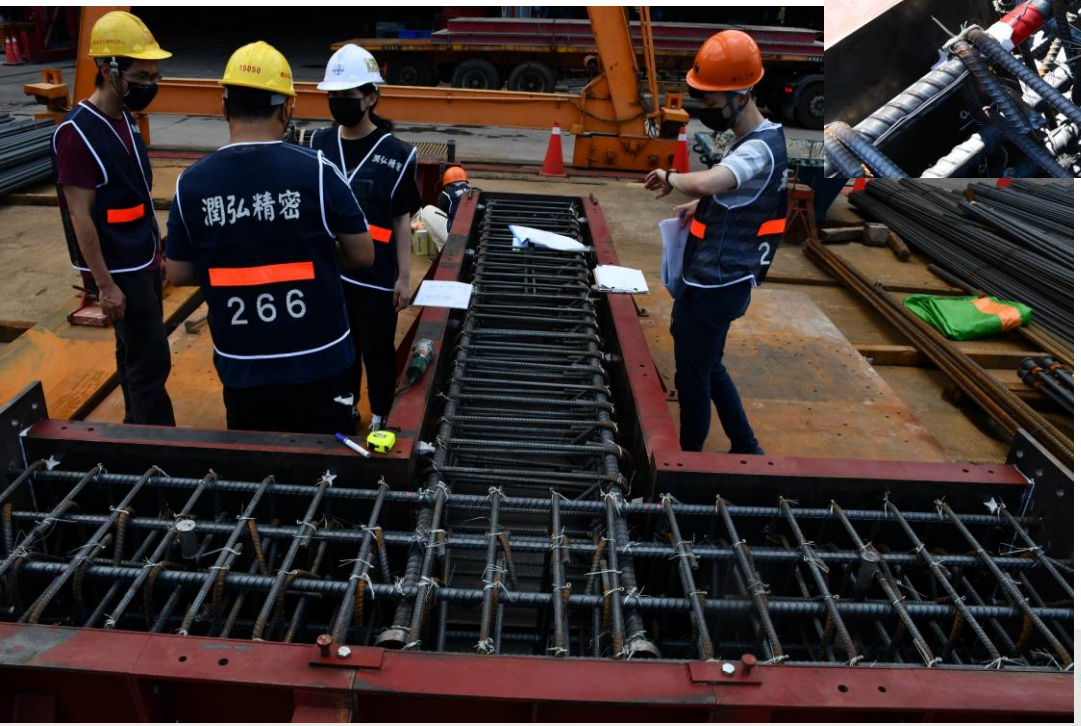
$$P = 0.45A_g f_c'$$



高軸力試體

M5A、M6A、H5A、H6A

試體製造@潤弘楊梅工廠

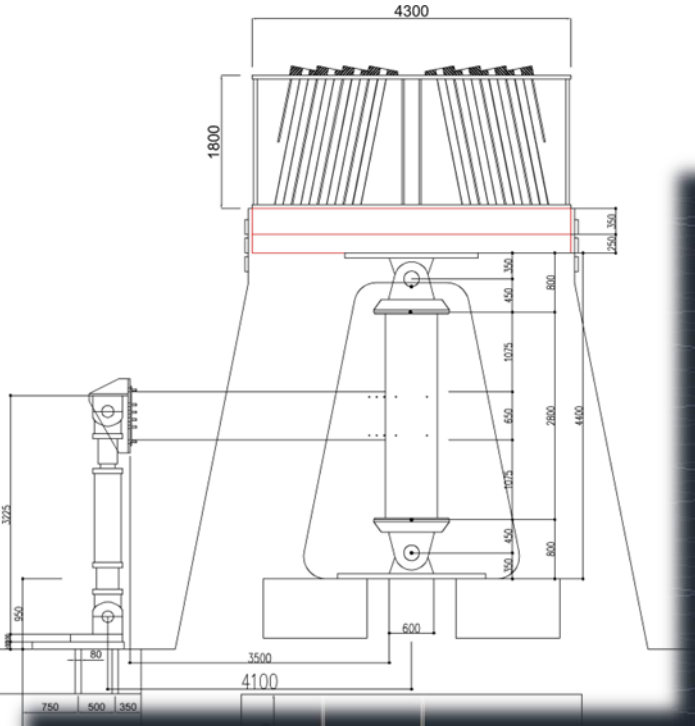


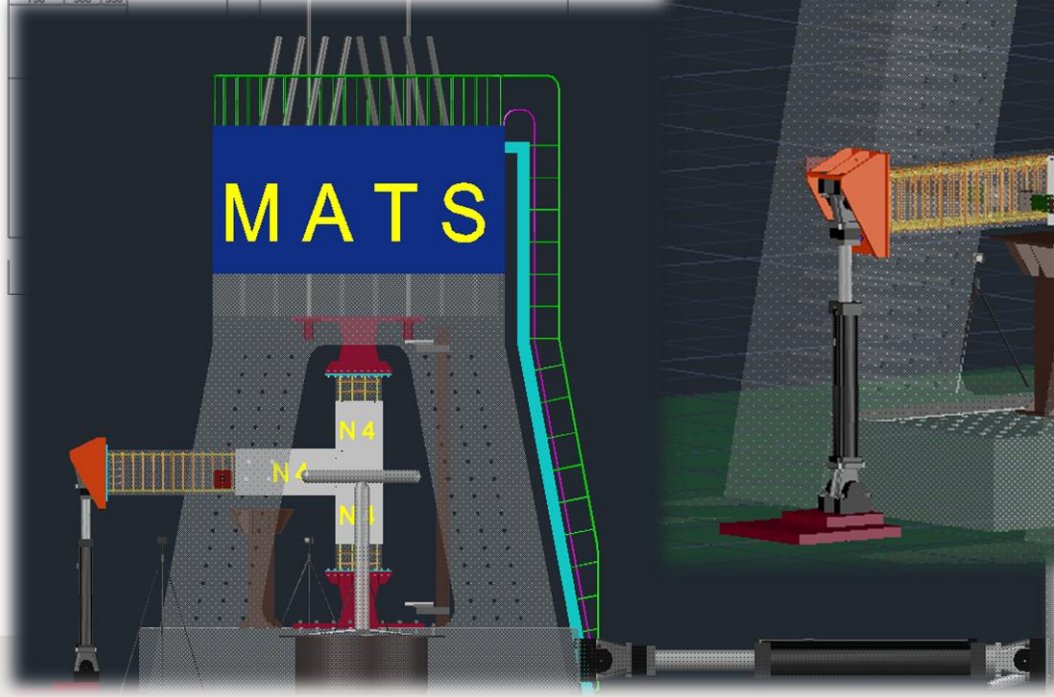
實際材料參數

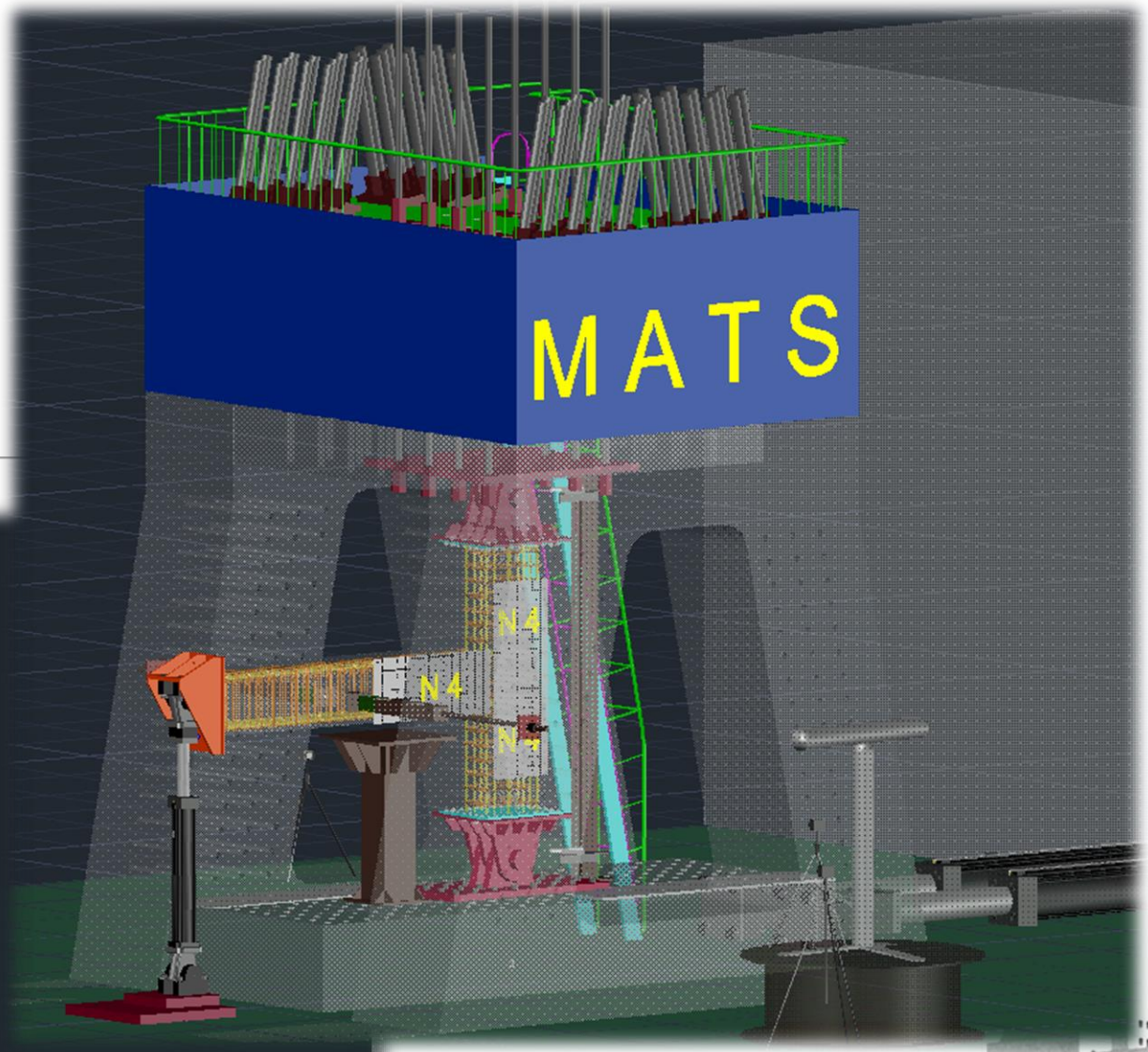
Group (f_c' / f_y)	Specimen	$f_{c,m}'$	$f_{y,m}$	$f_{u,m}$	$f_{u,m}/f_{y,m}$	*T-head* A_{brg}
N (28/420)	N4	39	462	649	1.41	5.2
	N5	34				
	N6	33				
M (42/550)	M4	43	568	722	1.27	5.3
	M5 (M5A)	50 (46)				
	M6 (M6A)	44 (44)				
H (69/690)	H4	79	749	962	1.29	5.3
	H5 (H5A)	70 (86)				
	H6 (H6A)	86 (76)				

$f_{c,m}'$ ：混凝土實際抗壓強度(MPa)　$f_{y,m}$ ：鋼筋實際抗拉強度(MPa)

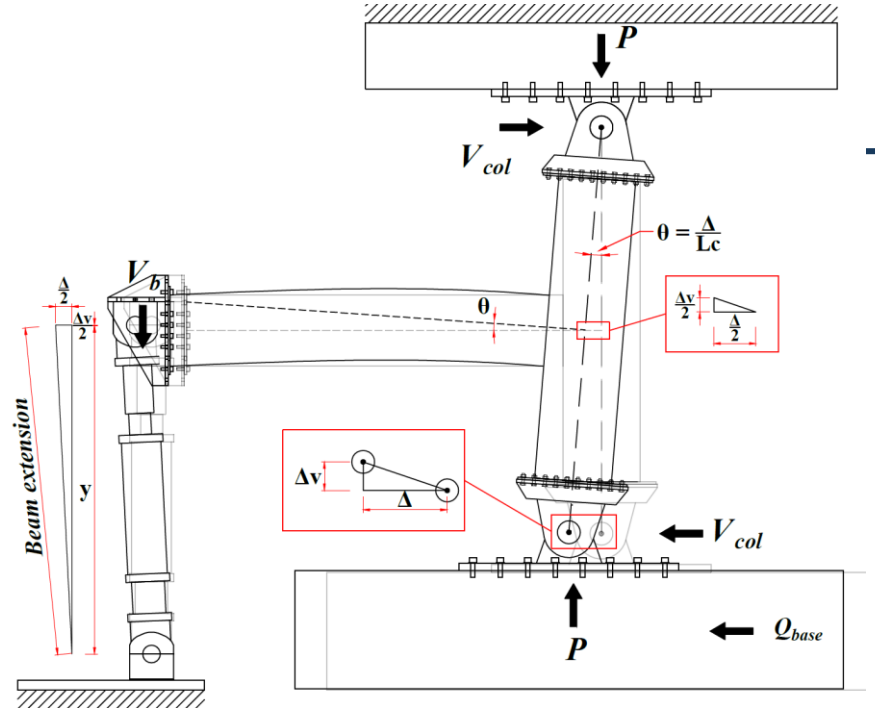
(A) : $0.45A_g f_c'$ 之軸力試體　$f_{u,m}$ ：鋼筋實際極限抗拉強度(MPa)

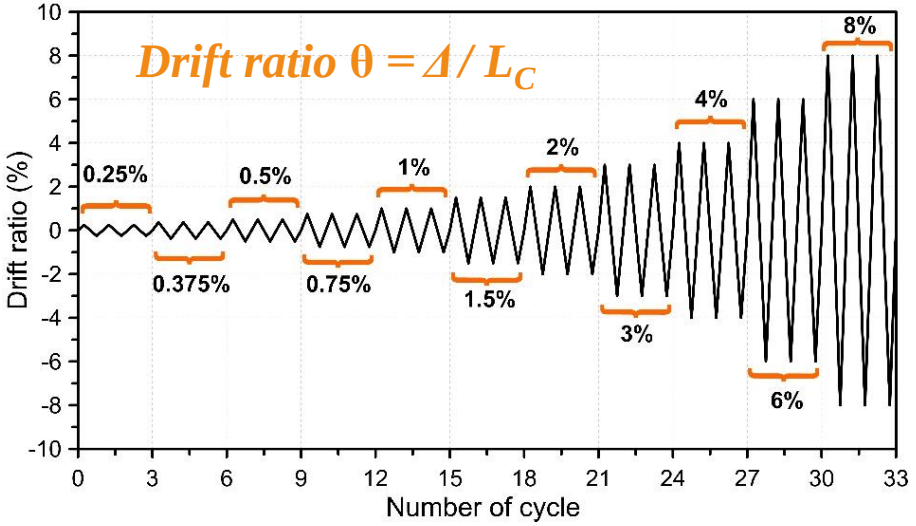

4300
1800
350
750
350
350
450
800
1075
650
2800
4400
1075
450
350
800
3225
950
600
80
3500
4100
750
500
350

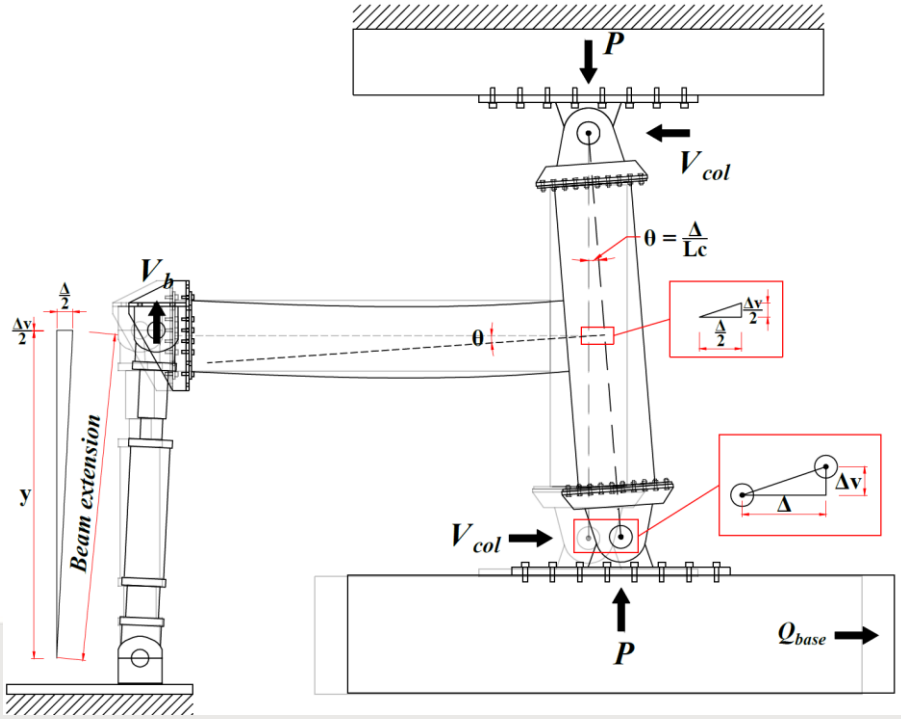

MATS
N4
N4
N4

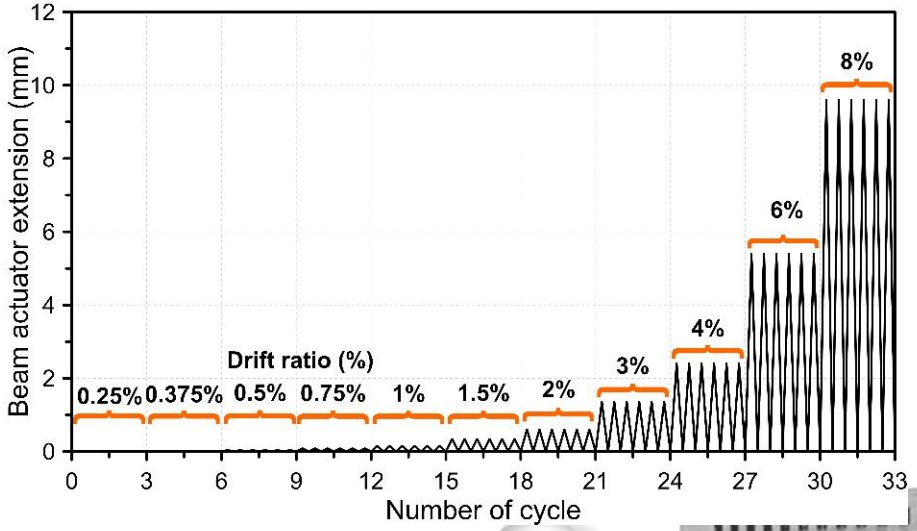

MATS
N4
N4
N4

Loading Procedures

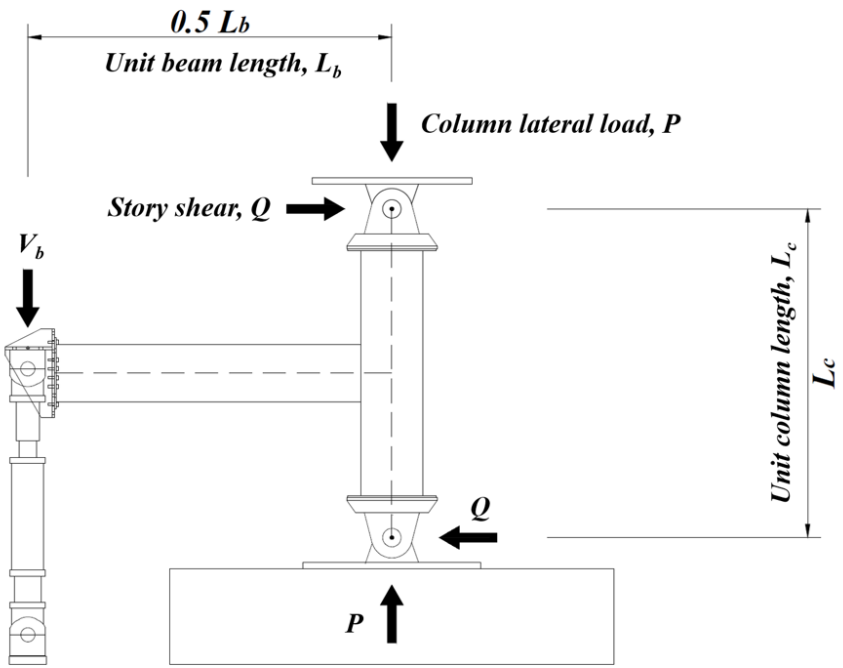


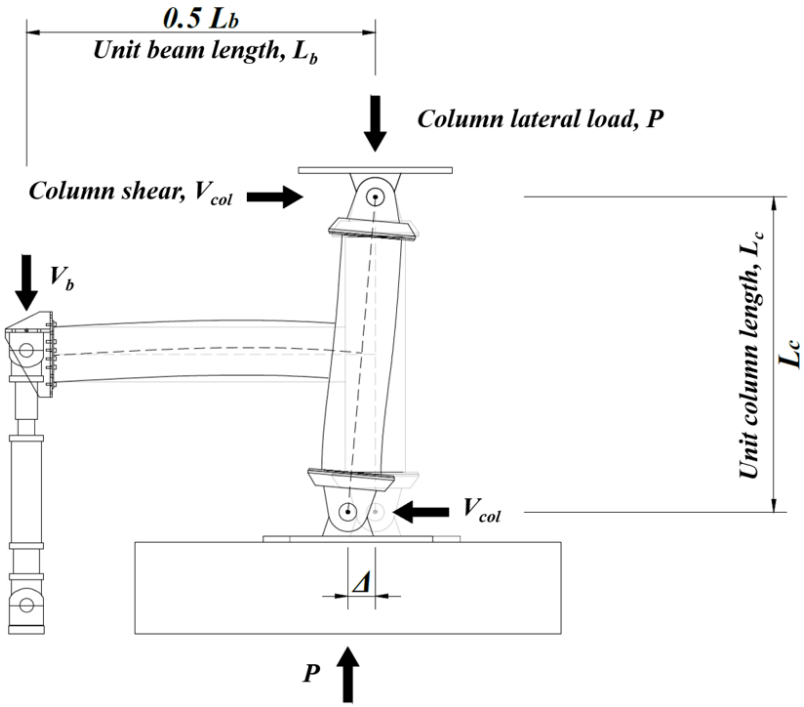






符號系統





$$Q \times L_c = V_b \times 0.5L_b = V_{col} \times L_c + P \times \Delta$$

$$\Rightarrow Q = V_b \times 0.5L_b/L_c = V_{col} + P \times \Delta/L_c \qquad \theta = \Delta/L_c$$

$$\Rightarrow Q = V_b \times 0.5L_b/L_c = V_{col} + P \times \theta$$

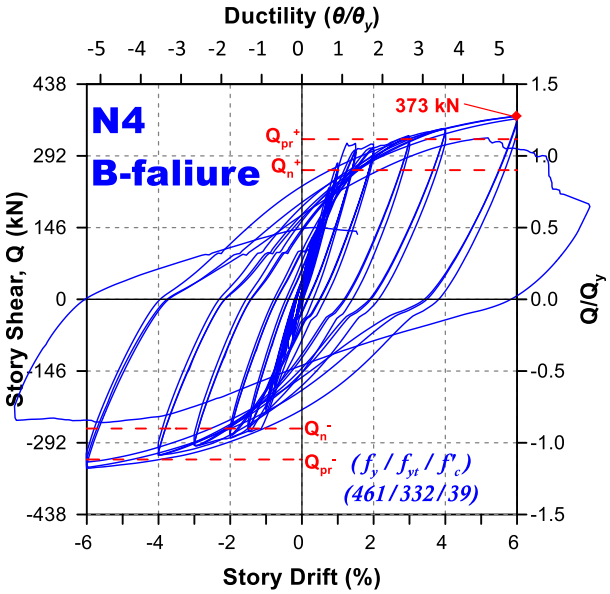
$$V_{col} = V_b \times 0.5L_b/L_c - P \times \theta$$

Q	Story shear
V_b	Beam shear
Δ	Story drift displacement
θ	Story drift ratio
V_{col}	Column shear (include PΔ effect)

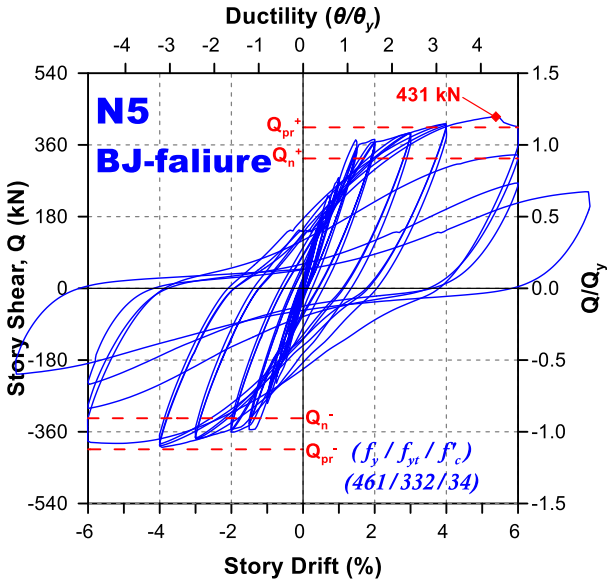
Group N

$(f_y/f_{yt}/f'_c)$=(420/280/28)

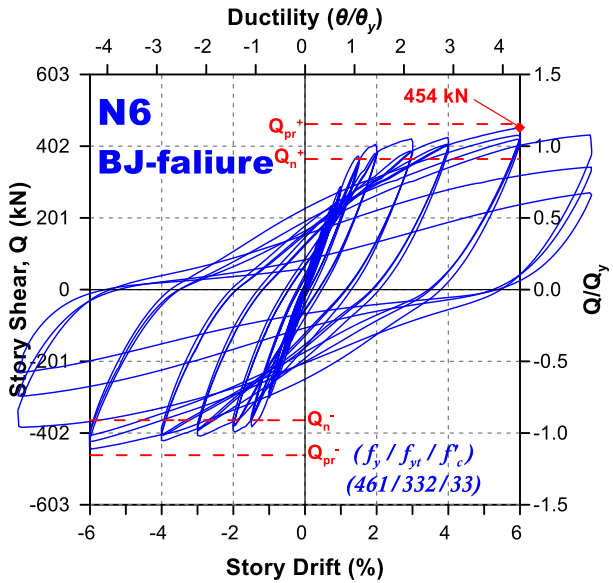
$Q_{max}/Q_y = 373/292 = 1.28$

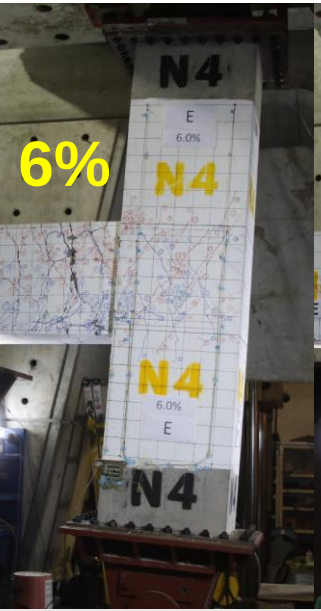


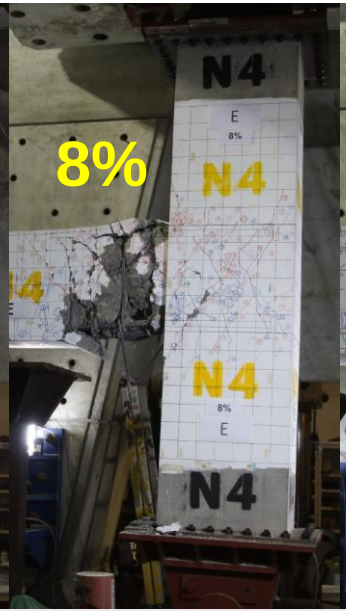
$Q_{max}/Q_y = 431/360 = 1.20$

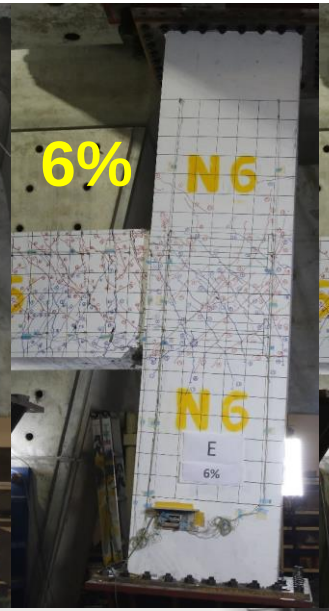


$Q_{max}/Q_y = 454/402 = 1.13$



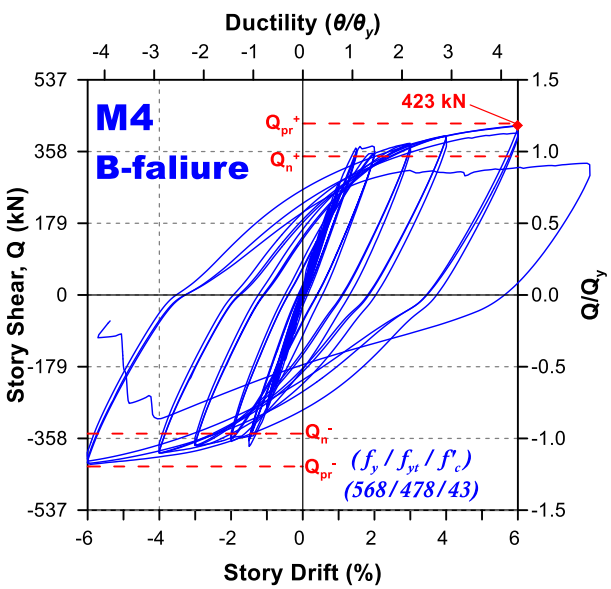




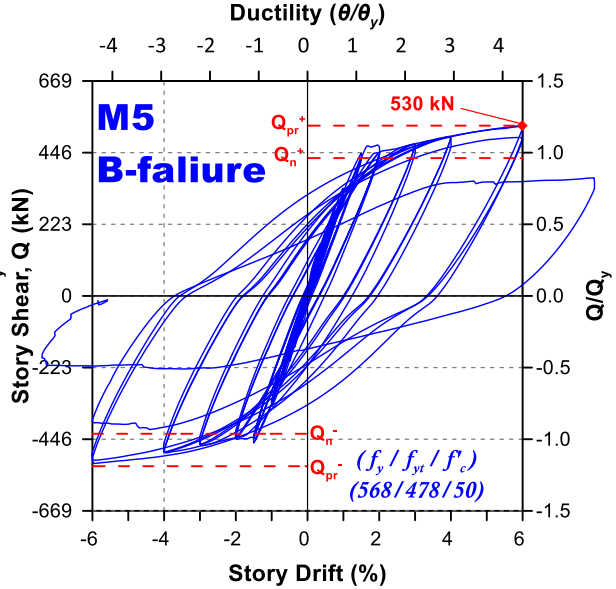


Group M $(f_y/f_{yt}/f'_c)$=(550/420/42)

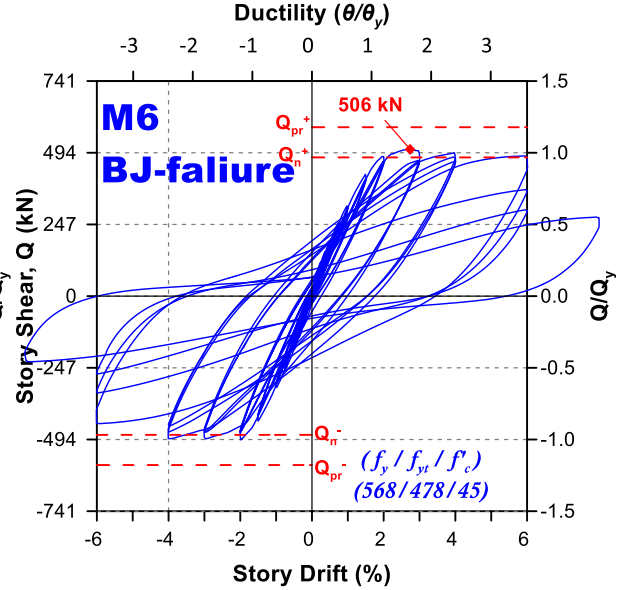
$Q_{max}/Q_y = 423/358 = 1.18$

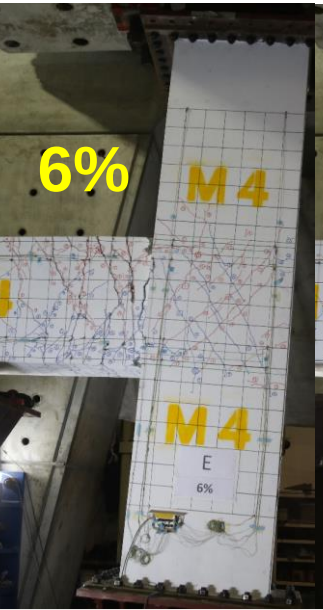


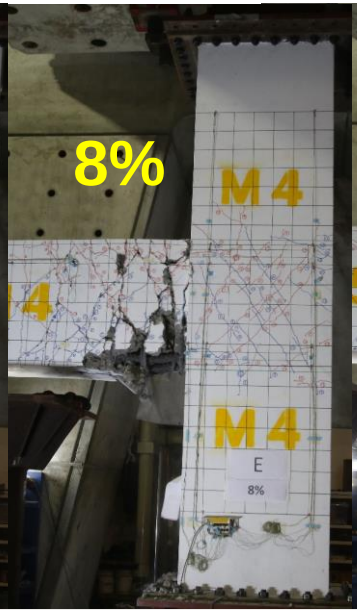
$Q_{max}/Q_y = 530/446 = 1.19$

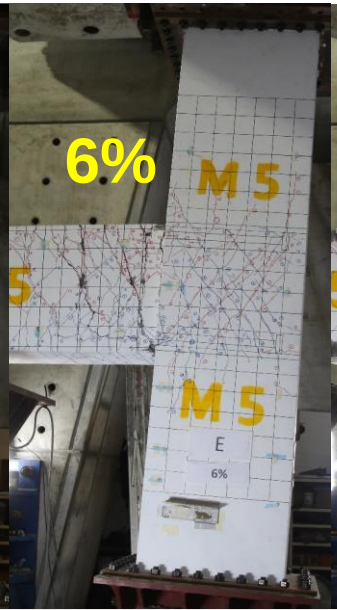


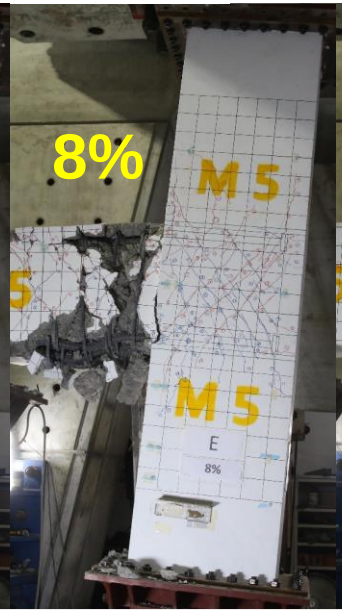
$Q_{max}/Q_y = 506/494 = 1.02$

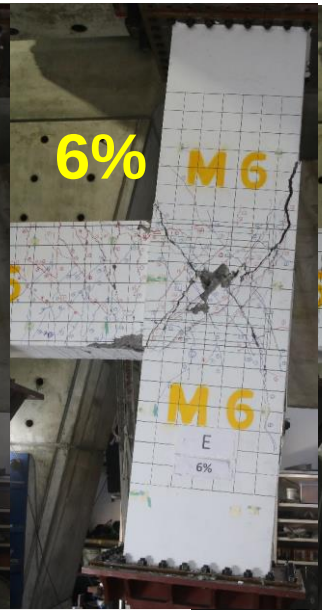


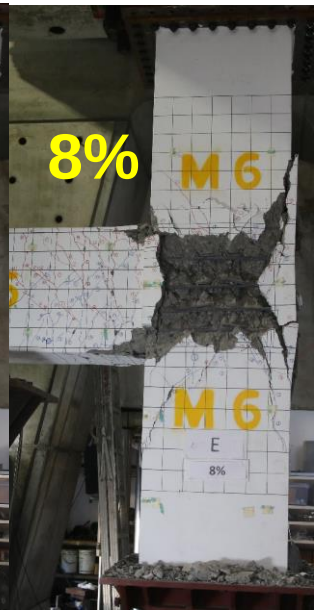






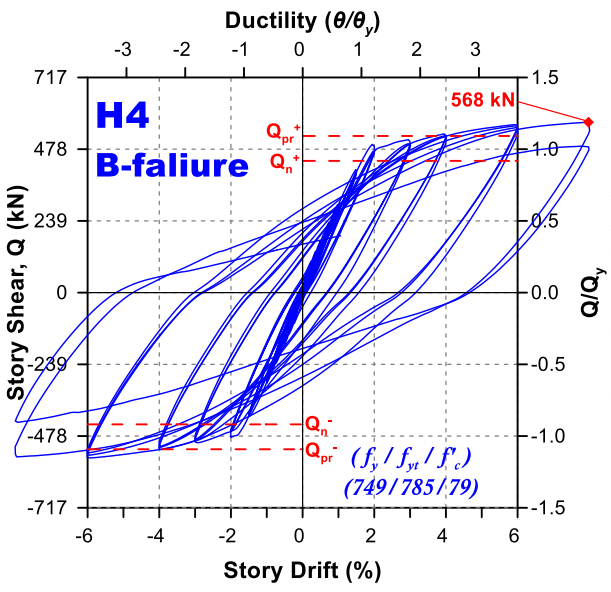




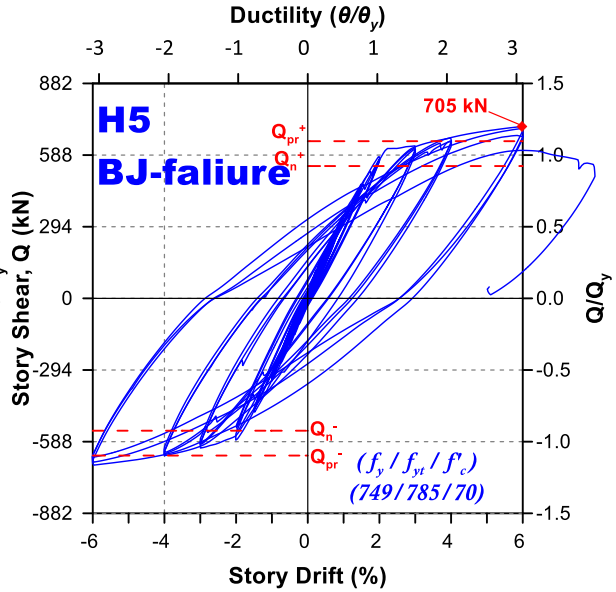


Group H $(f_y/f_{yt}/f'_c)$=(690/690/69)

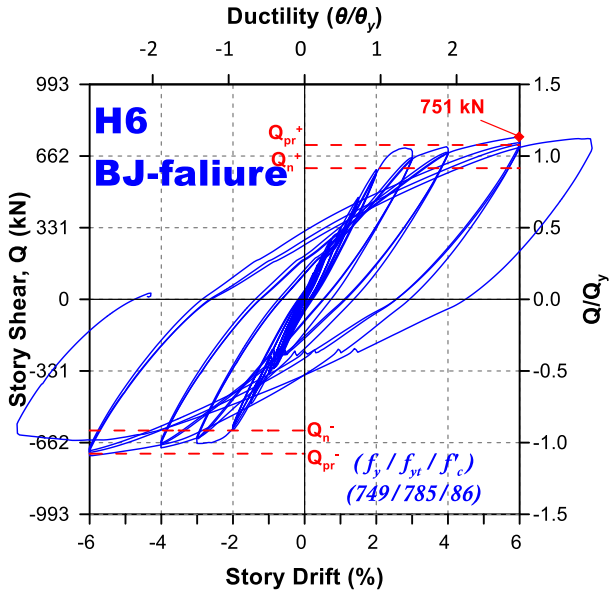
$Q_{max}/Q_y = 568/478 = 1.19$

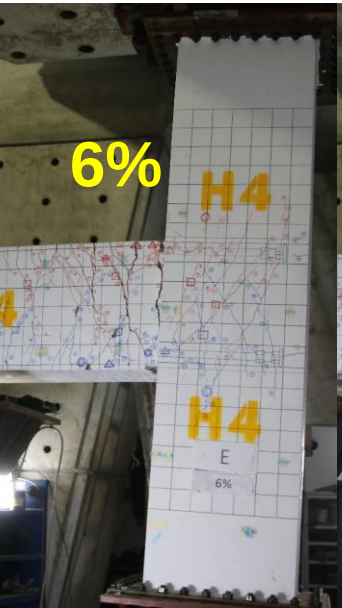


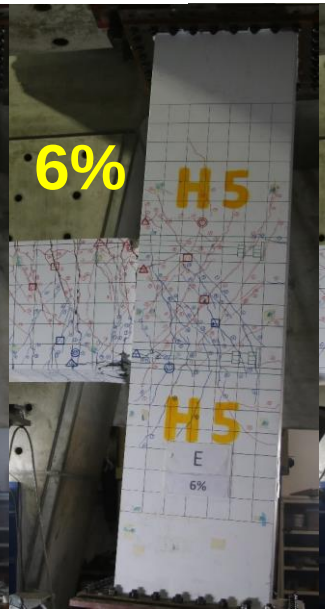
$Q_{max}/Q_y = 705/587 = 1.20$

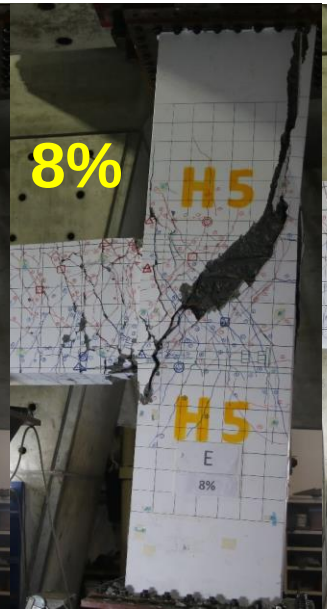


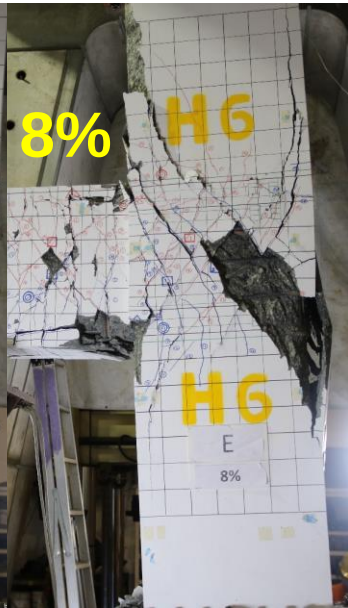
$Q_{max}/Q_y = 751/662 = 1.14$









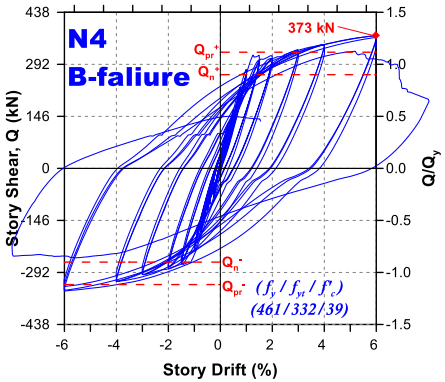


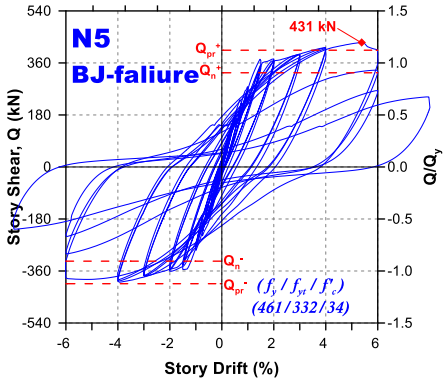
Global cyclic loading response

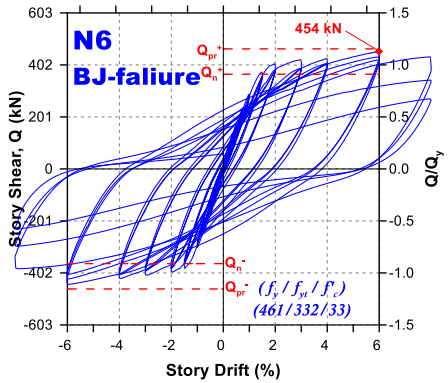
$(f_y/f_{yt}/f_c')$

N 組

(420/280/28)

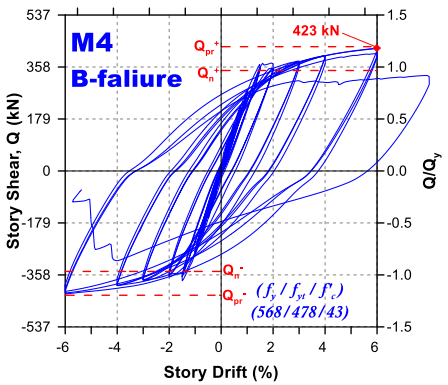


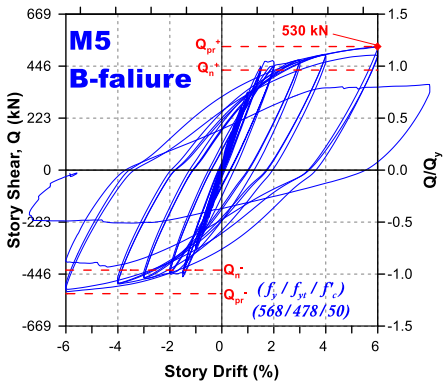


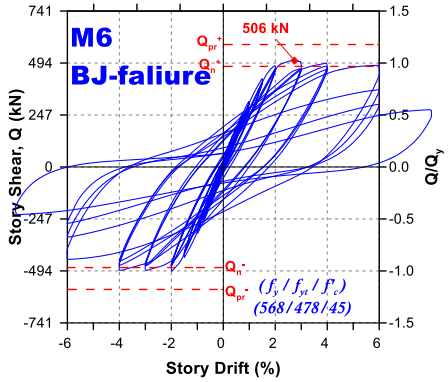


M 組

(550/420/42)

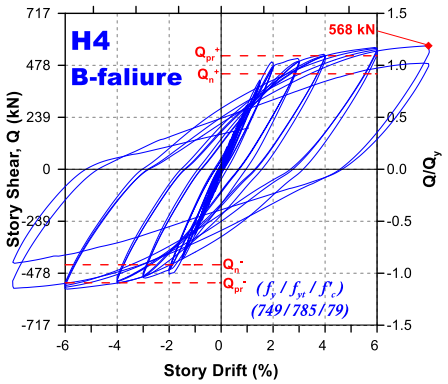


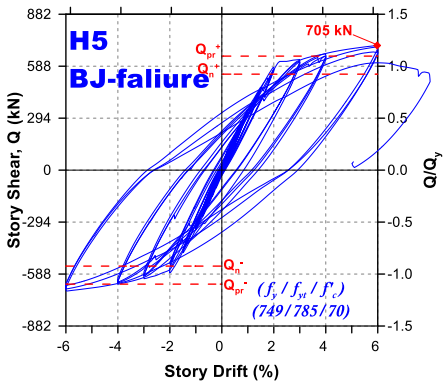


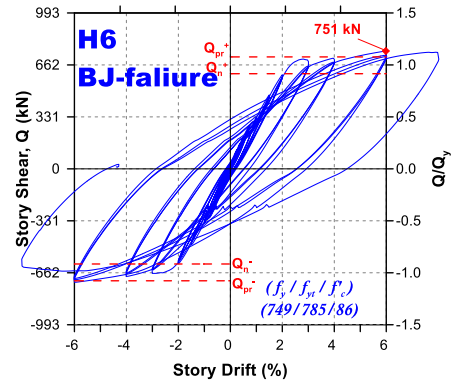


H 組

(690/690/69)



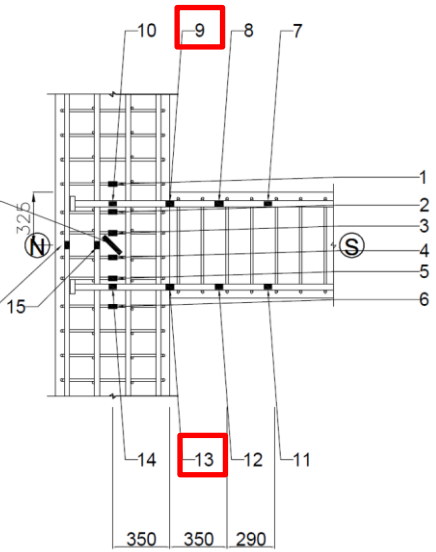


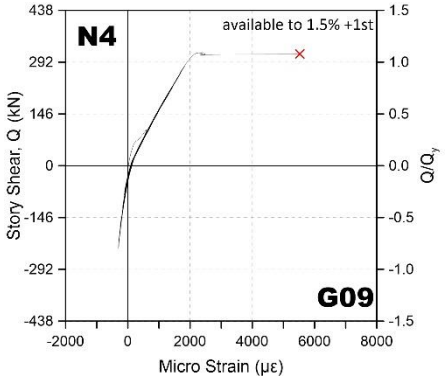


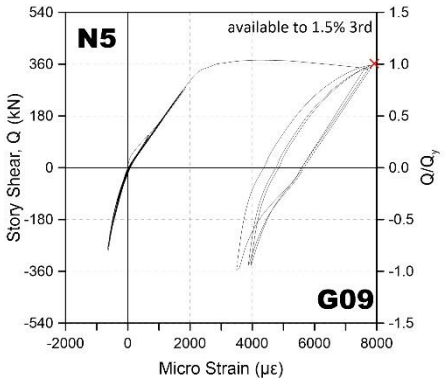
低軸力($P = 0.10A_g f_c'$)試體之試驗結果

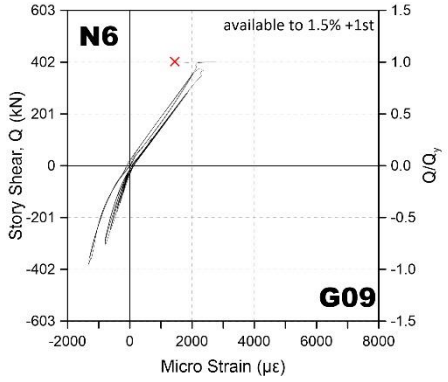
Parameter ($f_y/f_{yt}/f_c'$)	ID	$\frac{V_u}{V_n}$	Design Failure Mode	f_c'	Q_{max} (kN)	$\frac{f_{y,m}}{f_y}$	$\frac{Q_{max}}{Q_n}$	$\frac{Q_{max}}{Q_y}$	$\frac{Q_{max}}{Q_{pr}}$	$\frac{V_{jh,m}}{b_c \times h_c \times \sqrt{f_c'}}$	Test Failure Mode
N (420/280/28) α_o=1.25	N4	0.81	B	39	373	1.11	1.42	**1.28**	1.14	0.81	B
	N5	1.01	BJ	34	431		1.32	**1.20**	1.07	1.00	BJ
	N6	1.23	J	33	454		1.24	**1.13**	0.98	1.08	BJ
M (550/420/42) α_o=1.25	M4	0.86	B	43	423	1.03	1.22	**1.18**	0.99	0.88	B
	M5	1.08	BJ	50	530		1.24	**1.19**	1.00	1.01	B
	M6	1.31	J	45	506		1.06	**1.02**	**0.87**	1.03	**BJ**
H (690/690/69) α_o=1.20	H4	0.81	B	79	568	1.09	1.29	**1.19**	1.09	0.87	B
	H5	1.01	BJ	70	705		1.30	**1.20**	1.09	1.14	BJ
	H6	1.23	J	86	751		1.24	**1.14**	1.05	1.10	BJ

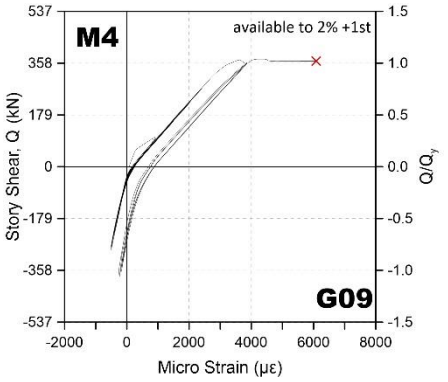
Local strain gauge readings

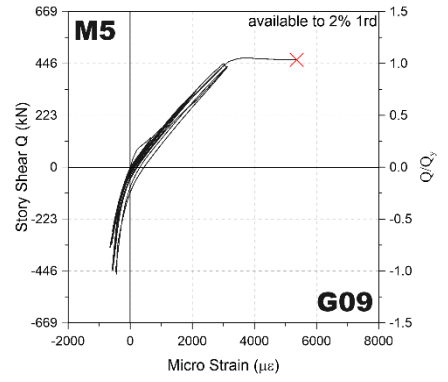


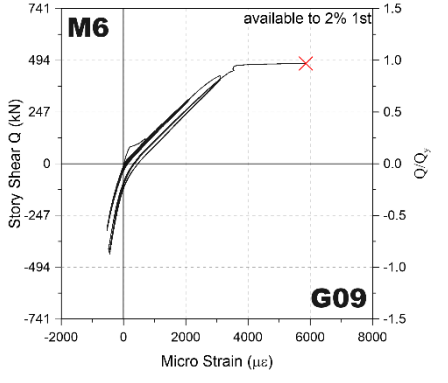


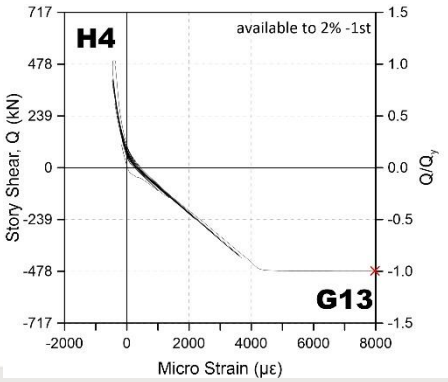


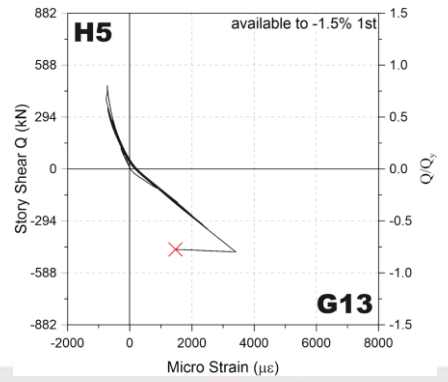


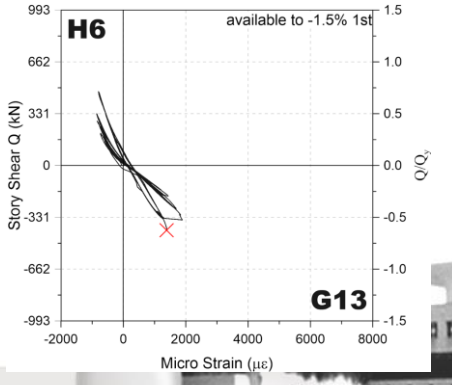












Cons for ℓ_{dt} nclusio

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

For Grade **60, 80, and 100** headed bars in SMF joints with
(a) #11 (36-mm) bars having $A_{brg}/A_b \approx 5.3$
(b) Joint transverse reinforcement conforming to ACI 318 Table 18.7.5.4(a)(b)
(c) Column subjected to an axial force of $0.10A_g f_c'$ at least

ACI 318-14 25.4.4.2

$$\ell_{dt} = \left(\frac{0.016 f_y \psi_e}{\sqrt{f_c'}}\right) d_b$$

psi unit

can be used with satisfactory anchorage performance

$$\ell_{dt} = \left(\frac{0.06 f_y \psi_e}{\sqrt{f_c'}}\right) d_b$$

kgf/cm^2 unit

ACI 318-19 18.8.5.2 For headed deformed bars satisfying 20.2.1.6, development in tension shall be in accordance with 25.4.4, by substituting a bar stress of **1.25*fy*** for ***fy***.

$$\ell_{dt} = \left(\frac{1.25 f_y \psi_e \psi_p \psi_o \psi_c}{75\sqrt{f_c'}}\right) d_b^{1.5}$$

psi unit

may be too conservative and shall be reviewed.

$$\ell_{dt} = \left(\frac{1.25 f_y \psi_e \psi_p \psi_o \psi_c}{32\sqrt{f_c'}}\right) d_b^{1.5}$$

kgf/cm^2 unit

我國新版規範18章(耐震特別條款)

18.5.5.2 符合第20.2.1.6節和25.4.4.1節規定之擴頭竹節鋼筋，其受拉伸展長度ℓdt應至少為式(18.5.5.1)、$8d_b$與 15 cm 之大值，且若擴頭鋼筋終止於符合第18.4.5.2節至18.4.5.4節橫向鋼筋圍束的構件自由端或符合第18.5.3節橫向鋼筋圍束之接頭核心內，則鋼筋間最小中心距允許降低至**$2.5d_b$**，柱鋼筋最小淨保護層允許降低至**$1.5\ d_b$**。

規範	ACI 318-14規範 我國新版規範(土木401-110)
第18章 SMF接頭內梁主筋擴頭伸展長度	$\ell_{dt} = \left(\frac{0.06 f_y \psi_e}{\sqrt{f'_c}}\right) d_b$

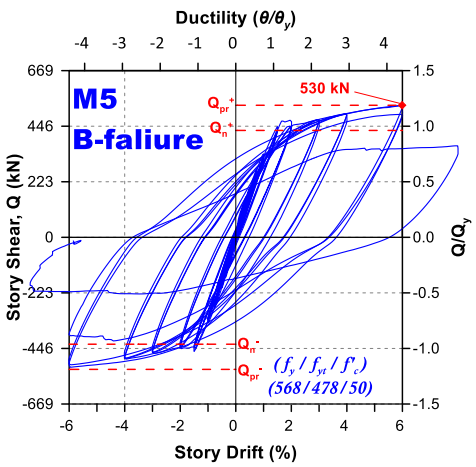
彎鉤伸展長度 $\ell_{dh} = \left(\frac{0.06 f_y \psi_e}{\sqrt{f'_c}}\right) d_b$

彎鉤(擴頭)受拉伸展長度 (kgf–cm單位)

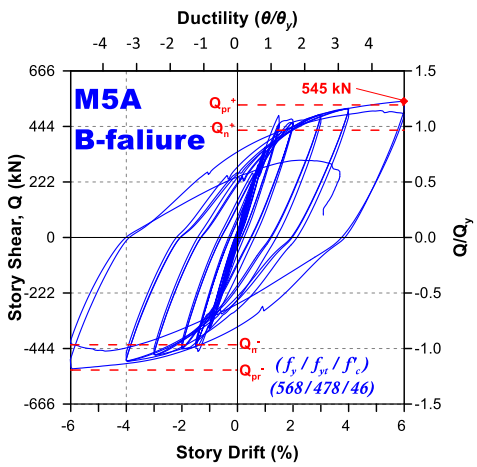
Ldt	Bar	#6	#7	#8	#9	#10	#11
fy	f'c	D19	D22	D25	D29	D32	D36
4200	280	29	33	38	43	48	54
4200	350	26	30	34	39	43	48
4200	420	23	27	31	35	40	44
4200	560	20	24	27	31	34	38
4200	700	18	21	24	27	31	34
5600	280	38	45	51	58	65	72
5600	350	34	40	46	52	58	64
5600	420	31	36	42	47	53	59
5600	560	27	32	36	41	46	51
5600	700	24	28	32	36	41	45

M5 & M5A / M6 & M6A (f_y=550 MPa)

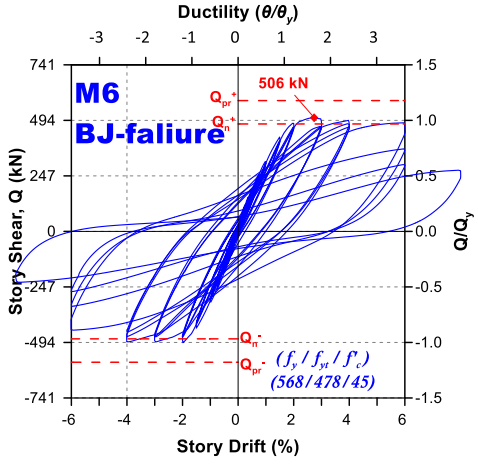
$Q_{max}/Q_y = 530/446 = 1.19$



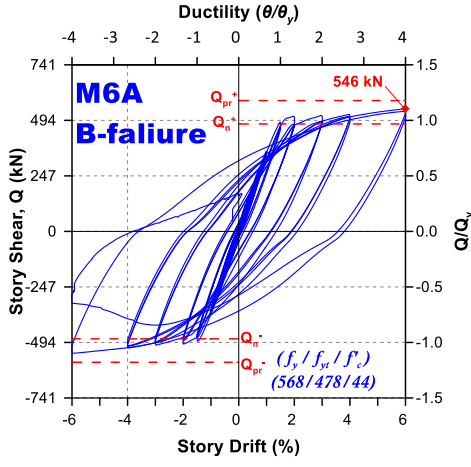
$Q_{max}/Q_y = 545/444 = 1.23$



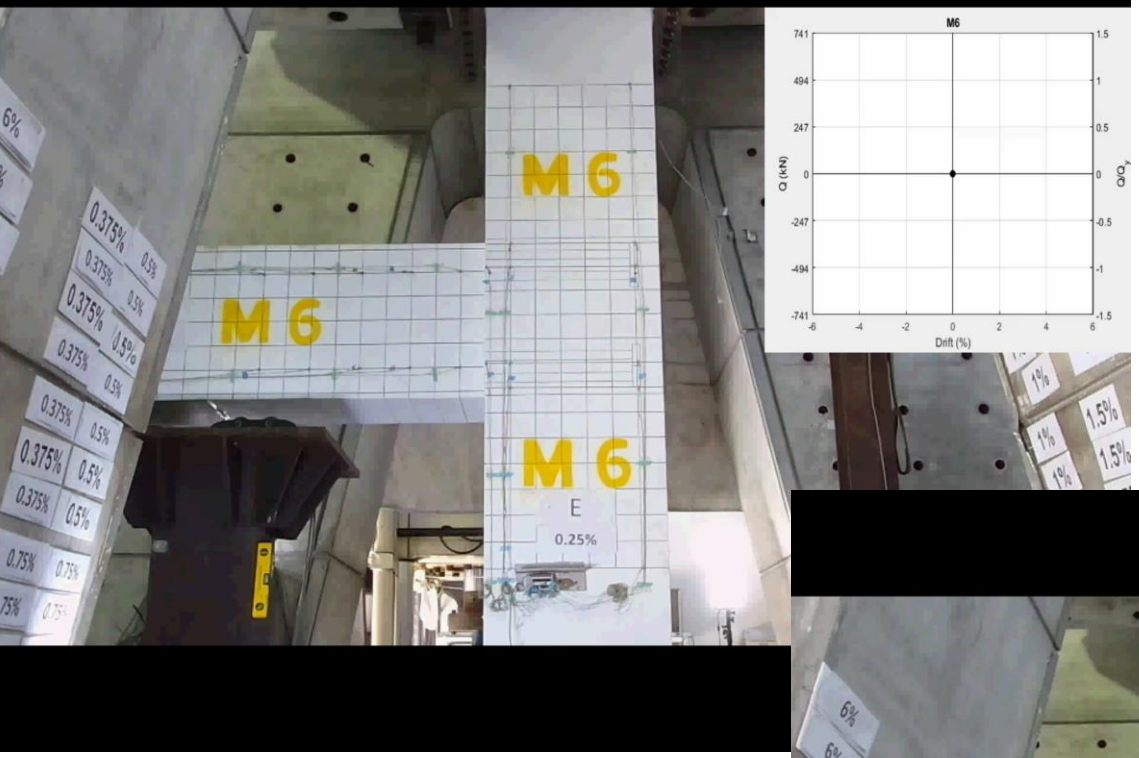
$Q_{max}/Q_y = 506/494 = 1.02$



$Q_{max}/Q_y = 546/494 = 1.11$

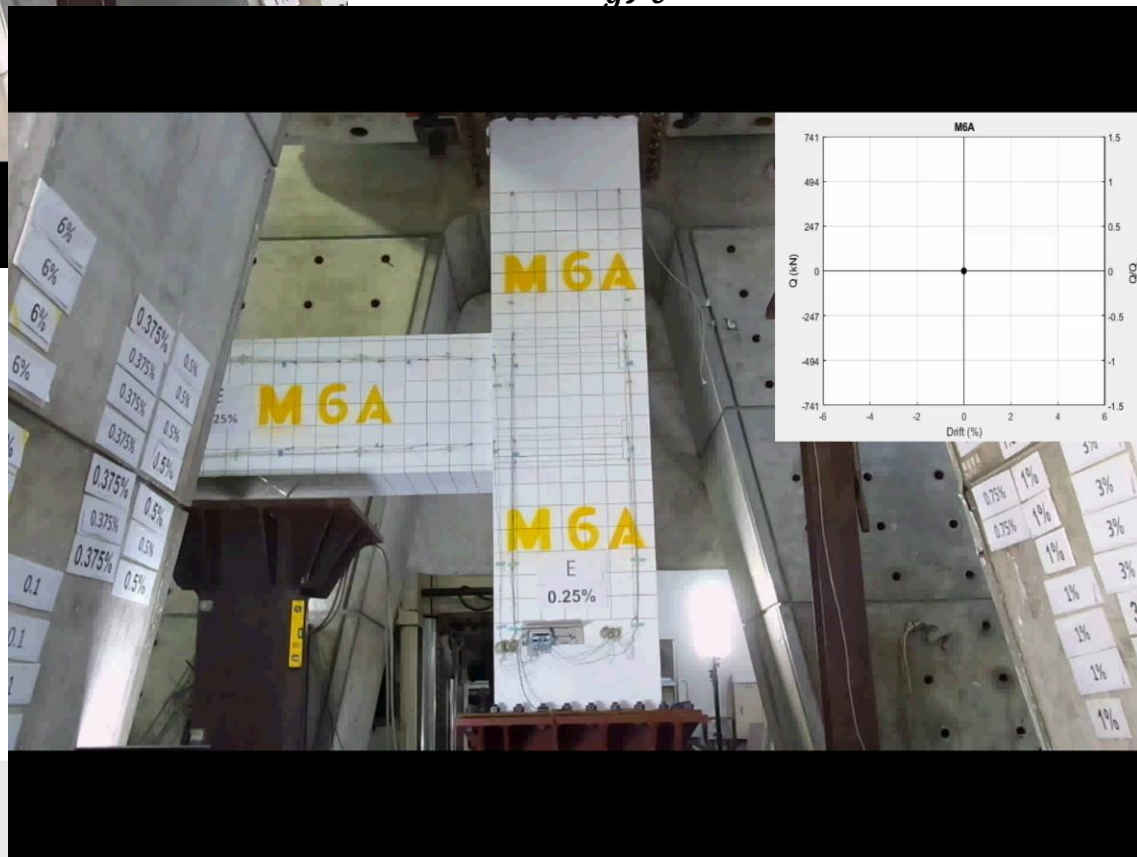


提高軸力對接頭之影響 M6 vs. M6A (f_y=550 MPa)



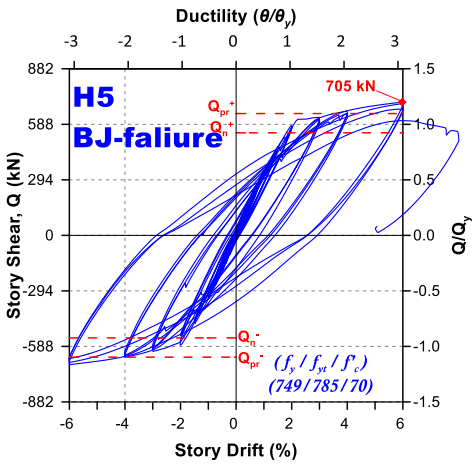
$0.10A_g f'_c$

$0.45A_g f'_c$

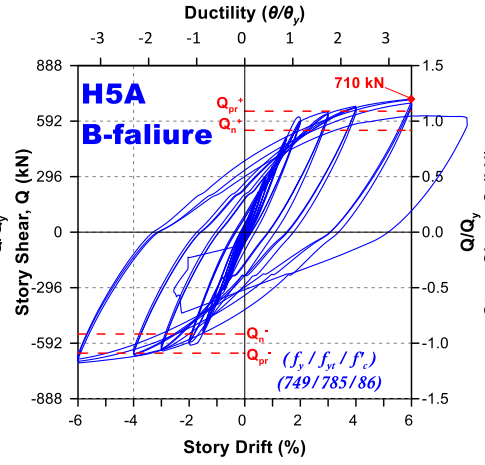


H5 & H5A / H6 & H6A (f_y=690 MPa)

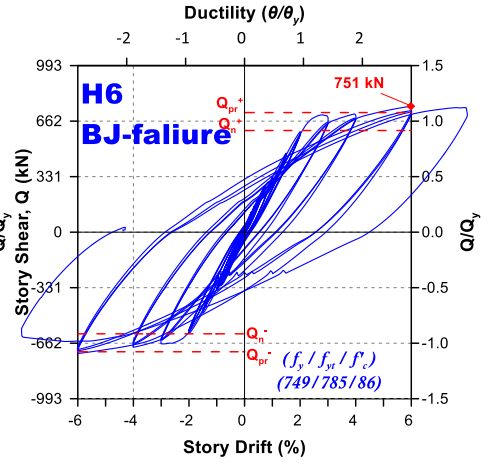
$Q_{max}/Q_y = 705/587 = 1.20$



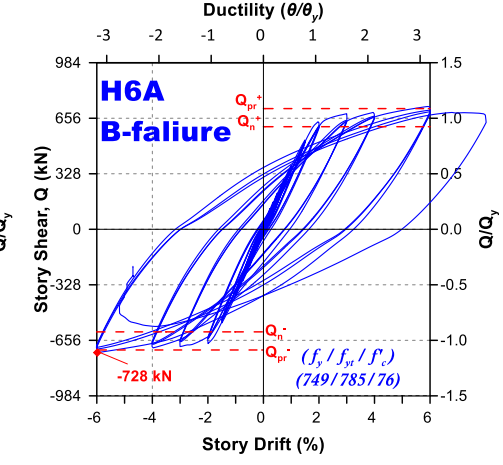
$Q_{max}/Q_y = 710/592 = 1.20$

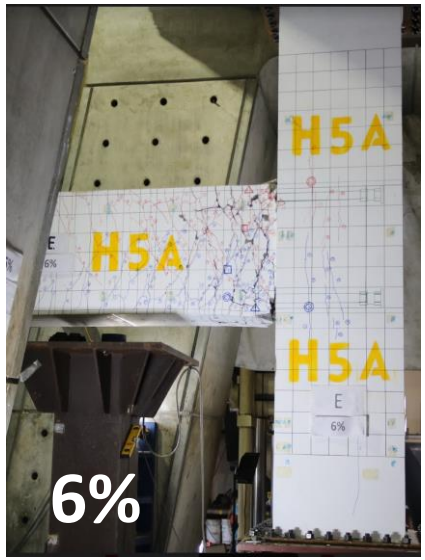


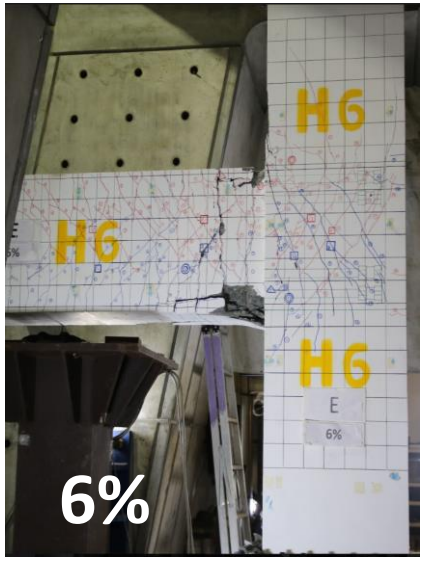
$Q_{max}/Q_y = 751/662 = 1.14$

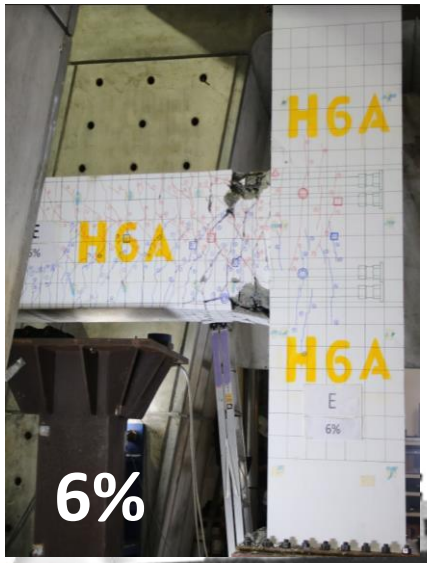


$Q_{max}/Q_y = 728/656 = 1.11$









高低軸力試體之試驗結果比較

Parameter / Group	ID	$\frac{V_u}{V_n}$	Design Failure Mode	f_c'	Q_{max} (kN)	$\frac{f_{y,m}}{f_y}$	$\frac{Q_{max}}{Q_n}$	$\frac{Q_{max}}{Q_y}$	$\frac{Q_{max}}{Q_{pr}}$	$\frac{V_{jh,m}}{b_c \times h_c \times \sqrt{f_c'}}$	Test Failure Mode
M (α_o=1.25)	M5	1.08	BJ	50	530	1.03	1.24	**1.19**	1.00	1.01	B
	M5A		BJ→B	46	545		1.27	**1.23**	1.03	1.09	B
	M6	1.31	J	45	506		1.06	**1.02**	**0.87**	1.03	**BJ**
	M6A		J→BJ	44	546		1.14	**1.11**	0.94	1.11	B
H (α_o=1.20)	H5	1.01	BJ	70	705	1.09	1.30	**1.20**	1.09	1.14	BJ
	H5A		BJ→B	86	710		1.31	**1.20**	1.10	1.04	B
	H6	1.23	J	86	751		1.24	**1.14**	1.05	1.10	BJ
	H6A		J→BJ	76	728		1.20	**1.11**	1.02	1.13	B

塑鉸區橫向鋼筋用量(18.4.5.4)

表18.4.5.4

橫向鋼筋	條件	適用表達式		
直線型閉合箍筋之 A_{sh}/sb_c	$P_u \le 0.3A_g f_c'$ 與 $f_c' \le 700\ \text{kgf/cm}^2$	(a)與(b)之較大值	$0.3\left(\frac{A_g}{A_{ch}}-1\right)\frac{f_c'}{f_{yt}}$	(a)
			$0.09\frac{f_c'}{f_{yt}}$	(b)
	$P_u > 0.3A_g f_c'$ 或 $f_c' > 700\ \text{kgf/cm}^2$	(a)、(b)與(c)之最大值	$0.2k_f k_n \frac{P_u}{f_{yt}A_{ch}}$	(c)
螺箍筋或圓形閉合箍筋 ρ_s	$P_u \le 0.3A_g f_c'$ 與 $f_c' \le 700\ \text{kgf/cm}^2$	(d)與(e)之較大值	$0.45\left(\frac{A_g}{A_{ch}}-1\right)\frac{f_c'}{f_{yt}}$	(d)
			$0.12\frac{f_c'}{f_{yt}}$	(e)
	$P_u > 0.3A_g f_c'$ 或 $f_c' > 700\ \text{kgf/cm}^2$	(d)、(e)與(f)之最大值	$0.35k_f \frac{P_u}{f_{yt}A_{ch}}$	(f)

(c)(f)不適用接頭

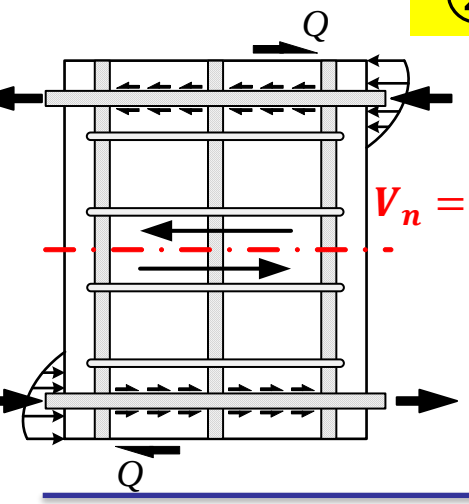
實驗證明
OK

簡報大綱

- 我國規範修訂之背景說明
- 接頭最小深度 (土木401-112　18.5.2)
- 接頭橫向鋼筋 (土木401-112　18.5.3)
- 接頭剪力強度 (土木401-112　18.5.4)
- 受拉錨定長度 (土木401-112　18.5.5)
- 結語

Concluding remarks

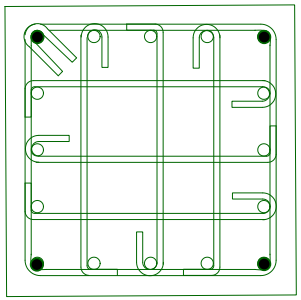
②Shear strength



$$V_u = T + C - Q$$

$$V_n = \left(\mathbf{k} + \frac{P_u}{A_g f'_c}\right)\sqrt{\frac{h_c}{h_b}}\sqrt{f'_c}A_j$$

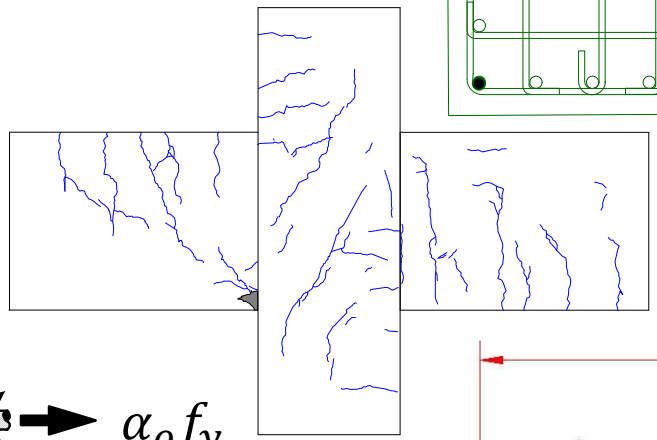
① Confinement



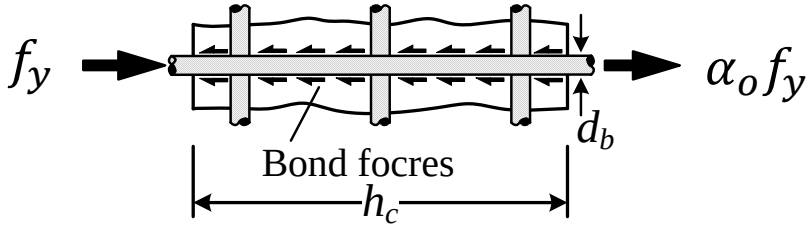
$$\frac{A_{sh}}{sb_c} \geq 0.3\frac{f'_c}{f_{yt}}\left(\frac{A_g}{A_{ch}} - 1\right)$$

$$\frac{A_{sh}}{sb_c} \geq 0.09\frac{f'_c}{f_{yt}}$$

$f_{yt} \leq 8000$ kgf/cm²



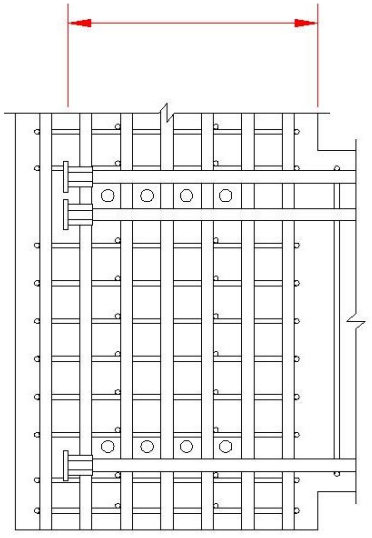
③ Min. joint depth



$$h_c = \frac{1}{40}\frac{f_y}{\sqrt{f'_c}}d_b \geq 24d_b \quad \text{(psi unit)}$$

$$h_c = \frac{1}{10.6}\frac{f_y}{\sqrt{f'_c}}d_b \geq 24d_b \quad \text{(kgf/cm}^2\text{ unit)}$$

④ Anchorage



$$\ell_{dt} \geq \frac{0.06 f_y d_b}{\sqrt{f'_c}} \text{ (kgf/cm}^2\text{)}$$

$$\geq 8d_b$$

$$\geq 150\ mm$$

$$\geq 0.75h_c$$

最小間距 2.5 d_b

$\sqrt{f'_c} \leq \sqrt{1000}$ kgf/cm²

避免3%層間變位角發生嚴重滑移

Thanks for your attention

- Questions?

2022 @ NCREE

簡報結束
敬請指教

NCREE

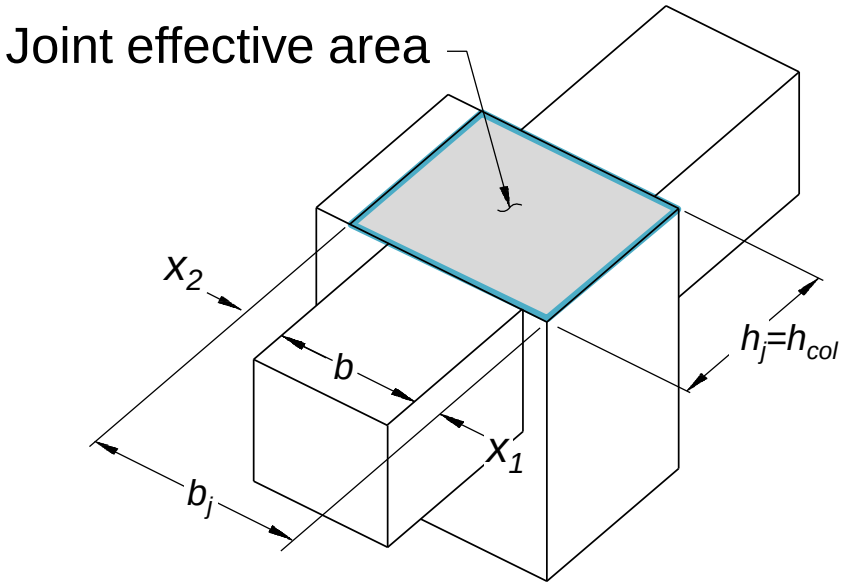
RUENTEX

NSTC 國家科學及技術委員會
National Science and Technology Council

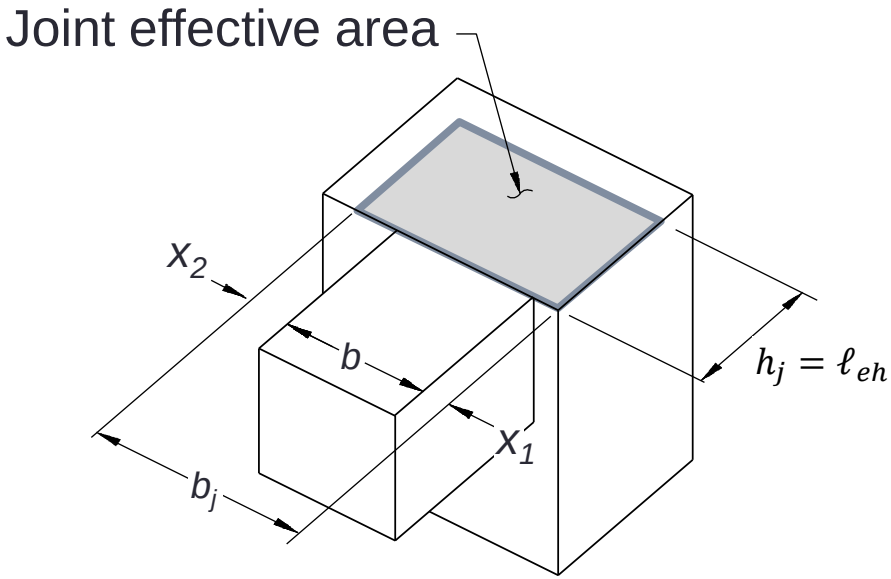
Effective Joint Area

$$b_j = b + x_1 + x_2 \leq b_{col} \qquad A_j = b_j \cdot h_j$$

其中x_1及x_2分別為梁兩邊至柱邊之距離(圖6.3)，代入上式計算時，x_1或x_2值不得超過$h_{col}/4$。[黃世建等人(2014)]



(a)梁主筋貫穿梁柱接頭, $h_j = h_{col}$

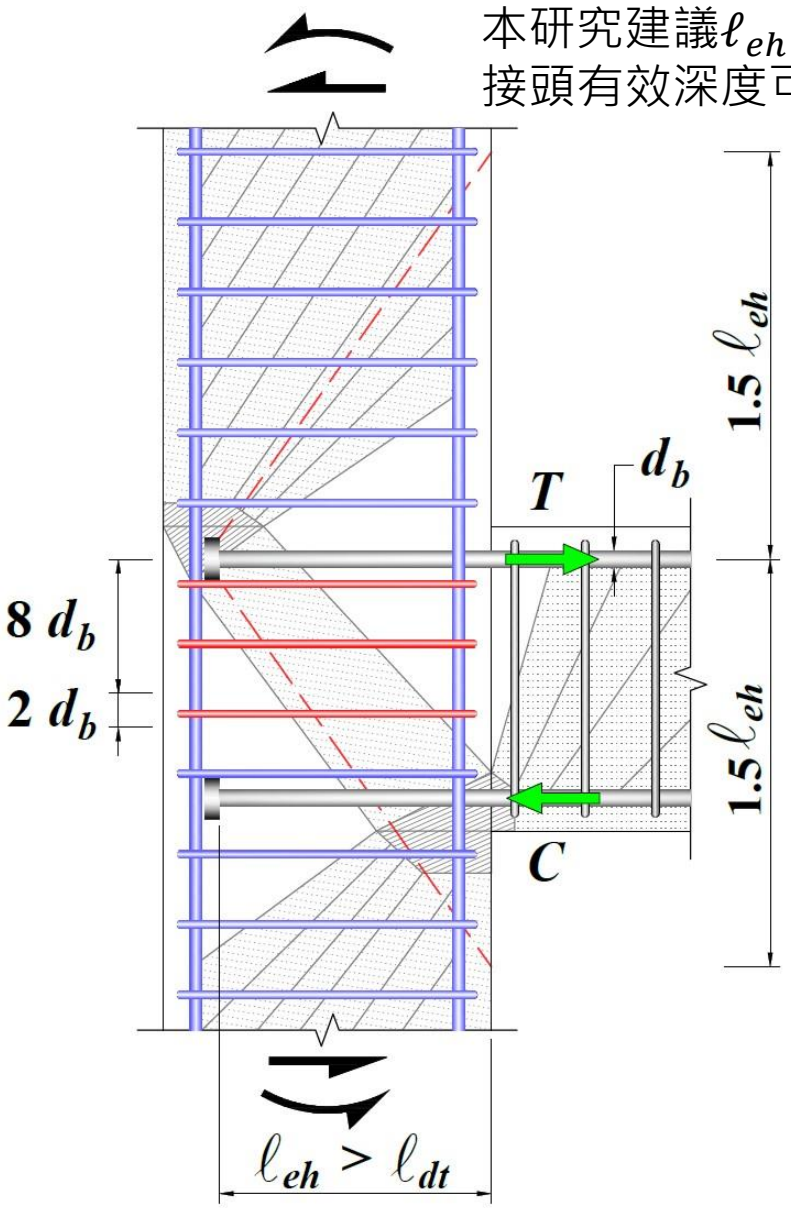


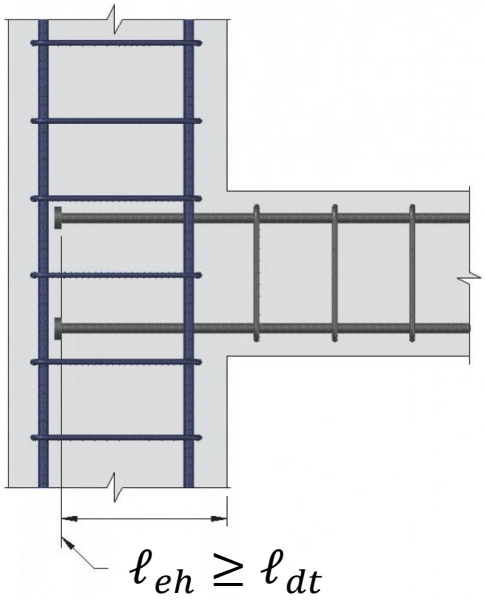
(b)梁主筋錨定於接頭內, $h_j = \ell_{eh}$

除非梁主筋之錨定長度皆超過3/4柱深度以上，接頭有效深度h_j得為柱深度h_{col}

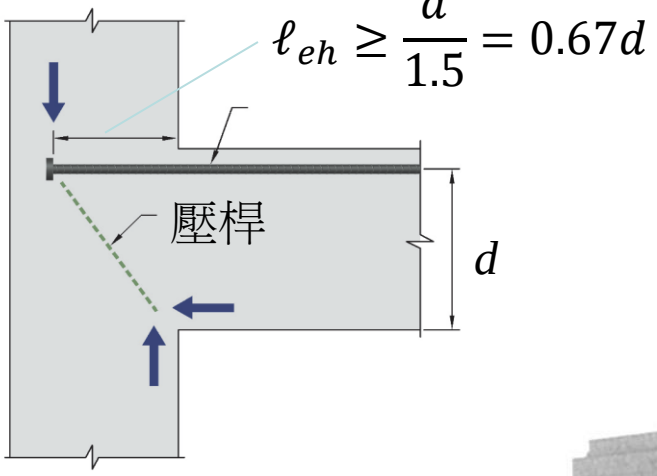
擴頭錨定與接頭有效深度

本研究建議ℓ_{eh}至少3/4 柱深度
接頭有效深度可以取柱全深





土木401-112
圖R25.4.4.2b 擴頭竹節鋼筋延伸至柱心之遠端面，其錨定長度超過 ℓ_{dt}



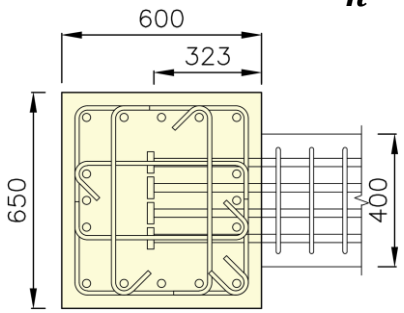
圖R25.4.4.2c 藉由保持**等於或大於*d*/1.5** 之錨定長度，以避免接頭拉破破壞

Authors	*Joint*	$f'_{c,m}$ *(MPa)*	$f_{y,m}$ *(MPa)*	d_b *(mm)*	ℓ_{eh} *(mm)*	*Failure Mode*
Yoshida et al. (2000)	No. 1	38	562	19.1	263	J
	No. 2	38	562	19.1	263	J
	No. 3	31	562	19.1	263	J
Takeuchi et al. (2001)	O-2	61	586	25.4	267	BJ
Kiyohara et al. (2004)	No. 1	95	707	28.7	365	BJ
	No. 2	148	707	28.7	365	J
	No. 3	44	707	28.7	365	BJ
	No. 4	95	707	28.7	460	BJ
	No. 5	95	707	28.7	275	BJa
Lin et al. (2010)	T7	47	448	25.4	528	B
	T9	49	448	25.4	528	B
	T8	50	448	25.4	309、216	BJa
Lin et al. (2012)	TT1	50	465	25.4	309、216	BJa
	TT2	54	465	25.4	450、380	B
	TT3	50	465	25.4	309、309	B
Lin et al. (2014)	A318S1	36	456	25.4	358	BJ
	A318S2	45	456	25.4	358	BJ
	A318S3	51	456	25.4	358	BJ
	A352S1	43	456	25.4	323	BJa
	A352S2	39	456	25.4	323	BJa
	AAIJS1	39	456	25.4	440	BJ
Lin et al. (2016)	LAMV	81	709	25.4	400	B
	HAMV	84	709	25.4	400	B
	LAHV	94	709	25.4	400	BJ
	HAHV	90	709	25.4	400	B
Lin et al. (2020)	JE1	50	494(#8)、483(#10)	25.4、32.2	500、400	B
	JE2	48	595	25.4	500、400	B
	JE3	43	595	25.4	400	B
Lee et al. (2023)	N4	39	462	35.8	580	B
	N5	34	462	35.8	580	BJ
	N6	33	462	35.8	580	BJ
	M4	43	568	35.8	580	B
	M5	50	568	35.8	580	B
	M5A	46	568	35.8	580	B
	M6	44	568	35.8	580	BJ
	M6A	44	568	35.8	580	B
	H4	79	749	35.8	580	B
	H5	70	749	35.8	580	BJ
	H5A	86	749	35.8	580	B
	H6	86	749	35.8	580	BJ
	H6A	76	749	35.8	580	B
Ou et al. (2023)	NEBC	47	745	35.8	937	B

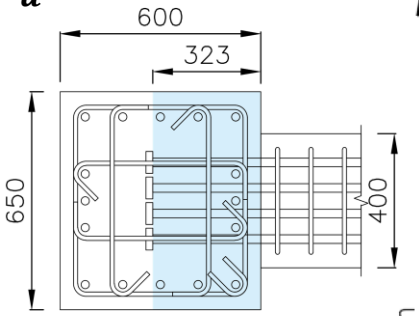
Coordinate of Exterior Beam-Column Joints

Shear strength

$$\Phi V_n > V_u$$



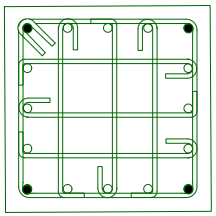
$$V_n = b_j \times h_c \times \gamma\sqrt{f_c'}$$



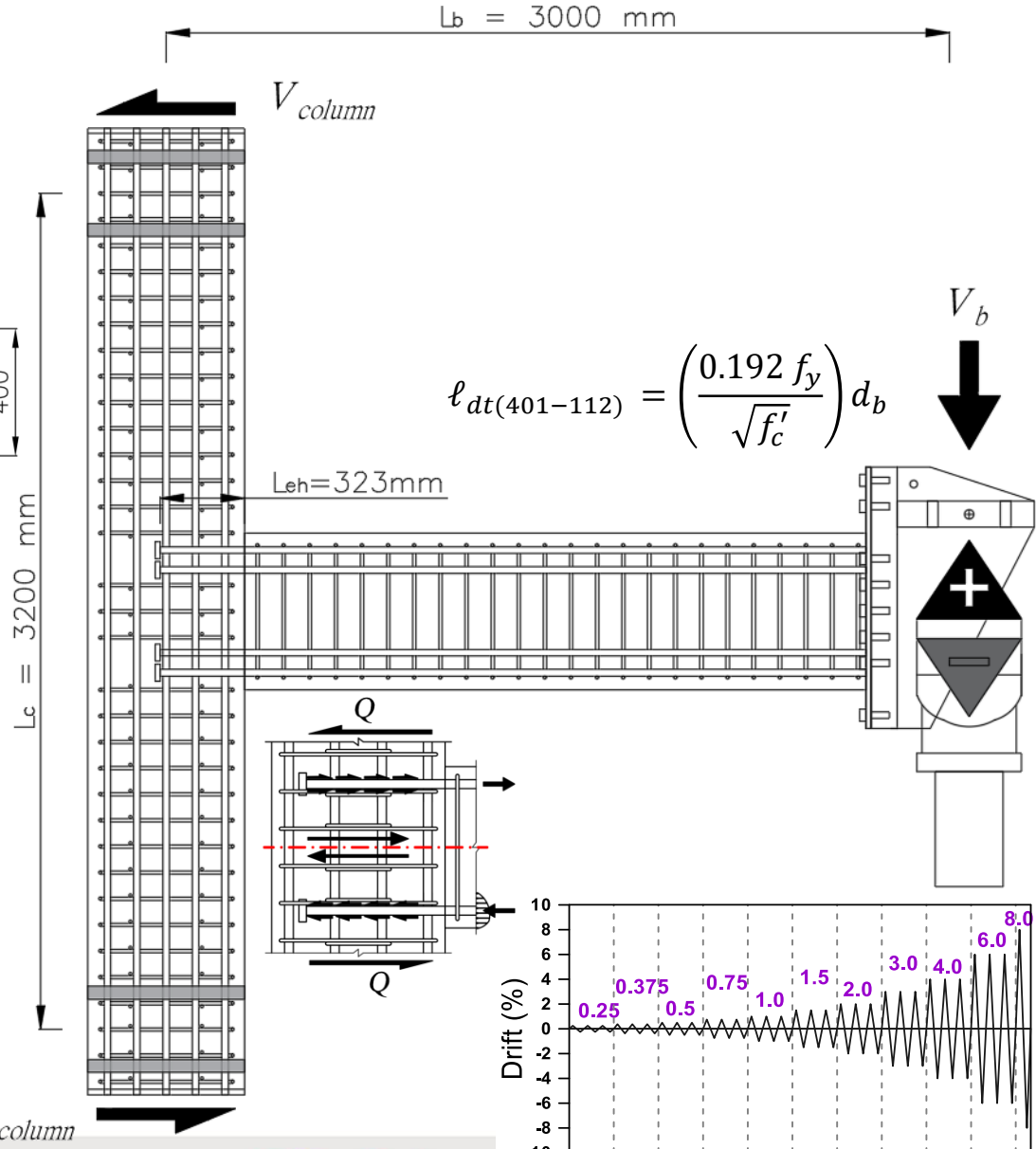
$$V_n = b_j \times \ell_{eh} \times \gamma\sqrt{f_c'}$$

Confinement

$$A_{sh,ratio} = \frac{A_{sh,pro}}{A_{sh,req}}$$

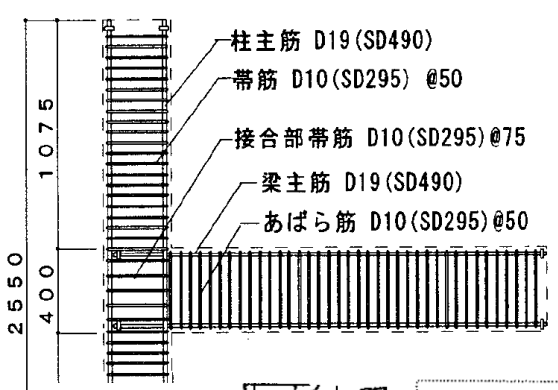


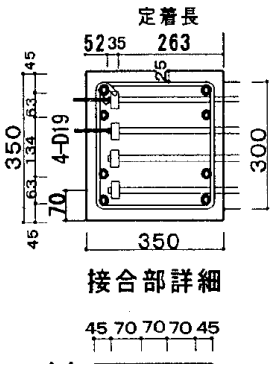
$$A_{sh,req} = max \begin{cases} 0.3\left(\frac{A_g}{A_{ch}} - 1\right)\frac{f_c'}{f_{yt}} \times sb_c \\ 0.09\frac{f_c'}{f_{yt}} \times sb_c \end{cases}$$

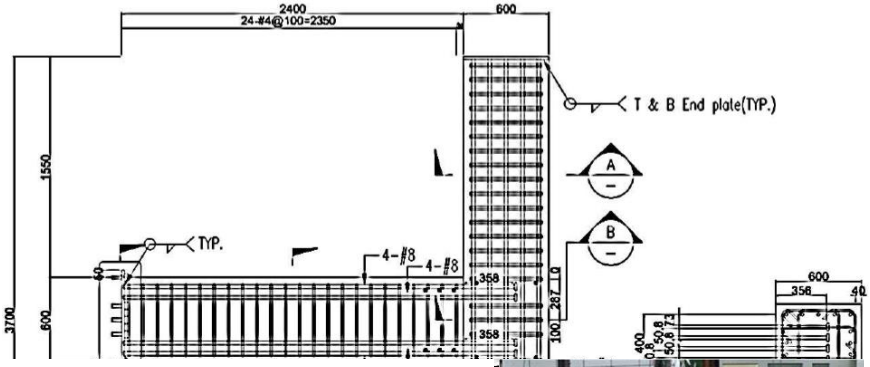


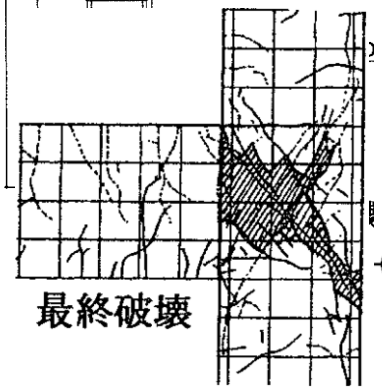
$$\ell_{dt(401-112)} = \left(\frac{0.192\, f_y}{\sqrt{f_c'}}\right) d_b$$

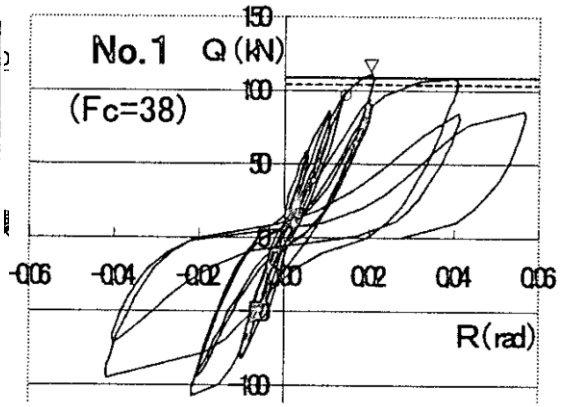
梁柱接頭接頭剪力與圍束試算

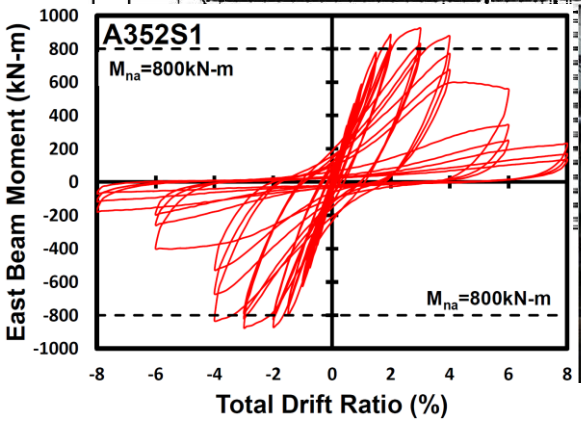










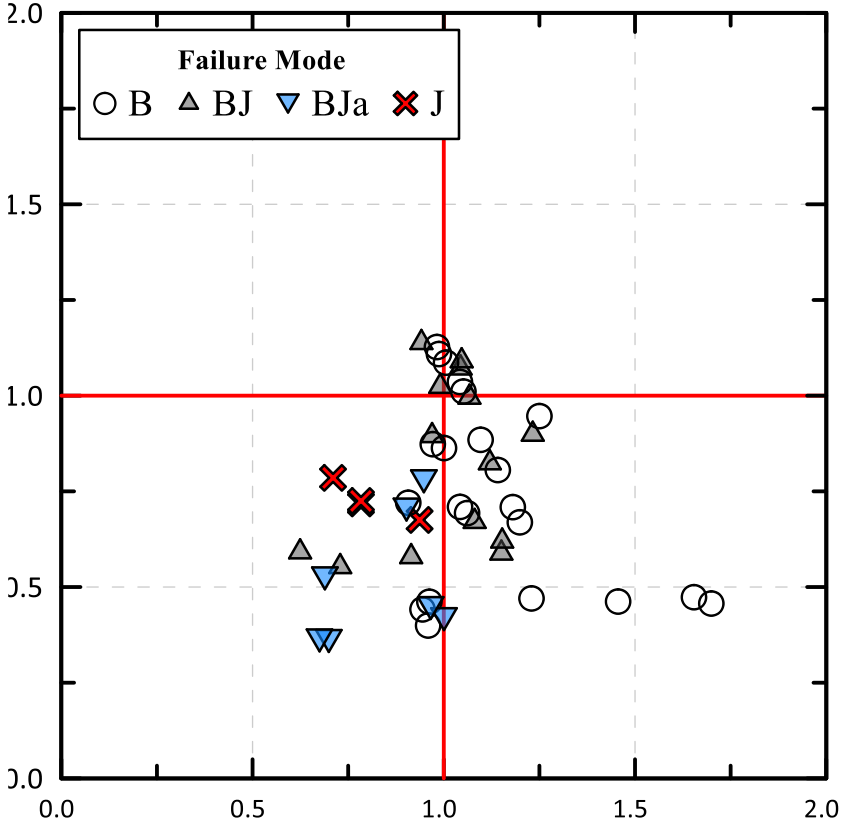


Authors	*Joint*	$f'_{c,m}$ *(MPa)*	$f_{y,m}$ *(MPa)*	$f_{yt,m}$ *(MPa)*	ℓ_{eh} *(mm)*	$\ell_{dt,m(401-112)}$ *(mm)*	$\frac{\ell_{eh}}{\ell_{dt,m(401-112)}}$	$\frac{A_{sh,pro}}{A_{sh,req}}$	*Failure Mode*
Yoshida et al. (2000)	No. 1	38	562	855	263	336	0.78	0.56	J
Lin et al. (2014)	A352S1	43	456	490	323	341	0.95	1.19	BJa

全試體47點之測試參數與破壞模式關係 (I)

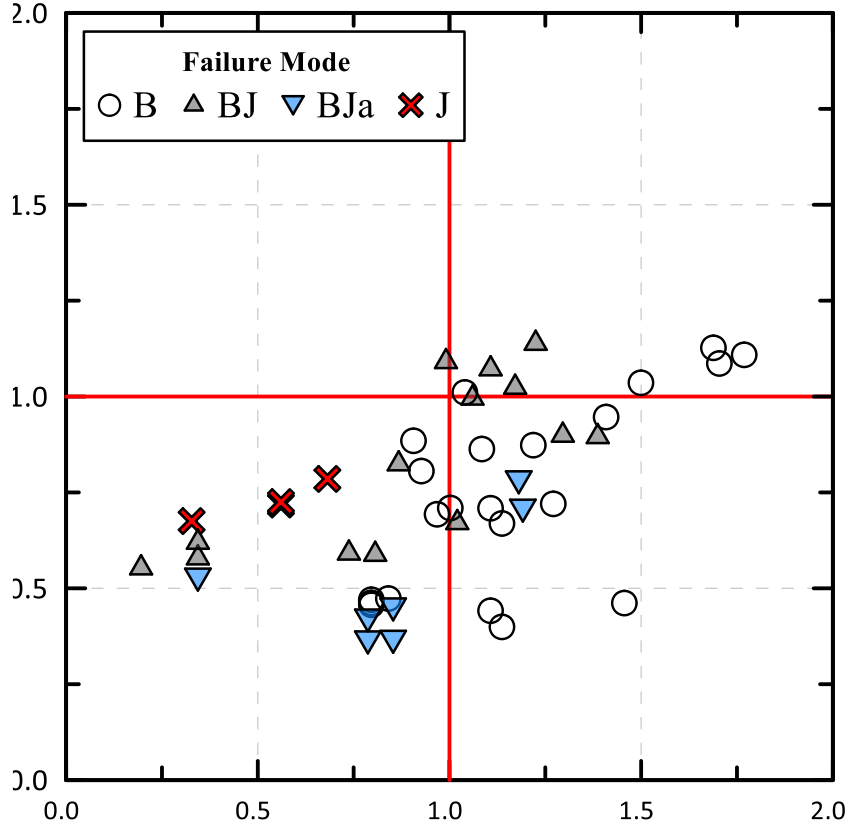
$$\frac{V_{jh,test}}{b_j h_c \sqrt{f_c'}}$$

(SI units)



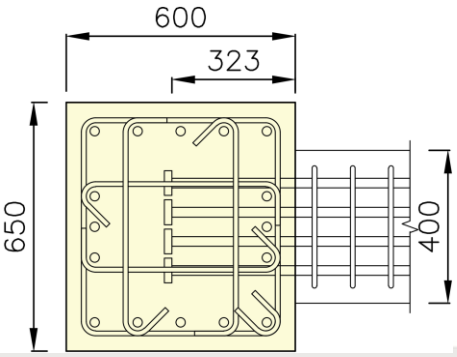
$\ell_{eh} / \ell_{dt,m\ (401\text{-}112)}$

符合規範

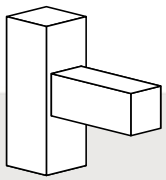


$A_{sh,ratio}$

符合規範

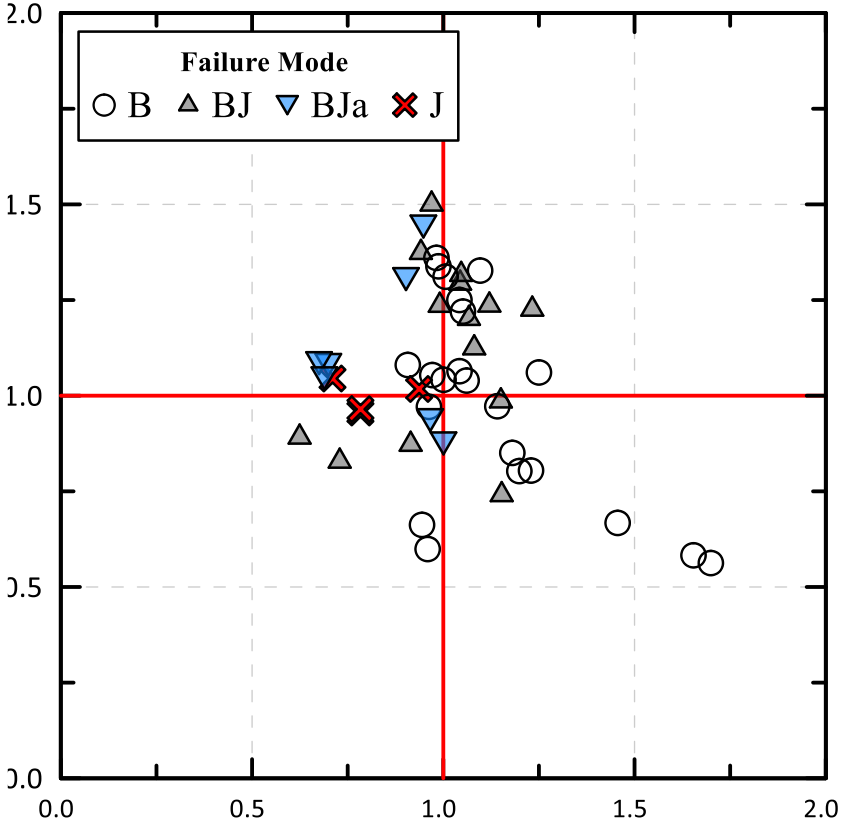


$A_j = b_j\ h_c$



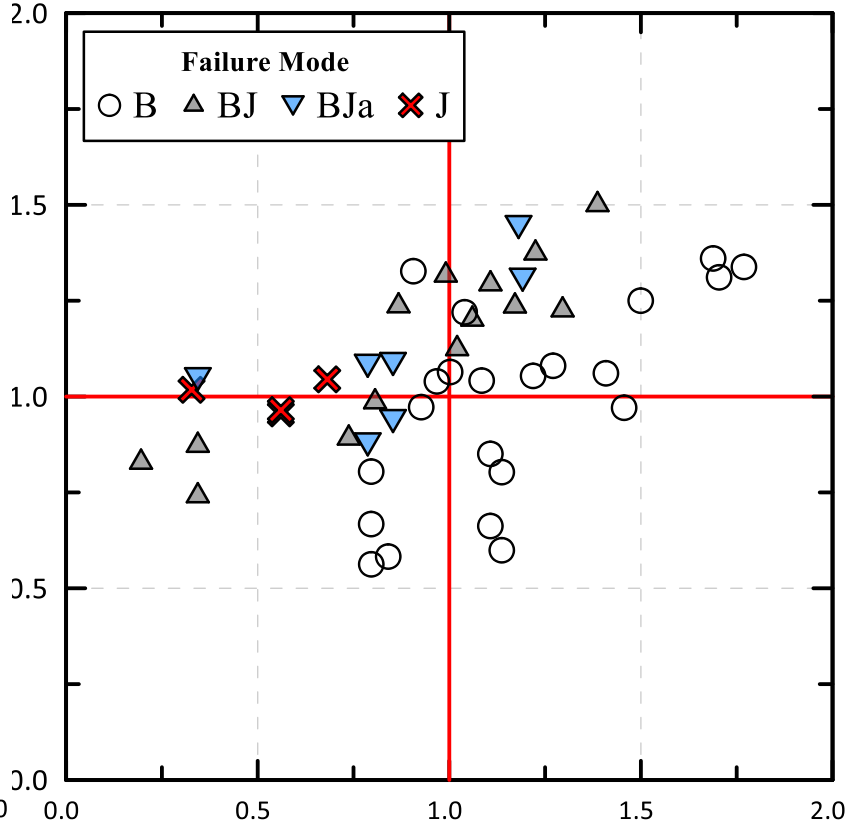
$$\frac{A_{sh}}{sb_c} \geq max \begin{cases} 0.3\left(\frac{A_g}{A_{ch}} - 1\right)\frac{f_c'}{f_{yt}} \\ 0.09\frac{f_c'}{f_{yt}} \end{cases}$$

全試體47點之測試參數與破壞模式關係 (II)



$\frac{V_{jh,test}}{b_j \ell_{eh} \sqrt{f_c'}}$

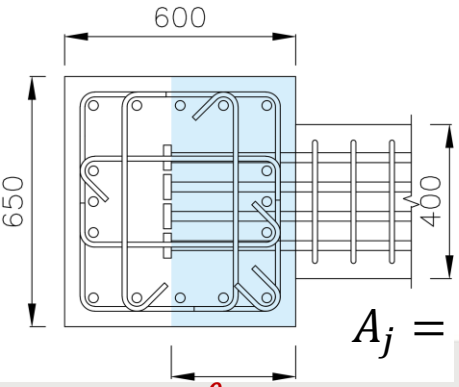
$\ell_{eh} / \ell_{dt,m\ (401\text{-}112)}$



$A_{sh,ratio}$

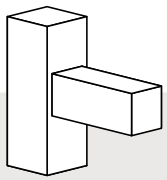
符合規範

符合規範



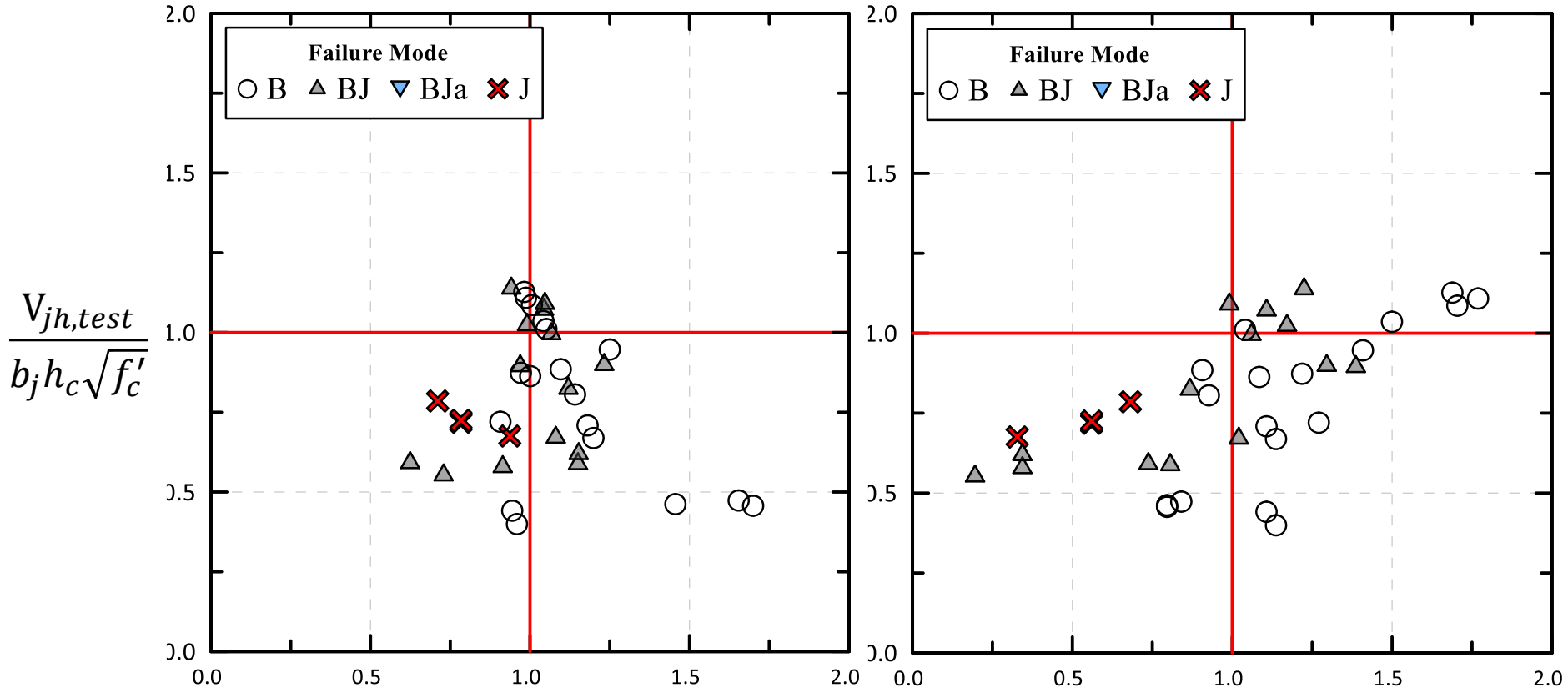
ℓ_{eh}

$A_j = b_j \ell_{eh}$



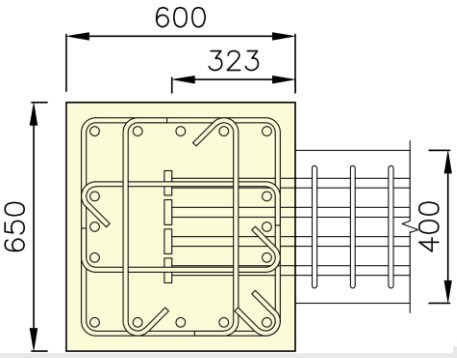
$$\frac{A_{sh}}{sb_c} \geq max \begin{cases} 0.3\left(\frac{A_g}{A_{ch}} - 1\right)\frac{f_c'}{f_{yt}} \\ 0.09\frac{f_c'}{f_{yt}} \end{cases}$$

$\ell_{eh} > d/1.5$ 之試體測試參數與破壞模式分布圖



符合規範

符合規範

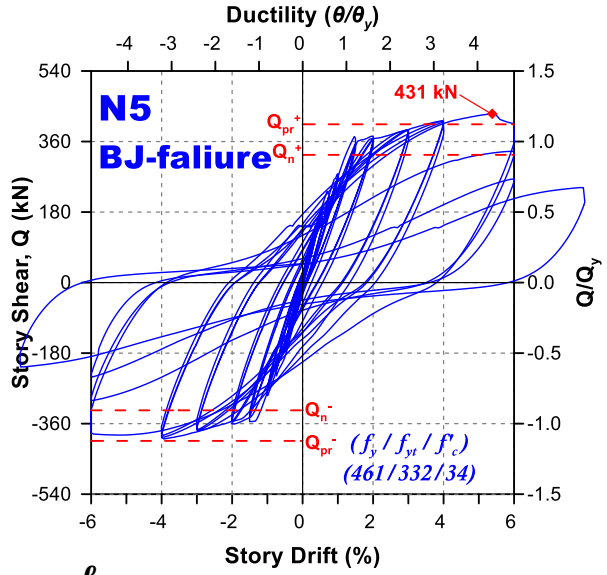


$A_j = b_j\ h_c$

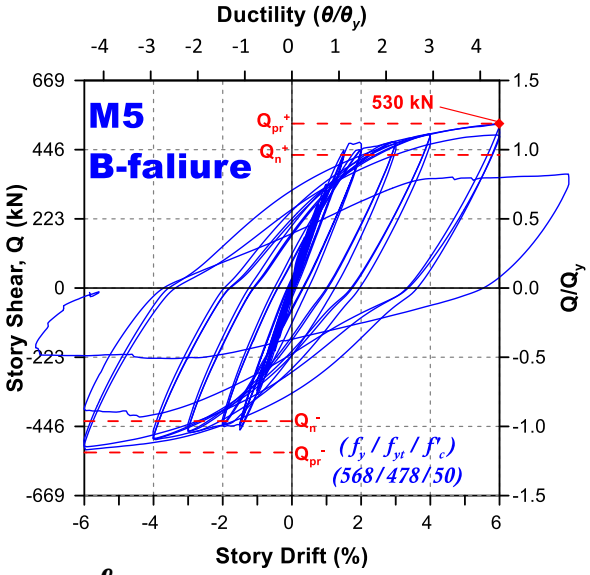
建議ℓ_{eh}至少3/4 柱深度(則$h_j = h_c$)
若短於3/4 柱深度，亦不得少於d/1.5
並修正接頭有效抗剪深度 $h_j = \ell_{eh}$

$$\frac{A_{sh}}{sb_c} \geq max \begin{cases} 0.3\left(\frac{A_g}{A_{ch}} - 1\right)\frac{f_c'}{f_{yt}} \\ 0.09\frac{f_c'}{f_{yt}} \end{cases}$$

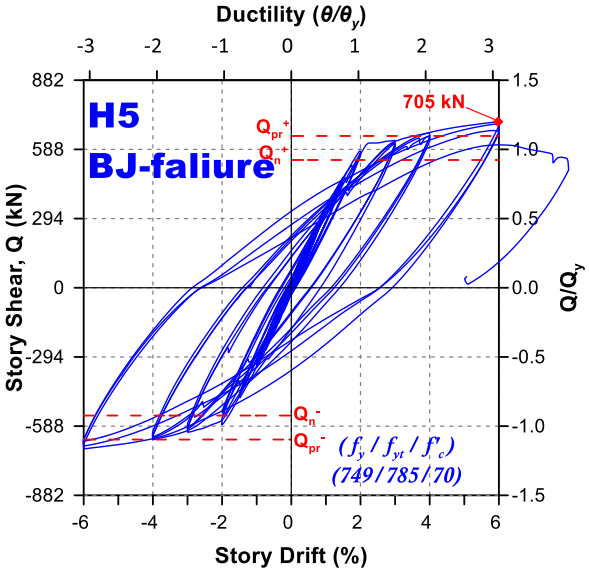
擴頭鋼筋梁柱接頭試驗 N5、M5、H5



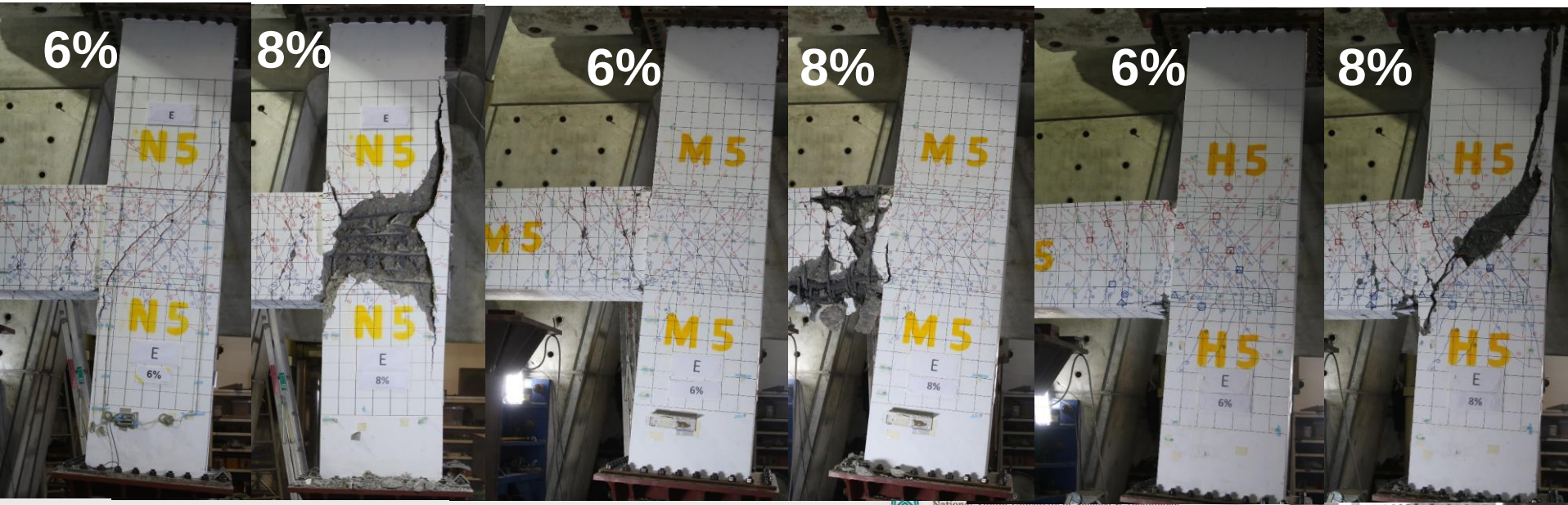
$$\frac{\ell_{eh}}{\ell_{dt,m(401-112)}} = 1.07\,,A_{sh}\ ratio = 1.06$$



$$\frac{\ell_{eh}}{\ell_{dt,m(401-112)}} = 1.05\,,A_{sh}\ ratio = 1.04$$



$$\frac{\ell_{eh}}{\ell_{dt,m(401-112)}} = 0.94\,,A_{sh}\ ratio = 1.22$$



我國新版規範18章(耐震特別條款)

18.5.5.2 符合第20.2.1.6節和25.4.4.1節規定之擴頭竹節鋼筋，其受拉伸展長度ℓdt應至少為式(18.5.5.1)、$8d_b$與 15 cm 之大值，且若擴頭鋼筋終止於符合第18.4.5.2節至18.4.5.4節橫向鋼筋圍束的構件自由端或符合第18.5.3節橫向鋼筋圍束之接頭核心內，則鋼筋間最小中心距允許降低至**$2.5d_b$**，柱鋼筋最小淨保護層允許降低至**$1.5\ d_b$**。

規範	ACI 318-14規範 我國新版規範(土木401-110)
第18章 SMF接頭內梁主筋擴頭伸展長度	$\ell_{dt} = \left(\frac{0.06 f_y \psi_e}{\sqrt{f'_c}}\right) d_b$

彎鉤伸展長度 $\ell_{dh} = \left(\frac{0.06 f_y \psi_e}{\sqrt{f'_c}}\right) d_b$

彎鉤(擴頭)受拉伸展長度 (kgf–cm單位)

Ldt	Bar	#6	#7	#8	#9	#10	#11
fy	f'c	D19	D22	D25	D29	D32	D36
4200	280	29	33	38	43	48	54
4200	350	26	30	34	39	43	48
4200	420	23	27	31	35	40	44
4200	560	20	24	27	31	34	38
4200	700	18	21	24	27	31	34
5600	280	38	45	51	58	65	72
5600	350	34	40	46	52	58	64
5600	420	31	36	42	47	53	59
5600	560	27	32	36	41	46	51
5600	700	24	28	32	36	41	45