高強度鋼筋混凝土柱之耐震圍束行為

沈文成

May 27, 2015

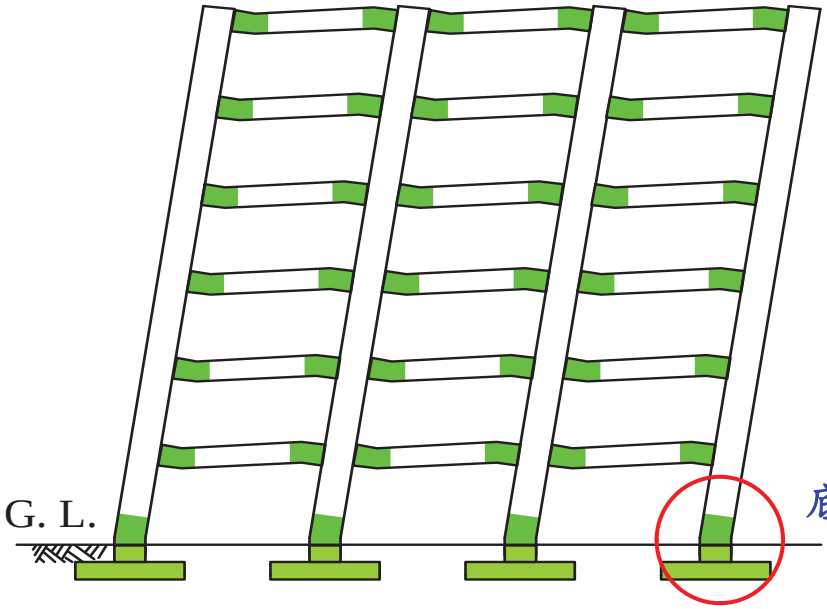
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簡報大綱

- 前言
- 設計公式
- 測試結果
- 結論

前言

超高樓層建築之底層柱



- 承受極大之軸力
- 變形能力降低

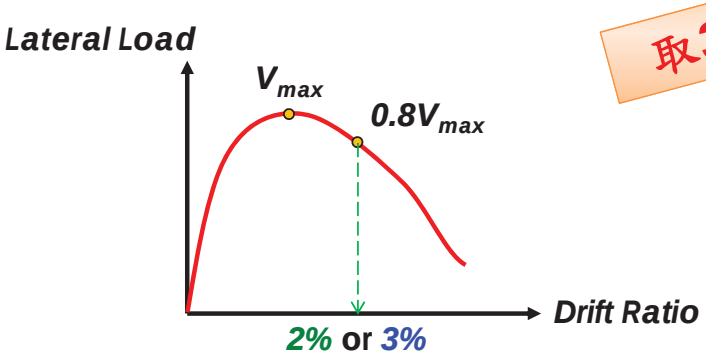
重要

➡ 耐震圍束效果？

變形能力之合格標準

	$0.8V_{max}$時之 層間位移比
設計地震 (回歸期為 475 年)	2%
最大考量地震 (回歸期為 2500 年)	3%

取3%為合格標準



設計公式

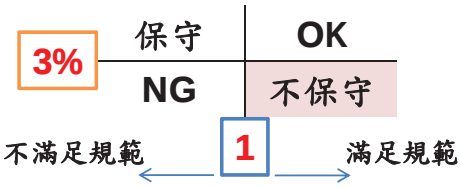
橫箍柱設計公式

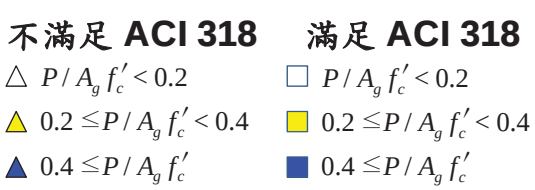
- 美國 **ACI 318-14**

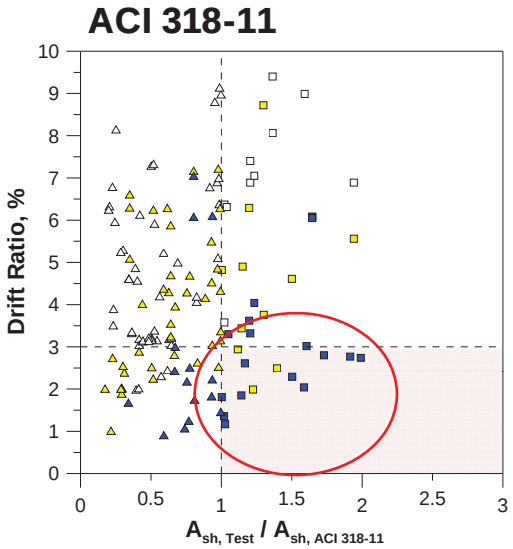
$$\frac{A_{sh}}{sb_c} \geq \max \begin{cases} 0.09\dfrac{f_c'}{f_{yt}} \\ 0.3\dfrac{f_c'}{f_{yt}}\left(\dfrac{A_g}{A_{ch}}-1\right) \\ 0.2k_f k_n k_p \dfrac{f_c'}{f_{yt}}\dfrac{A_g}{A_{ch}} \end{cases} \quad \begin{array}{l} \text{當} \quad P > 0.3A_g f_c' \\ \text{或} \quad f_c' > 70MPa \end{array}$$

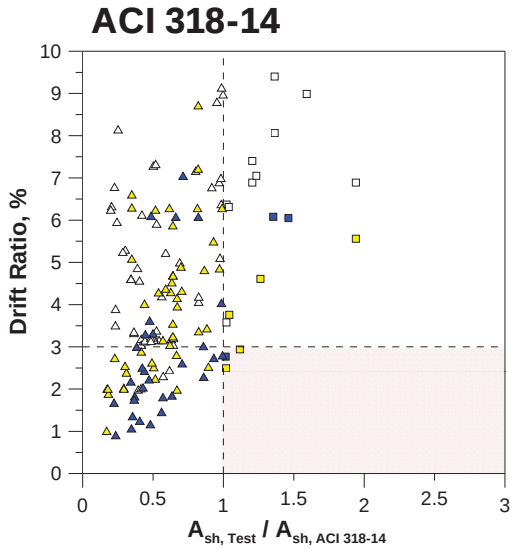
其中 $f_{yt} \leq 700MPa$

以 ACI 318-14 檢核 PEER 資料庫









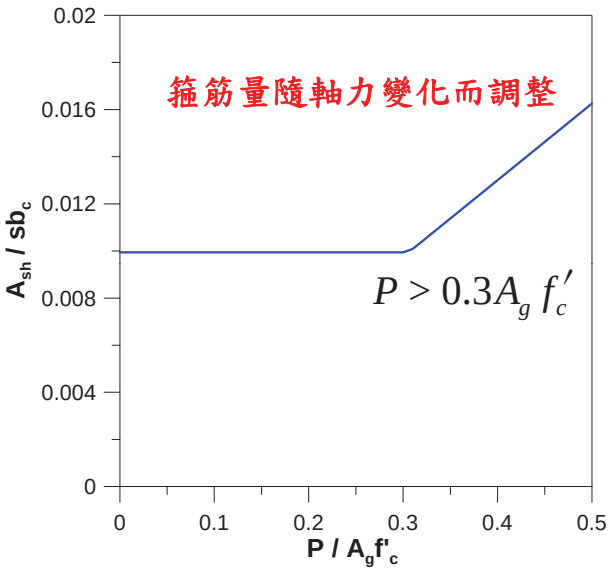
軸力參數 k_p

$$\frac{A_{sh}}{sb_c} = 0.2k_f k_n k_p \frac{f'_c}{f_{yt}} \frac{A_g}{A_{ch}}$$

$$k_p = \frac{P}{A_g f'_c}$$

承受軸力 ↑

圍束箍筋量 ↑



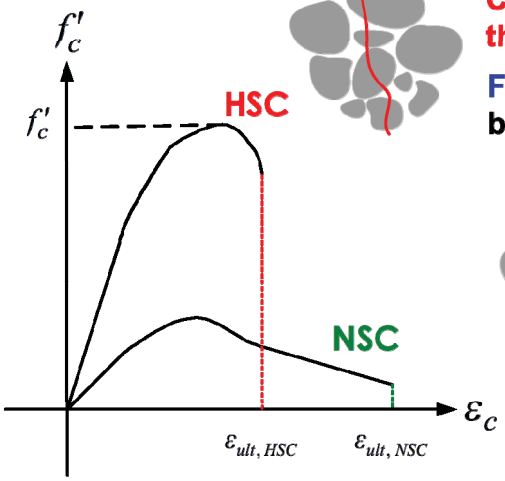
混凝土強度參數 k_f

$$\frac{A_{sh}}{sb_c} = 0.2k_f k_n k_p \frac{f'_c}{f_{yt}} \frac{A_g}{A_{ch}}$$

$$k_f = \frac{f'_c}{175} + 0.6 \geq 1.0$$

混凝土強度 ↑

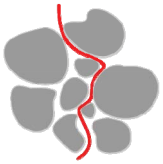
圍束箍筋量 ↑



Cracks propagate cutting through aggregates

Fewer crack paths

brittle



Mortar paste cracking

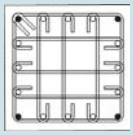
Abundant crack paths

less brittle

彎鉤有效性參數 k_n

$$\frac{A_{sh}}{sb_c} = 0.2k_f k_n k_p \frac{f'_c}{f_{yt}} \frac{A_g}{A_{ch}}$$

$$k_n = \frac{n_l}{n_l - 2}$$

n_l	k_n
 16	1.14

n_l 為受耐震彎鉤圍束之主筋數量

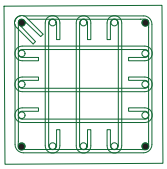
主筋必須使用耐震彎鉤加以圍束

測試結果

實驗規劃

T70-1

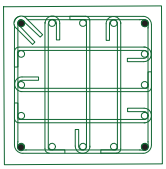
180˚-180˚彎鉤

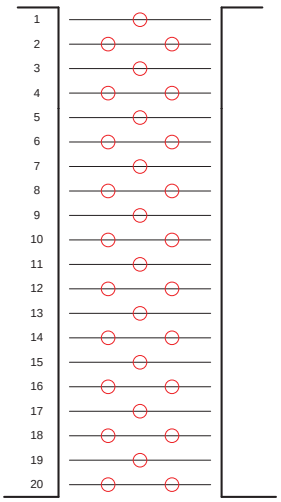


T70-2

90˚-180˚彎鉤

交錯配置



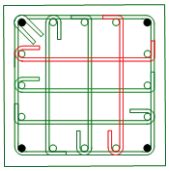


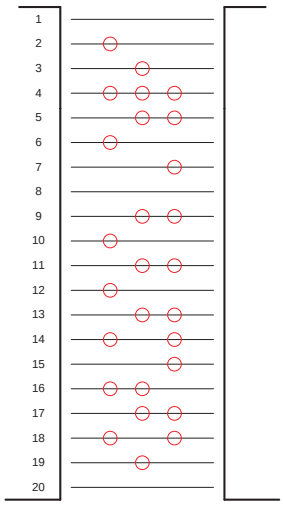
○ 90˚彎鉤端

T70-4

90˚-180˚彎鉤

非交錯配置





○ 90˚彎鉤端

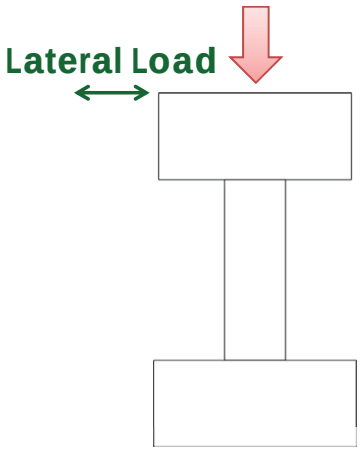
側力加載歷時

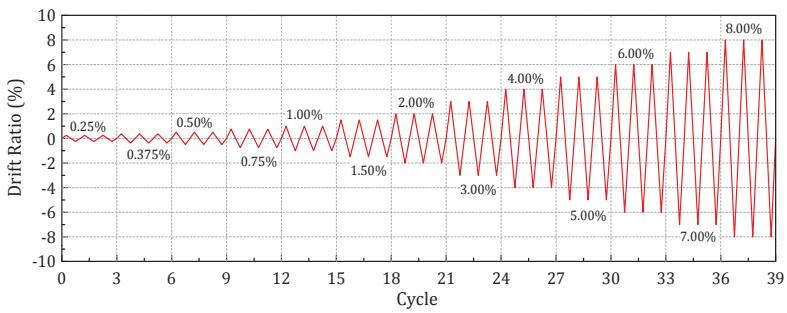
$$P = 12749\,kN$$

$$0.455 A_g f_c'$$

High Axial Load

Lateral Load

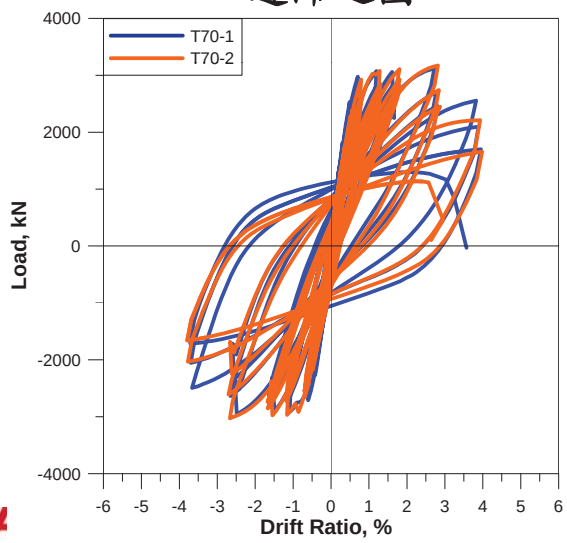




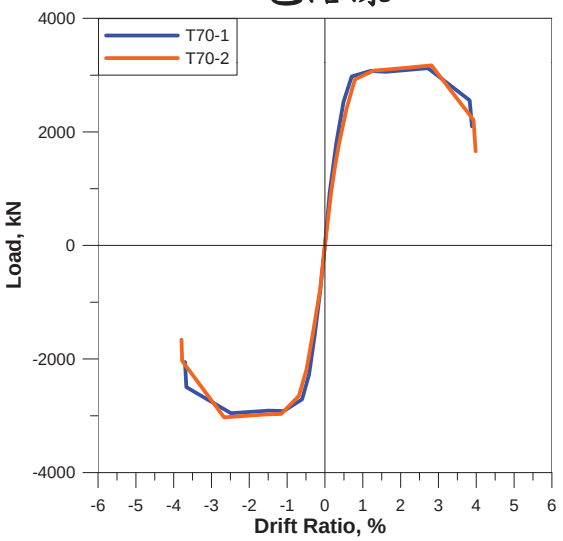
遲滯迴圈與包絡線

Specimen ID	f'_c (MPa)	V_{max} (kN)	Drift@V_{max} (%)	0.8V_{max}	Drift@0.8V_{max} (%)
T70-1	81.94	3122	2.72	2498	3.83
T70-2	81.94	3174	2.82	2539	3.55

遲滯迴圈



包絡線

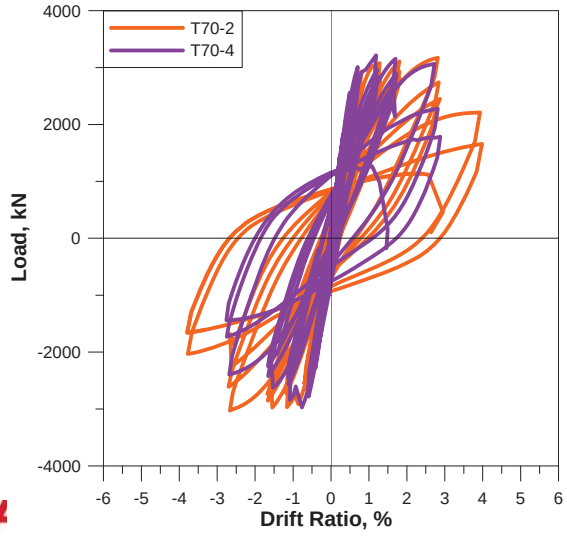


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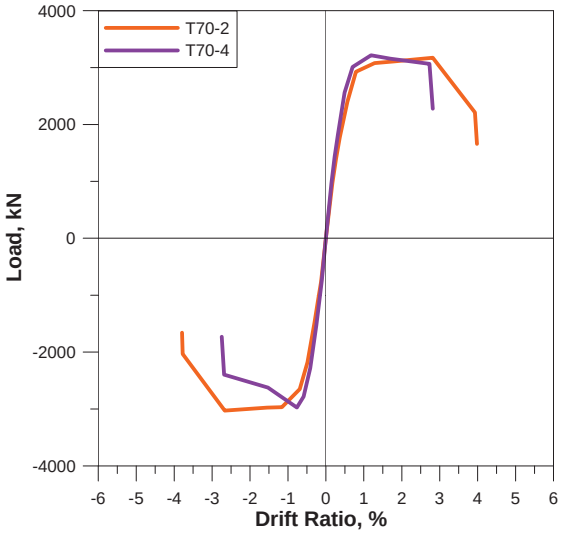
遲滯迴圈與包絡線

	Specimen ID	f'_c (MPa)	V_{max} (kN)	Drift@V_{max} (%)	0.8V_{max}	Drift@0.8V_{max} (%)
交錯	T70-2	81.94	3174	2.82	2539	3.55
非交錯	T70-4	77.91	3216	1.19	2573	2.78

遲滯迴圈



包絡線



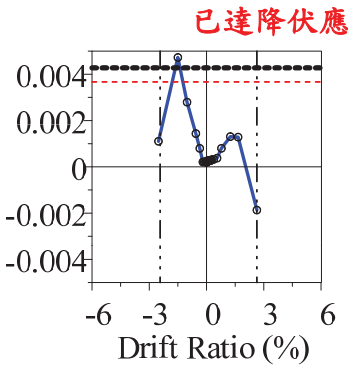
16

橫向鋼筋降伏

- 部分橫向鋼筋達降伏應變，甚至斷裂







設計強度： $f_{yt} = 785\ MPa$

強度上限： $f_{yt} \leq 700 MPa$



橫箍柱設計公式

- 美國 **ACI 318-14**

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

其中 ~~$f_{yt} \leq 700 MPa$~~

$f_{yt} \leq 800 MPa$

- 紐西蘭 **NZS 3101: 2006**

$$\frac{A_{sh}}{sb_c} = \left(\frac{1.3-\rho_t m}{3.3}\frac{A_g}{A_{ch}}\frac{f_c'}{f_{yt}}\frac{P}{\phi A_g f_c'}\right) - 0.006$$

其中 $f_{yt} \leq 800 MPa$

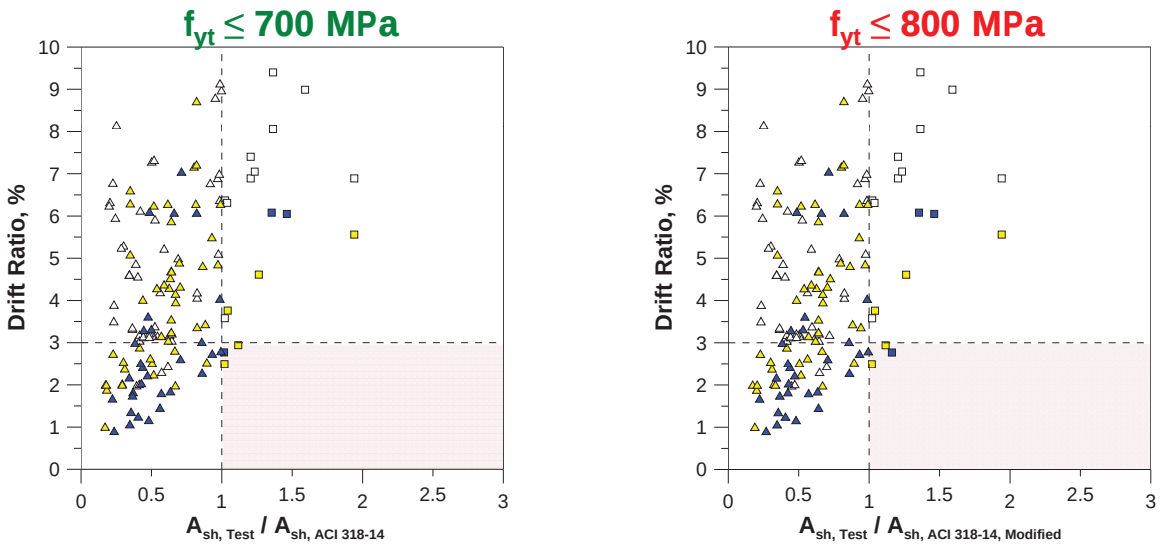
放寬橫向鋼筋強度限制

△ $P/A_g f_c' < 0.2$

▲ $0.2 \le P/A_g f_c' < 0.4$

▲ $0.4 \le P/A_g f_c'$

PEER 資料庫



設計公式之修訂

ACI 318-14	修訂建議
$\frac{A_{sh}}{sb_c} \ge \max\left\{0.09\frac{f_c'}{f_{yt}},\ 0.3\frac{f_c'}{f_{yt}}\left(\frac{A_g}{A_{ch}}-1\right),\ 0.2k_f k_n k_p \frac{f_c'}{f_{yt}}\frac{A_g}{A_{ch}}\right\}$	$\frac{A_{sh}}{sb_c} \ge \max\left\{0.09\frac{f_c'}{f_{yt}},\ 0.3\frac{f_c'}{f_{yt}}\left(\frac{A_g}{A_{ch}}-1\right),\ 0.2k_f k_n k_p \frac{f_c'}{f_{yt}}\frac{A_g}{A_{ch}}\right\}$
其中 $f_{yt} \le 700MPa$	① 其中 $f_{yt} \le$ ~~$700MPa$~~ $800MPa$
主筋必須使用耐震彎鉤加以圍束	② 放寬使用90°-180°彎鉤且交錯配置，但須提高橫向鋼筋量

圍束箍筋配置之差異

Study Case:

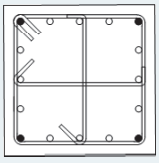
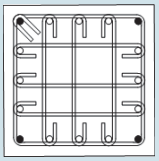
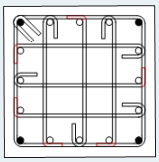
(600×600mm)

高強度材料

$f_c' = 70\ MPa$

$f_{yt} = 785\ MPa$

$$\frac{P}{A_g f_c'} = 0.45 > 0.3$$

ACI 318-11	$f_{yt} \le 700 MPa$	$\frac{A_{sh}}{sb_c} = 1.04\%$		**D16@110mm**
ACI 318-14	$f_{yt} \le 700 MPa$ $k_n = 1.14$	$\frac{A_{sh}}{sb_c} = 1.46\%$		耐震彎鉤 **D16@130mm**
修訂建議	$f_{yt} \le 800 MPa$ $k_n = 1.25$	$\frac{A_{sh}}{sb_c} = 1.36\%$		**90˚-180˚彎鉤** 交錯配置 **D16@140mm**

+40%

-7%

結論

結論

- 彎鉤型式對於高強度鋼筋混凝土柱之變形能力影響並不明顯，但90°-180°彎鉤之試體強度衰減速度較快

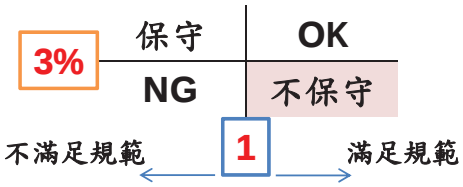
- 使用90°-180°彎鉤且交錯配置，柱變形能力仍可達合格標準

- 橫向鋼筋 f_{yt} 由 700 MPa 放寬至 800 MPa，可降低圍束箍筋量，改善過於保守之現象

NARLabs 國家實驗研究院
國家地震工程研究中心

簡報結束，敬請指教

以 ACI 318-14 檢核 NCREE 資料庫



不滿足 ACI 318

△ $P/A_g f_c' < 0.2$

▲ $0.2 \le P/A_g f_c' < 0.4$

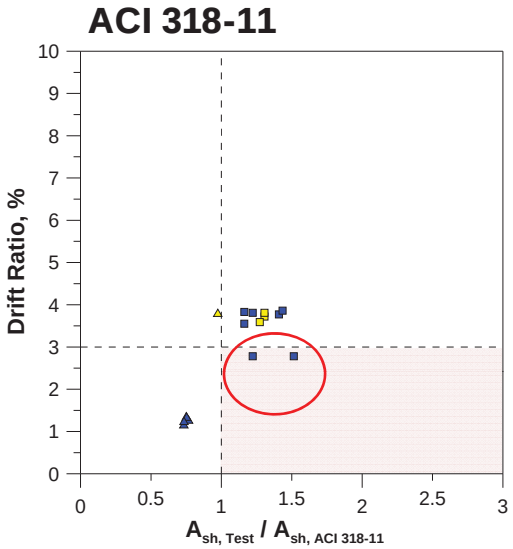
▲ $0.4 \le P/A_g f_c'$

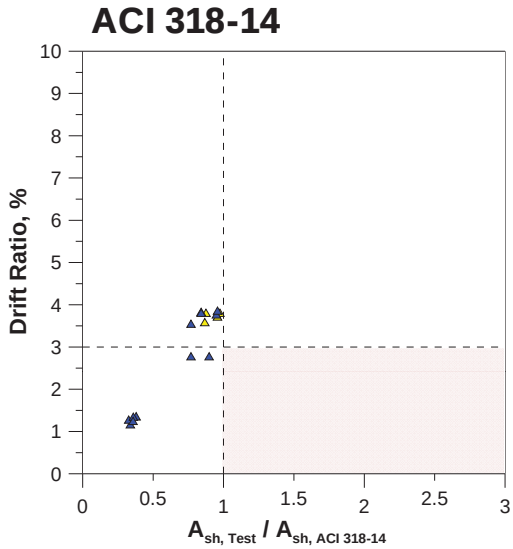
滿足 ACI 318

□ $P/A_g f_c' < 0.2$

■ $0.2 \le P/A_g f_c' < 0.4$

■ $0.4 \le P/A_g f_c'$





放寬橫向鋼筋強度限制

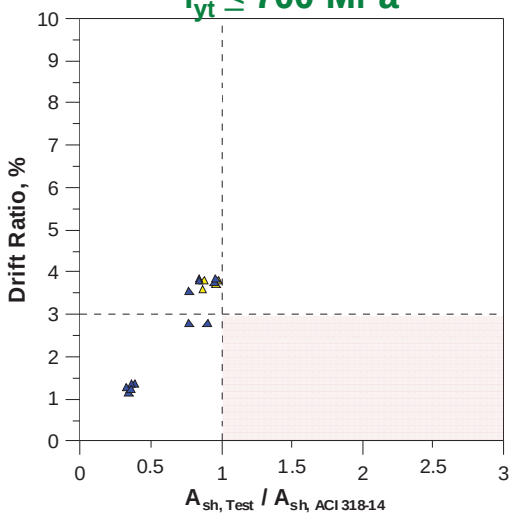
△ $P / A_g f_c' < 0.2$

▲ $0.2 \le P / A_g f_c' < 0.4$

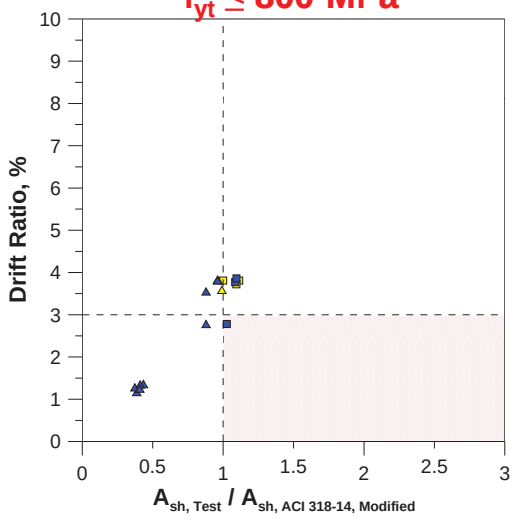
▲ $0.4 \le P / A_g f_c'$

NCREE 資料庫

$f_{yt} \le 700$ MPa



$f_{yt} \le 800$ MPa



圍束箍筋配置之差異

Study Case:

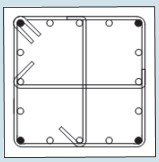
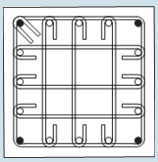
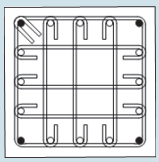
(600x600mm)

高強度材料

$f_c' = 70\ MPa$

$f_{yt} = 785\ MPa$

$$\frac{P}{A_g f_c'} = 0.45 > 0.3$$

ACI 318-11	ACI 318-14	放寬 f_{yt} 之上限 (700 → 800 MPa)
$f_{yt} \le 700 MPa$	$f_{yt} \le 700 MPa$	$f_{yt} \le 800 MPa$
	$k_n = 1.14$	$k_n = 1.14$
$\frac{A_{sh}}{sb_c} = 1.04\%$	$\frac{A_{sh}}{sb_c} = 1.46\%$	$\frac{A_{sh}}{sb_c} = 1.27\%$
		
D16@110mm	D16@130mm	D16@150mm

-13%

圍束箍筋配置之差異

Study Case:

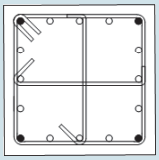
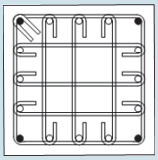
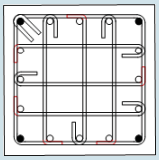
(600×600mm)

高強度材料

$f'_c = 70\ MPa$

$f_{yt} = 785\ MPa$

$$\frac{P}{A_g f'_c} = 0.45 > 0.3$$

ACI 318-11	ACI 318-14	放寬交錯配置 90°-180°彎鉤
$f_{yt} \le 700MPa$	$f_{yt} \le 700MPa$ $k_n = 1.14$	$f_{yt} \le 700MPa$ $k_n = 1.25$
$\frac{A_{sh}}{sb_c} = 1.04\%$	$\frac{A_{sh}}{sb_c} = 1.46\%$	$\frac{A_{sh}}{sb_c} = 1.59\%$
		
D16@110mm	D16@130mm	D16@120mm

+9%

NARLabs 國家實驗研究院 National Applied Research Laboratories

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高強度鋼筋混凝土 New RC專家座談會

高拉力強度鋼纖維於New RC柱之應用評估

廖文正

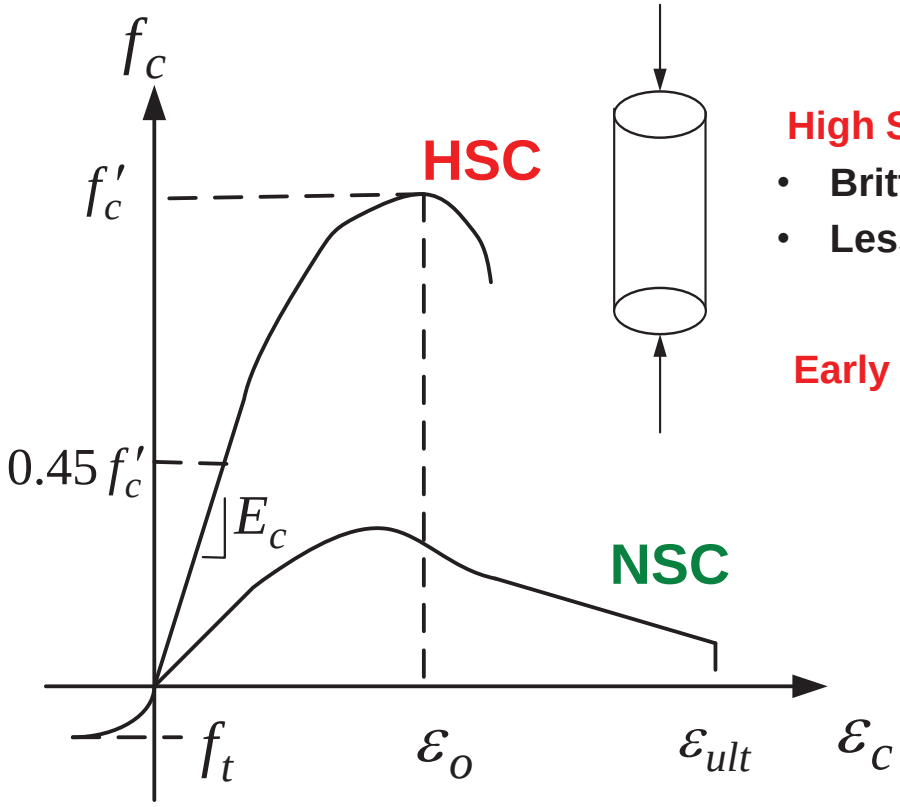
國立台灣大學 土木工程學系



Outline

- Background and Objective
- Highly Flowable Strain Hardening Fiber Reinforced Concrete (HF-SHFRC)
- Proposed Toughness Ratio / Shear Strength Equation
- Experimental Programs
- Conclusions

Background and Objective

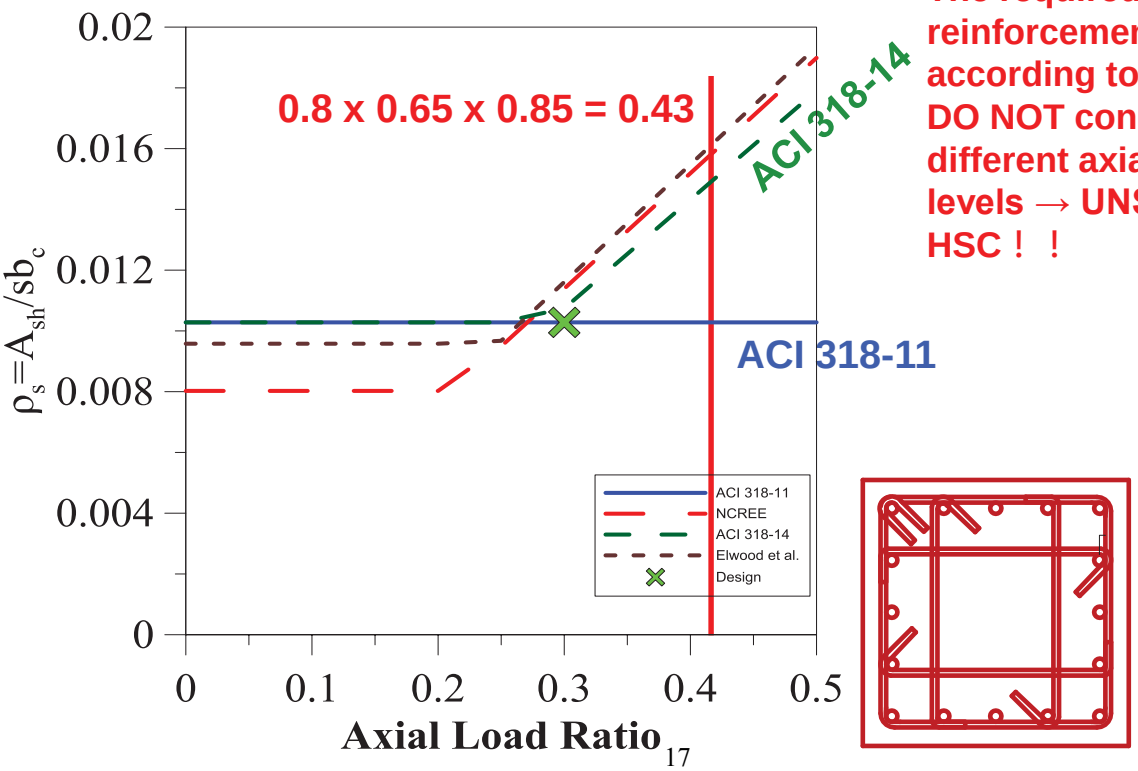


High Strength Concrete

- **Brittle**
- **Less Deformation Capacity**

Early spalling and brittle failure

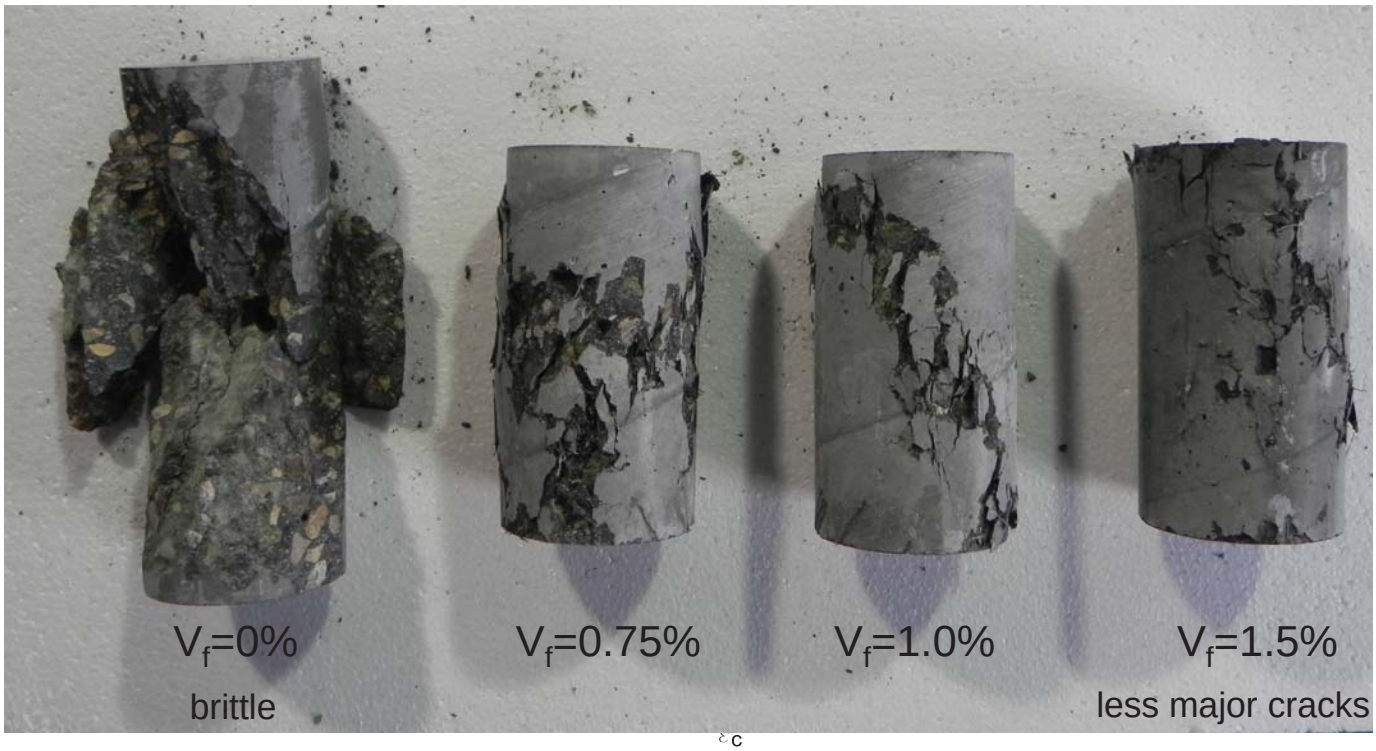
Background and Objective



The required transverse reinforcement calculated according to ACI 318-11 DO NOT consider different axial loading levels → UNSAFE for HSC！！

Background and Objective

Alternative solution: Fibers



ε c

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台北, May 27, 2015

Background and Objective

- **ACI 318-11**

11.4.6 — **Minimum shear reinforcement**

11.4.6.1 (f)

Beams constructed of steel fiber-reinforced, normal weight concrete with f_c' not exceeding 6000 psi, h not greater than 24 in., and V_u not greater than $2\sqrt{f_c'}\, b_w d$.

R11.4.6.1(f) — This exception is intended to provide a design alternative to the use of shear reinforcement, as defined in 11.4.1.1, for members with longitudinal flexural reinforcement in which V_u does not exceed $2\sqrt{f_c'}\, b_w d$. Fiber-reinforced concrete beams with hooked or crimped steel fibers in dosages as required by 5.6.6.2 have been shown, through laboratory tests, to exhibit shear strengths larger than $3.5\sqrt{f_c'}\, b_w d$.

Highly Flowable Strain Hardening Fiber Reinforced Concrete (HF-SHFRC)

Main Obstacles for Fiber Reinforced Concrete

1. Workability
2. Mechanical performances is not as good as expected.
3. Expensive; no main advantages on normal structural applications.

Highly Flowable Strain Hardening Fiber Reinforced Concrete



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Highly Flowable Strain Hardening Fiber Reinforced Concrete





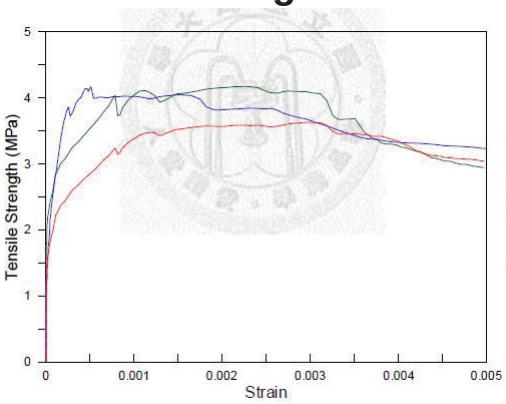


圖 5－12　C30 直接拉力應力應變圖（0.5％）

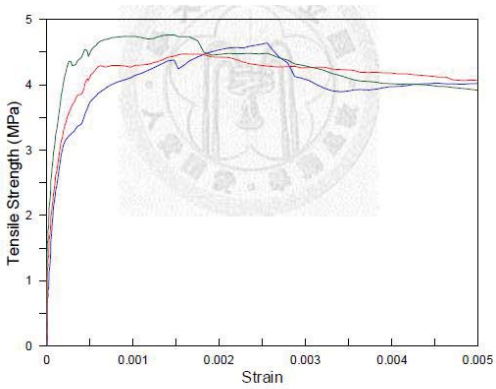


圖 5－14　C40 直接拉力應力應變圖（0.5％）

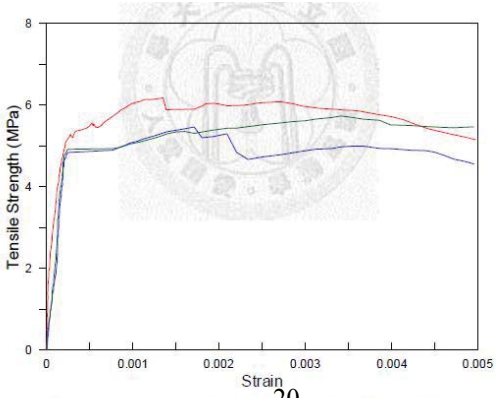


圖 5－16　C50 直接拉力應力應變圖（0.5％）

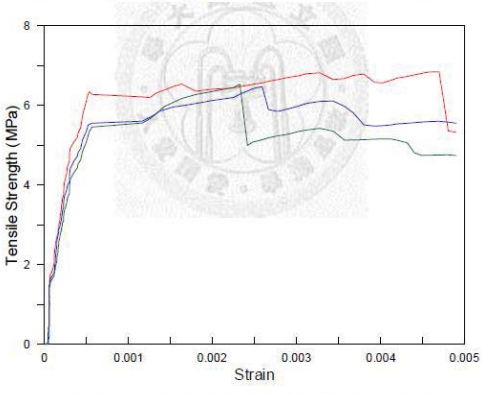
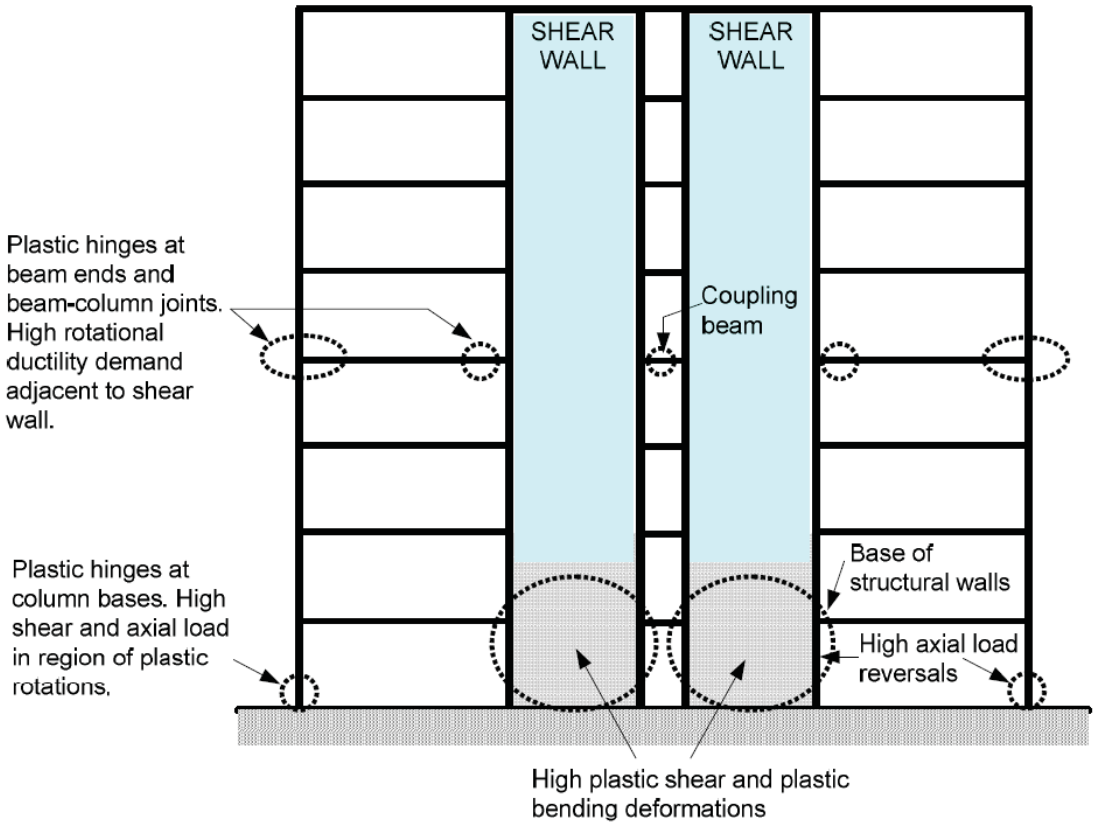


圖 5－18　C60 直接拉力應力應變圖（0.5％）

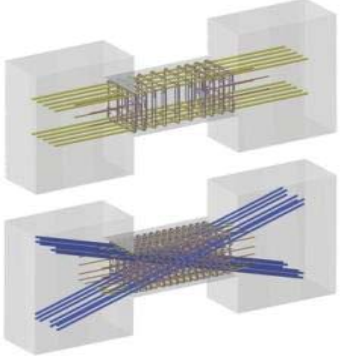
Highly Flowable Strain Hardening Fiber Reinforced Concrete



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台北, May 27, 2015

Highly Flowable Strain Hardening Fiber Reinforced Concrete





DAILY JOURNAL OF COMMERCE

Thursday, January 31, 2013

Steel fiber concrete cuts need for rebar

■ *A new mix allowed crews to build shear wall link beams with 40 percent less rebar at The Martin apartments in Belltown.*

By LUCY BODILLY
Special to the Journal

Pouring concrete into congested rebar is difficult: Concrete is hard to place and vibrating around the tangle of rebar to remove the voids makes it even harder.

Structural engineer Cary Kopczynski, senior principal and CEO of Cary Kopczynski & Co. in Bellevue, found a solution. At his latest project, The Martin, his firm specified steel fiber concrete in the shear wall link beams, a place that is especially congested with rebar.

CONSTRUCTION INDUSTRY SPOTLIGHT

The Martin is a 24-story apartment building under construction at Fifth and Lenora in Seattle's Belltown neighborhood, particularly during an earthquake.

"Our objective in using fiber was to reduce the quantity of rebar, making the beams easier to construct, and replace the lost strength (due to the reduction in rebar) with steel fiber mixed into the concrete," Kopczynski said.

He first heard about testing of the steel fibers while working on a national committee that reviews building codes. Also on the committee was a University of Michigan professor who was studying the steel fibers.

"The goal is to introduce modifications to the International Building Code that will make the design approach used for The Martin mainstream," Kopczynski said.

The recession and construction downturn stalled use of steel fiber concrete, adding to the hurdles the new technology faced. The first hurdle was to convince building owner Vulcan Real Estate to try the product.

"There is a wide range of

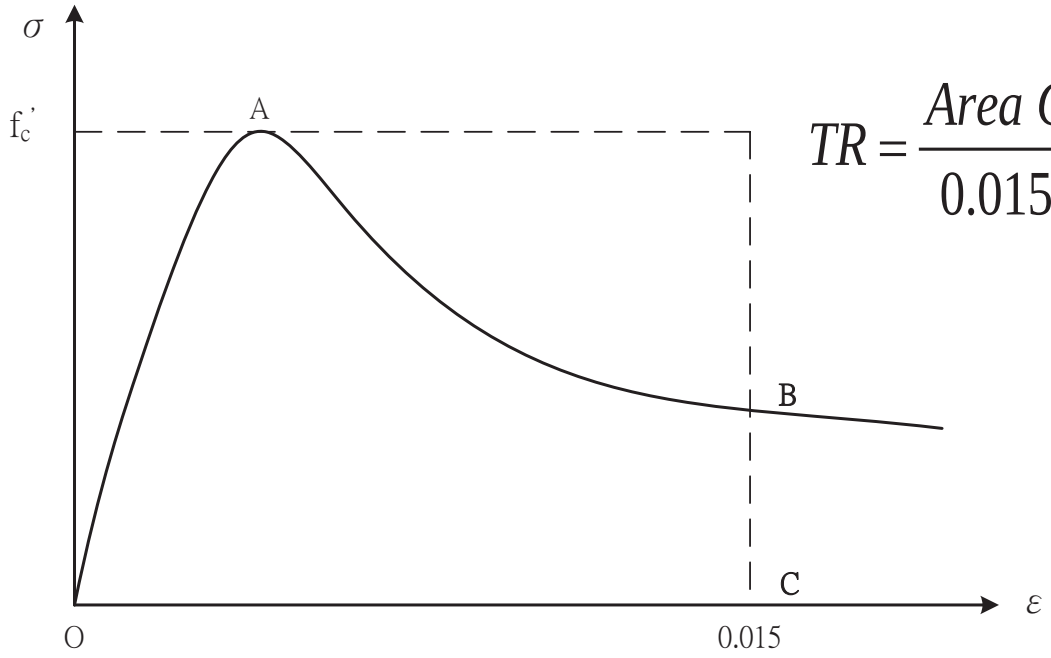


The city required two peer reviews before it would accept the new technology. Photos courtesy of Cary Kopczynski & Co.

Proposed Toughness Ratio / Shear Strength Equation

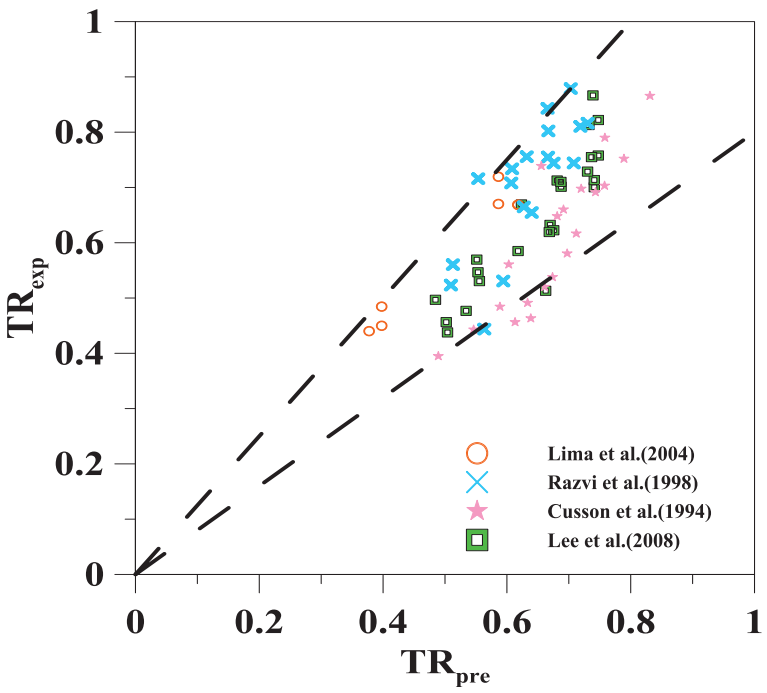
Confinement Efficiency

Toughness Ratio, TR



$$TR = \frac{Area\ OABC}{0.015 \times f'_c}$$

Proposed Toughness Ratio Equation



$$TR = 0.45\left(\frac{k_e \rho_s f_{yt}}{f'_c}\right)^{0.19} + \chi_f \left(V_f \times \frac{L}{\varphi} \times \tau_{eq}/f'_c\right)$$

k_e = confinement effectiveness coefficient

ρ_s = transverse reinforcement ratio

f_{yt} = yield strength of transverse reinforcement

f_c' = compressive strength of concrete

V_f = steel fiber volume fraction

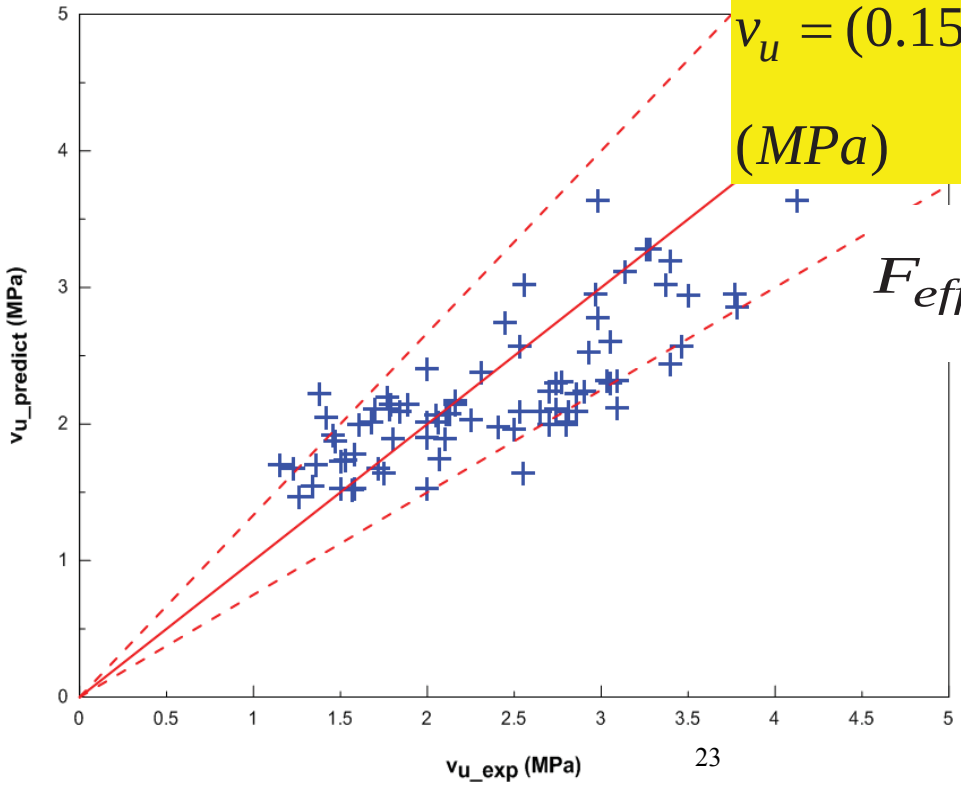
L/φ = aspect ratio of steel fiber

X_f = fiber efficiency factor, f (stirrups spacing)

τ_{eq} = equivalent bond strength

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Proposed Shear Strength Equation



$$v_u = (0.15\sqrt{f'_c} + 70\rho\frac{d}{a})F_{eff} \quad (MPa)$$

$$F_{eff} = 1 + \frac{F_{po}}{0.7\sqrt{f'_c}}$$

$$F_{po} = \left(\frac{L_f}{D_f}\right)V_f \tau_{eq}$$

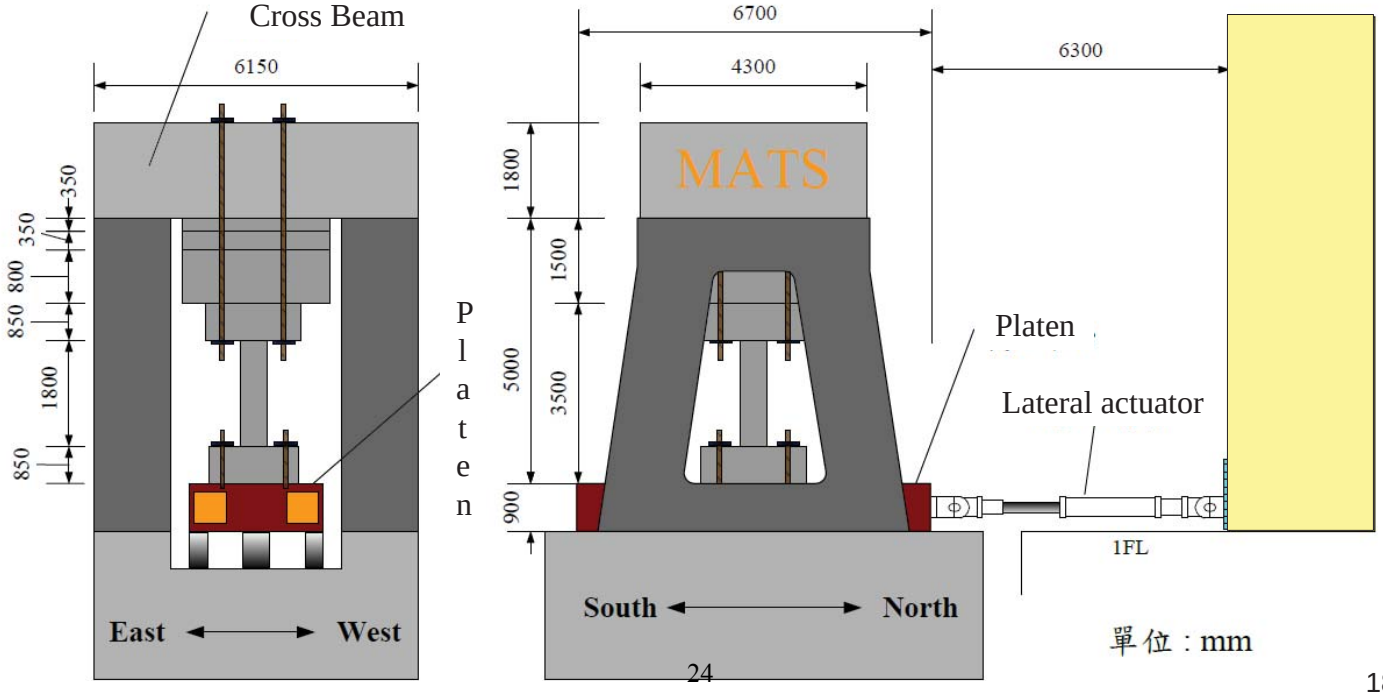
Experimental Program

Experimental Program

Multi-Axial Resting System (MATS)

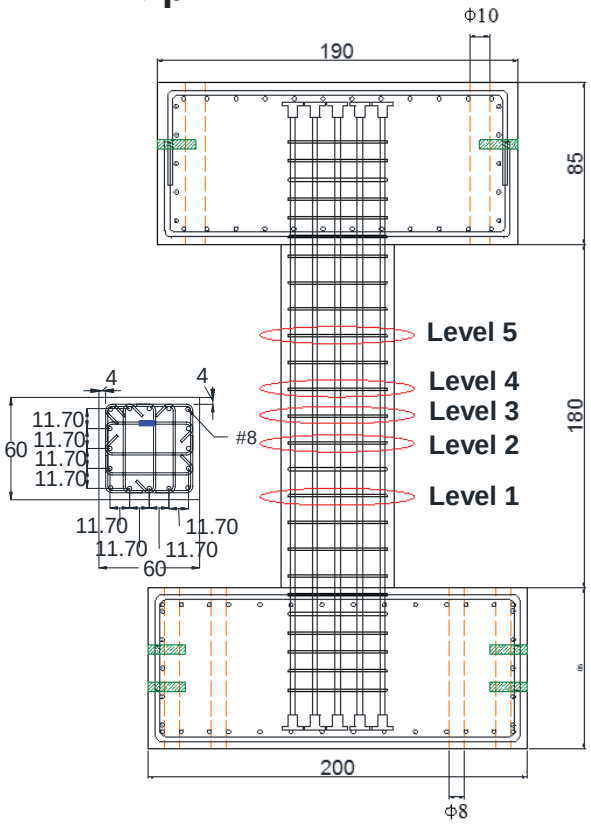
Axial loading capacity: up to 60 MN

National Center for Research on Earthquake Engineering (NCREE)



Experimental Program

Specimen: 600 x 600 x 1800 mm



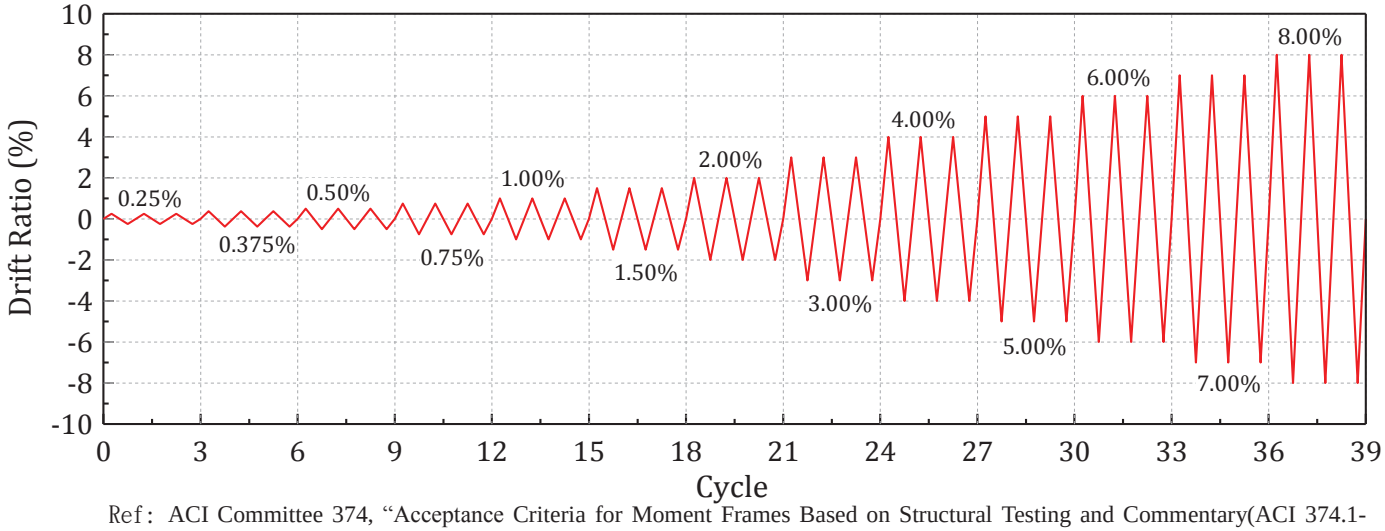


1. Double Curvature
2. Aspect ratio is 3 only

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Experimental Program

Loading Protocol



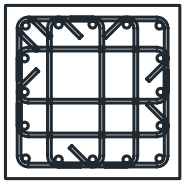
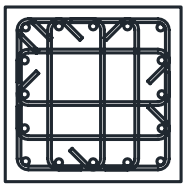
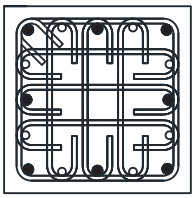
Ref: ACI Committee 374, "Acceptance Criteria for Moment Frames Based on Structural Testing and Commentary(ACI 374.1-05)," American Concrete Institute, 2006.

Criteria:

Ultimate drift ratio, UDR (drift ratio corresponding to 80% Max lateral capacity) > 3.0%

25

Experimental Program

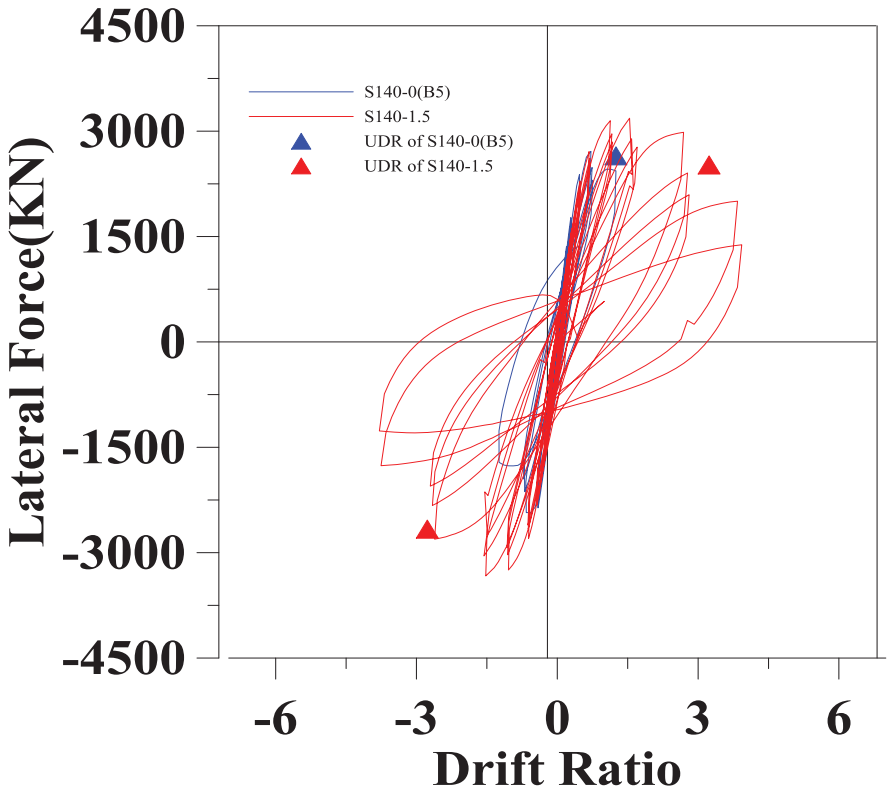
No.	Section	ρ_s (%)	f'_c (MPa)	f_{yl} (MPa)	f_{yt} (MPa)	s (mm)	$\frac{P}{A_g f'_c}$	TR
S140-0 **(ACI 318-11)**		1.74	83.4	#8, SD685	#4, SD785	140	**0.57**	0.71
S140-1.5		1.74	73 (**V_f= 1.5%**)	#8, SD685	#4, SD785	140	**0.57**	0.80
S260-1.5		**1.46**	72 (**V_f= 1.5%**)	#8, #10, SD685	#5, SD785	**260 (d/2)**	**0.43**	0.80

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台北, May 27, 2015

Test Results
S140-0 VS. S140-1.5

Test Results

S140-0 (UDR: 1.25%) VS. S140-1.5 (UDR: 3.23%)

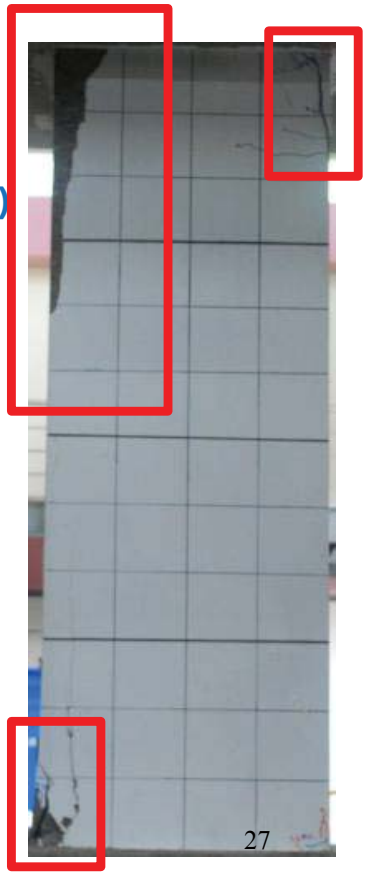


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Test Results

Drift ratio: 1.0%

S140-0 (Max Lateral Capacity)



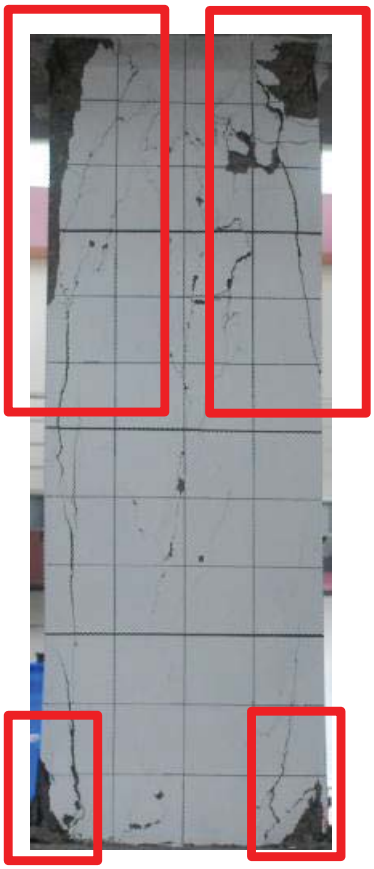


S140-1.5

Test Results

Drift ratio: 1.5%

S140-0





S140-1.5 (Max Lateral Capacity)

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Test Results

Failure

S140-0 (2nd cycle of 1.5%)





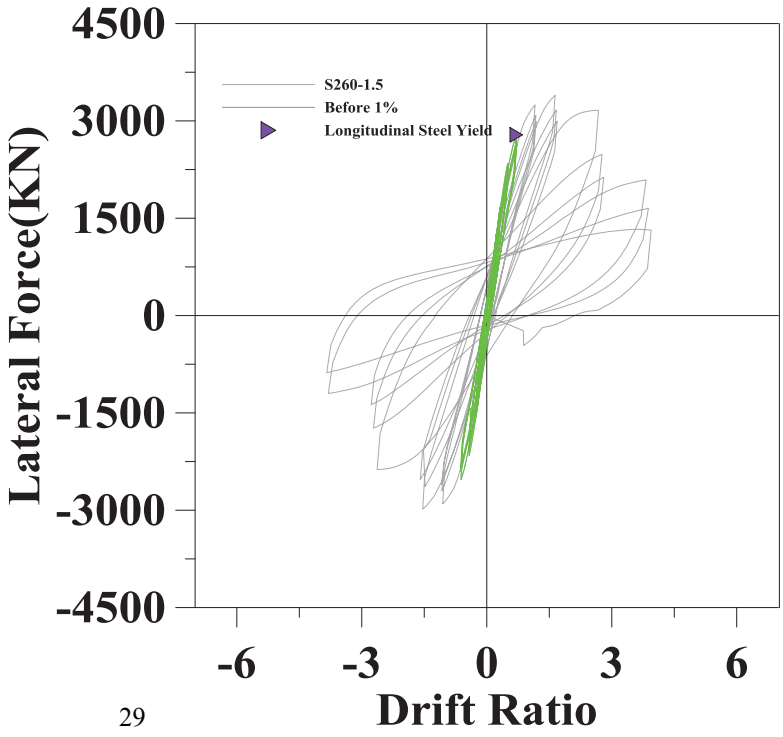
S140-1.5 (4.0%)

Test Results
S260-1.5 (spacing D/2)

Test Results



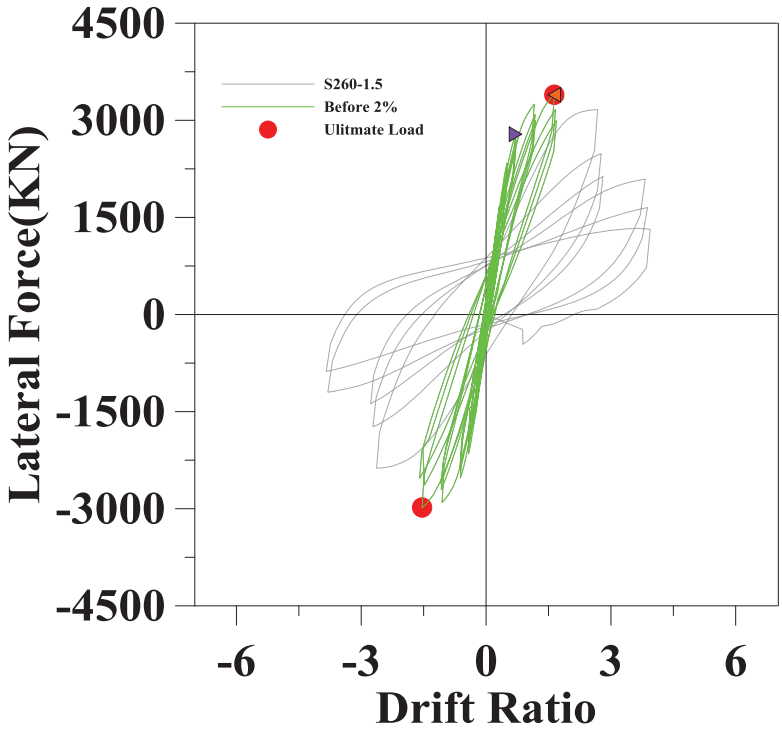
Test Results (S260 – 1.5) 1.0%



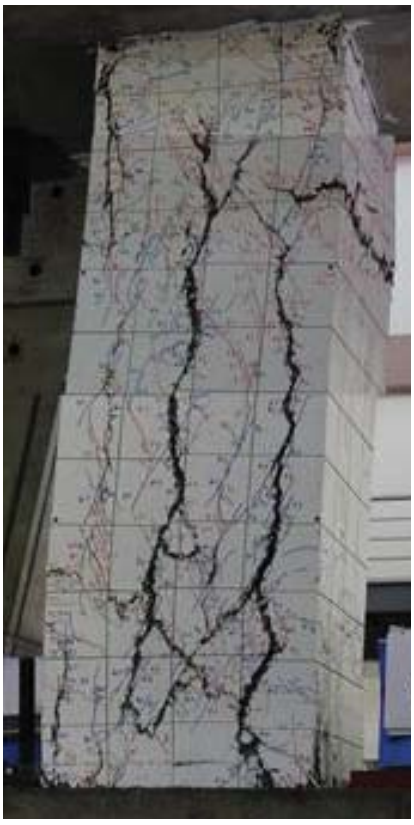
Test Results



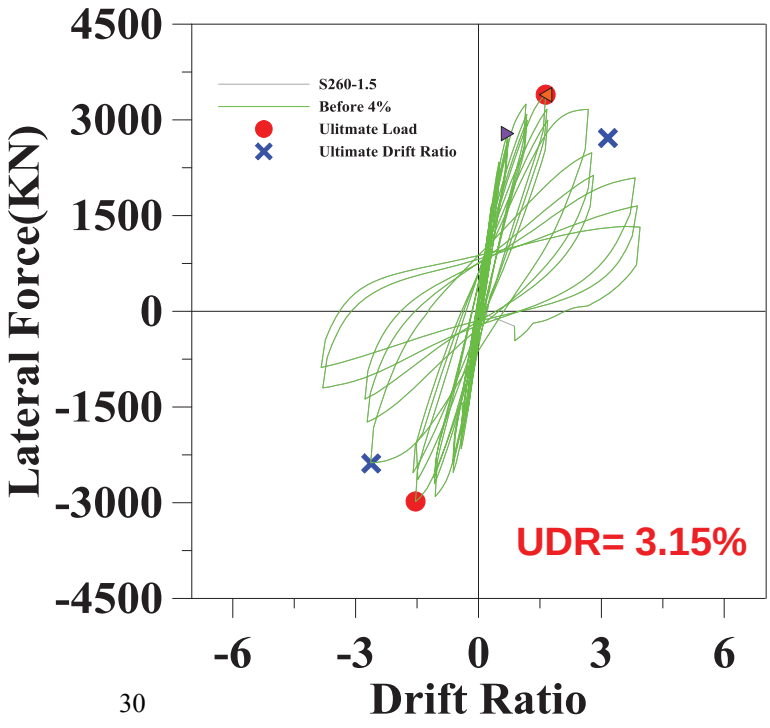
Test Results (S260 – 1.5) 2.0%



Test Results



Test Results (S140 – 1.5) failure



Conclusions

(1) Addition of fibers provides **high ductility** to brittle concrete, particularly for high strength concrete (NEW RC)

(2) The transverse reinforcement requirement in **ACI 318-11 seems not adequate for high strength concrete / under high axial loading** (S140-0).

(3) Compared to HSC, HF-SHFRC presents **high toughness, damage tolerance and excellent shear capacity** while allowing **good workability**. The experimental results shows that addition of **fibers can be an effective alternative** for transverse reinforcement in NEW RC columns.

(4) By **increasing the spacing to d/2 and under 0.43 axial loading level**, excellent performance and damage tolerance can be still observed in S260-1.5. It shows the **great potential** to apply **HF-SHFRC to NEW RC columns**.

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高強度鋼筋混凝土 New RC專家座談會
台北, May 27, 2015

Thank you ☺

國立台灣大學土木工程學系
助理教授 廖文正
wcliao@ntu.edu.tw

PRELIMINARY DESIGN RECOMMENDATION FOR RC SQUAT WALL USING HIGH STRENGTH REINFORCEMENT

Min-Yuan Cheng

National Taiwan University of Science and Technology



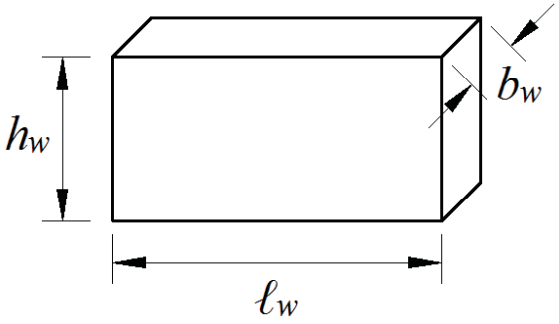
NCREE, May 26th, 2015

Outline

- Definition
- Design
  - Axial – Flexure
  - Shear
- Detailing
- Experimental Verification

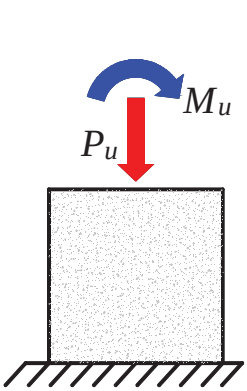
Definition :

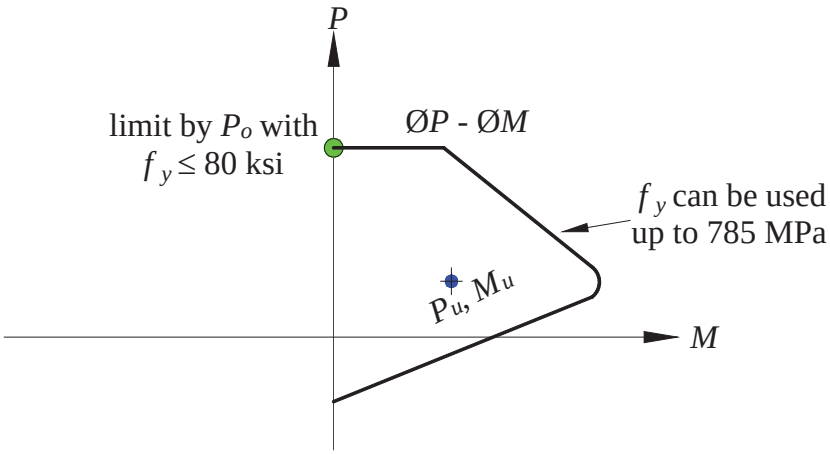
$$\frac{h_w}{\ell_w} \le 1.5 \text{ and } \frac{\ell_w}{b_w} > 6.0$$



DESIGN :

Axial - Flexure





- Ensure P_u, M_u is within the $ØP - ØM$ curve

DESIGN :

Shear

$$V_n = A_{cv}\left(3\sqrt{f'_c} + \rho_t f_{yt}\right)$$

A_{cv} : gross−section area of the wall

f_{yt} : ≤ 600 MPa

$\sqrt{f'_c}$: limit to 0.7 MPa (100 psi)

- Ensure $\emptyset V_n \geq \frac{M_n}{h_w}$

DETAILING :

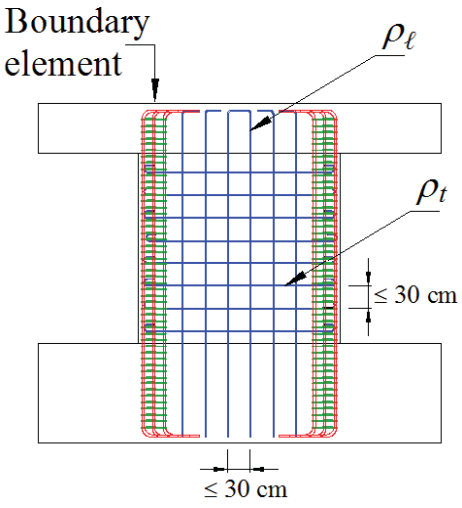
1. ρ_ℓ should be at least ρ_t

2. Spacing of distributed $\left\{\begin{matrix}\text{vertical}\\ \text{horizontal}\end{matrix}\right\}$ web reinforcement ≤ 30 cm (12 in.)

3. $V_u \leq \emptyset 10\sqrt{f'_c}\, A_{cv}$

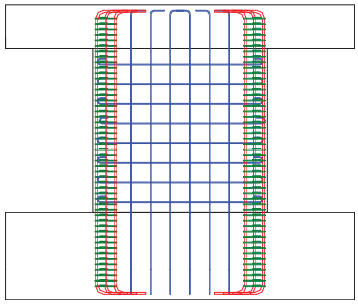
   ⇒ suggest to be less than $\emptyset 6\sqrt{f'_c}\, A_{cv}$

4. Satisfy boundary element requirements as current ACI Building Code (318-14)



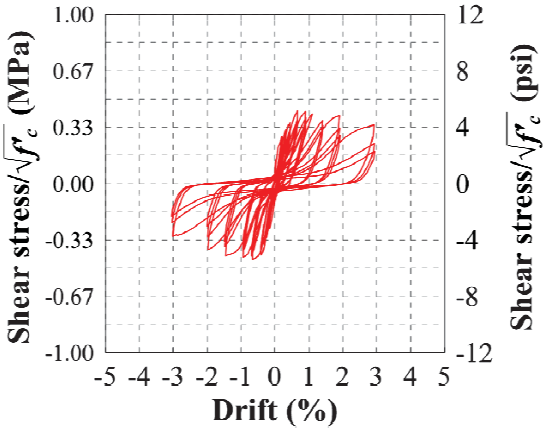
EXPERIMENTAL VERIFICATION:

M1

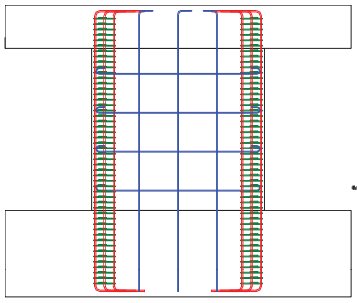


$\frac{V_n}{M_n / a} = 1.20$

f_{yt} = test data

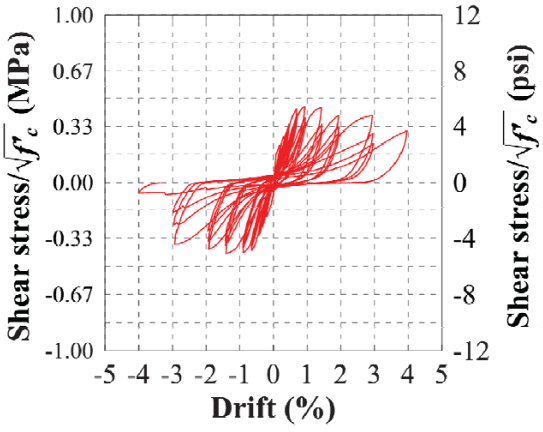


M2



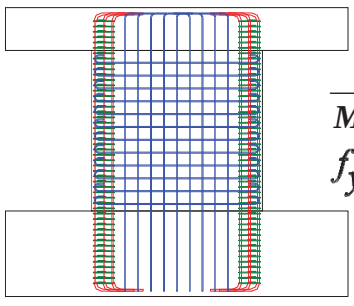
$\frac{V_n}{M_n / a} = 1.05$

f_{yt} = 600 MPa



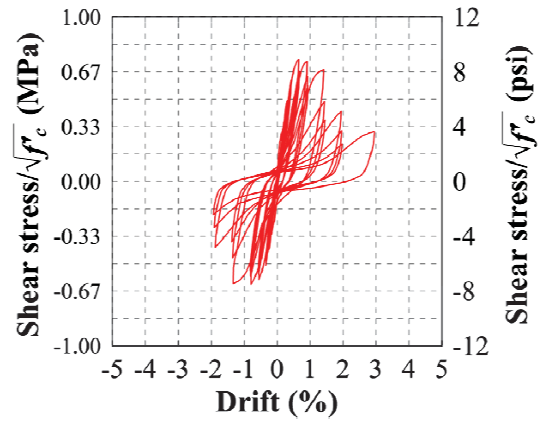
EXPERIMENTAL VERIFICATION:

H1

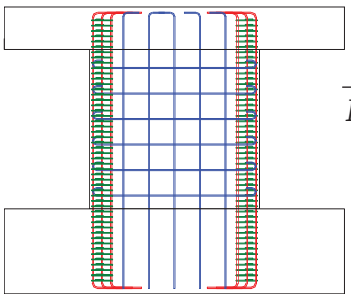


$\frac{V_n}{M_n / a} = 1.28$

f_{yt} = test data

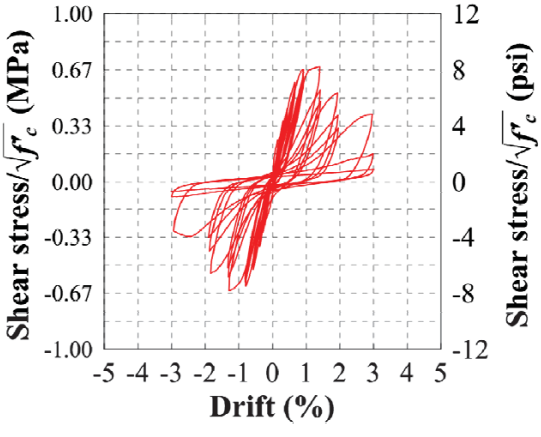


H2



$\frac{V_n}{M_n / a} = 1.05$

f_{yt} = 600 MPa



Thank you
for your attention !

高強度鋼筋混凝土剪力牆連接梁之試驗驗證

林敏郎、黃世建、鄭敏元、洪崇展
2015/5/27
國家地震工程研究中心

簡報大綱

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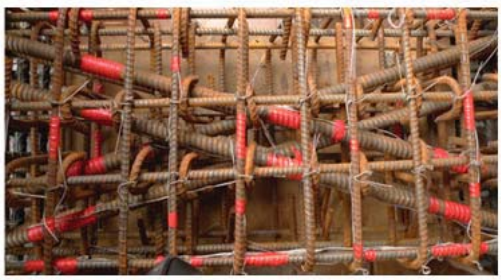
- ACI 318規範對於剪力牆連接梁之相關規定
- 國震中心高強度鋼筋混凝土連接梁試驗
- 結論

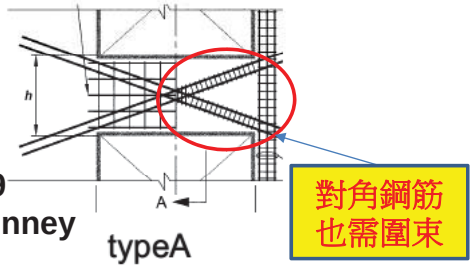
ACI 318規範對於
剪力牆連接梁之規定

ACI 318-08 規範

- 跨深比大於4，須滿足特殊抗彎構架之相關規定
- 跨深比（ln/h）小於2時，需配置對角鋼筋籠



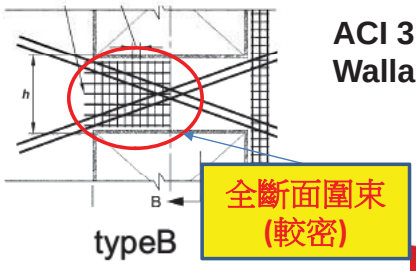




ACI 318-99
Paulay&Binney
1974

typeA

對角鋼筋
也需圍束



typeB

全斷面圍束
(較密)

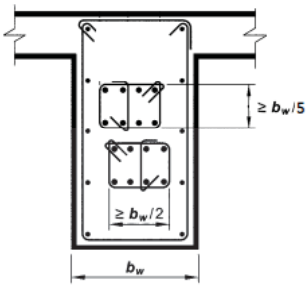
ACI 318-08
Wallance 2007

- ACI318對於連接梁之規定
- 跨深比（ln/h）小於2，時需配置對角鋼筋籠
- 對角鋼筋籠尺寸須滿足：寬度大於bw/2；深度大於bw/5
- ACI318規定之剪力標稱強度

$$V_n = 2A_d f_{yd} \sin\alpha \leq 2.65\sqrt{f'_c} A_{cw}$$

僅由對角鋼筋提供

高強度材料是否適用?



國震中心高強度鋼筋混凝土連接梁試驗研究

國震中心近年連接梁試驗研究

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- 2010 鄭志宏,鋼筋混凝土剪力連接梁反覆載重測試之研究(8組)
- 2011 王亭惟,鋼筋混凝土剪力連接梁耐震鋼筋配置之探討(7組)
- 2012 張于軒,鋼筋混凝土剪力牆連接梁鋼筋配置之研究(8組)
- **2013 楊善淳,高強度鋼筋混凝土剪力牆連接梁耐震行為研究(9組)**
- **2013 蔡尚錡,鋼筋混凝土剪力牆連接梁耐震行為之研究(8組)**
- 2014 林秉誼,鋼筋混凝土剪力牆連接梁耐震配筋之研究(8組)

（未計入鄭敏元教授、洪崇展教授之試體）

◆ 探討重點參數包括：

1. 跨深比
2. 對角鋼筋配置形式
3. 對角鋼筋配置量
4. 圍束量
5. 力傳遞機制
6. **混凝土強度**、鋼纖維混凝土使用
7. **鋼筋強度**

高強度鋼筋混凝土剪力牆連接梁試驗

NARLabs

- 2013 楊善淳,高強度鋼筋混凝土剪力牆連接梁耐震行為研究
- 2013 蔡尚錡,鋼筋混凝土剪力牆連接梁耐震行為之研究(針對簡化配筋)

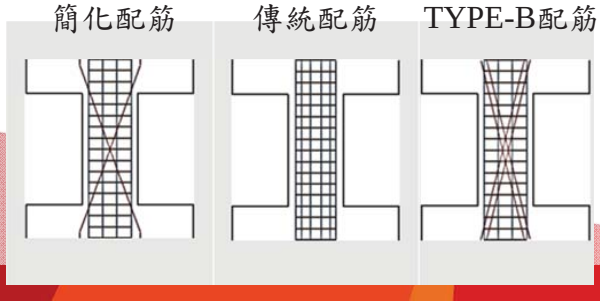
◆ 目的：

探討高強度材料是否適用於美國規範ACI318，探討提升縱向筋、對角筋、箍筋與混凝土的強度之效益，並希望改善複雜的對角鋼筋籠配置

◆ 探討參數：

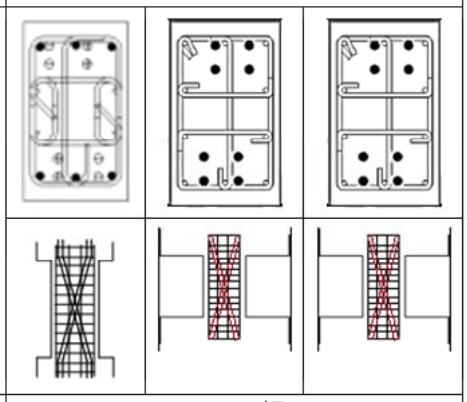
1. 對角鋼筋配置形式
2. 材料強度
3. 對角鋼筋比

對最大強度、延展性、韌性（消能行為）進行觀察



ACI 318 TYPE-B型式配筋試體(I)

NARLabs

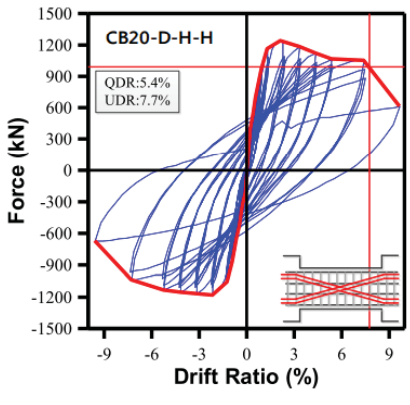
名稱		CB20-D-H-H	CB203	CB302
跨深比		2	2	3
斷面(cm^2)		30x50		
鋼筋 (kgf/cm^2)	#4	4200	7850	7850
	#8	6850		
混凝土(kgf/cm^2)		700		
斷面圖示				
縱向筋		◦ #4(10 根)		
對角筋	根數	● #8(8 根)		
	角度	15˚	14˚	9˚
箍筋		#4@10		
受拉鋼筋比		0.0196	0.0197	0.0199
對角鋼筋比 η		75.5%	75.6%	76%

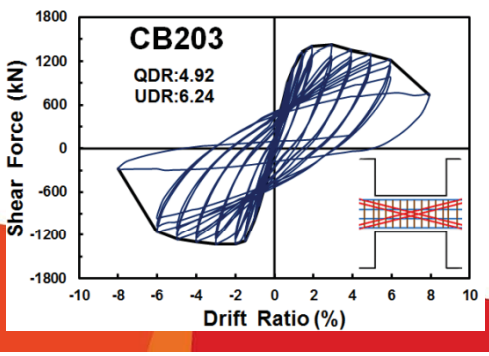
CB20-D-H-H：(蔡尚錡)
箍筋與縱向鋼筋使用4號SD420
CB203與CB302：(楊善淳)
箍筋與縱向鋼筋使用4號SD685

ACI 318 TYPE-B型式配筋試體(II)

NARLabs

試體編號	CB20-D-H-H	CB203	CB302
Vfn=2Mn/L (剪力需求)	1226	1447	964.8
V=Vs+Vc+Vds (假設值)	1607	2172	1935
V=Vs*+Vc+Vds (箍筋值)	1607 (100%)	1843 (75%)	1277 (50%)
Vn= $2A_{vd}f_{yd}sin\alpha$ (ACI僅對角筋)	718	671	434
Vn≦ $2.65\sqrt{f_c'}A_{cw}$ (ACI上限值)	927	927	927
Vmax (實驗值)	1238	1420	1064.2
極限層間位移 (UDR)	7.7	6.24	6.22

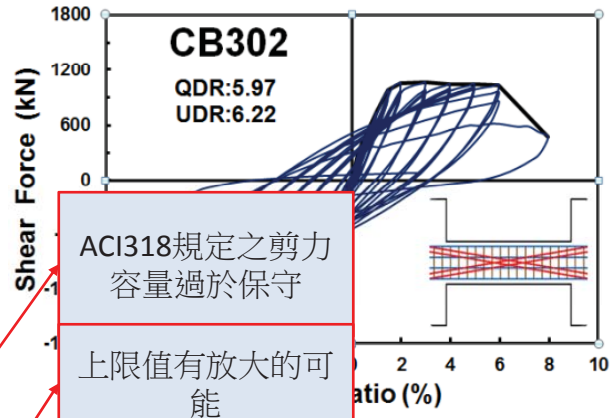




ACI 318 TYPE-B型式配筋試體(III)

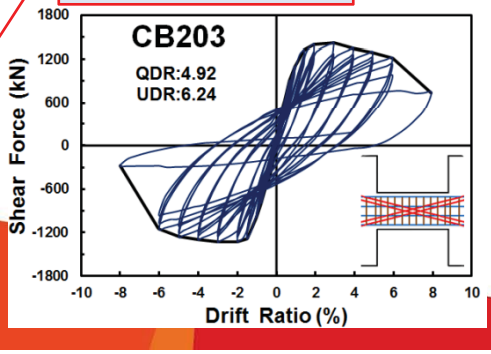
NARLabs

試體編號	CB20-D-H-H	CB203	CB302
Vfn=2Mn/L (剪力需求)	1226	1447	964.8
V=Vs+Vc+Vds (假設值)	1607	2172	1935
V=Vs*+Vc+Vds (箍筋值)	1607 (100%)	1843 (75%)	1277 (50%)
Vn= $2A_{vd}f_{yd}sin\alpha$ (ACI僅對角筋)	718	671	434
Vn≦ $2.65\sqrt{f_{c'}}A_{cw}$ (ACI上限值)	927	927	927
Vmax (實驗值)	1238	1420	1064.2
極限層間位移 (UDR)	7.7	6.24	6.22



ACI318規定之剪力容量過於保守

上限值有放大的可能



NARLabs

結論

結論

NARLabs

1. 經試驗驗証，ACI Type B型式之高強度鋼筋混凝土連接梁可有良好的變形能力(UDR>6%)
2. 連接梁的剪力強度不該僅由對角鋼筋提供，實驗結果呈現可以加入箍筋與混凝土貢獻
3. ACI 318對於剪力強度僅考慮對角向鋼筋之貢獻，可保守估算連接梁之剪力強度，但恐過於保守而造成連接梁配筋量增加或斷面尺寸過大

本年度新試體設計

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- CB1.5-D-H vs CB1.5-D-N

討論提升對角鋼筋強度之行為

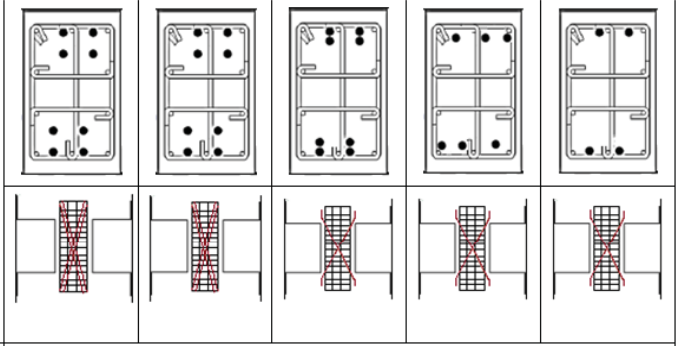
- CB1.5-P4-N vs CB1.5-D-H

討論對角鋼筋籠限制的必要性

- CB1.5-P3-N vs CB1.5-D-H
- CB1.5-P2-N vs CB1.5-D-H

討論

1. **對角鋼筋籠限制的必要性**
2. **使用高強度鋼筋是否可減少配筋量，以便施工**

名稱		CB1.5-D-N	CB1.5-D-H	CB1.5-P4-H	CB1.5-P3-H	CB1.5-P2-H
跨深比		1.5				
斷面(cm^2)		30x50				
鋼筋	#3/#4	4200				
(kgf/cm^2)	#8	4200	6850			
混凝土(kgf/cm^2)		700				
斷面圖示						
縱向筋		◦ #3(10 根)				
對角筋	根數	● #8(8 根)			● #8(6 根)	● #9(4 根)
	角度	22.4°		25°		26.5°
箍筋		#3@7.5				
受拉鋼筋比		0.0171	0.0171	0.0168	0.0133	0.0116
對角鋼筋比 η		84%	84%	83.7%	79.4%	76.5%
Mn(KN-m)		378.6	534.3	529.2	417.4	375.8