東和鋼鐵企業股份有限公司
TUNG HO STEEL ENTERPRISE CORPORATION

高強度鋼筋及續接器之鋼材測試研究

主講人：陳昌雄

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經歷：	東和高雄廠副廠長(兼行政處)	10111~ 10212
	東和高雄廠廠務處	9611 ~ 10111
	東和苗栗廠廠務處(技術處)	9211 ~ 9611
	東和高雄廠生管課	8701 ~ 9211
	東和高雄廠煉鋼課	7709 ~ 8612
	粉末冶金 / 精密鑄造	7210 ~ 7708
學歷：	成大冶金及材料工程學系	6610 ~ 7006

一、前言
二、目前 CNS 標準及相關
三、未來高強度鋼筋相關規範
四、強化理論
五、490提升到→685→785
六、測試與研究
七、目前具體作法
八、未來努力方向
九、結論

一、前言

2013年1-12月全球鋼鐵粗鋼產量累計為158,249萬噸，較2012年152,917萬噸上升3.5%，較2011年150,025萬噸上升5.5%。

2013年1~12月台灣累計粗鋼產量，電爐 969.90萬噸小於高爐1,258.28萬噸，年累計相比電爐成長1.3%，高爐成長13.4%。

鋼筋： 2013年1~12月累計表消579萬噸，與去年同期573萬噸相比為101%，年度自給率101%。 預估2014年累表消592萬噸，與2013年同期比103%，年度自給率約101%。

大陸今年預估2014年粗鋼產量8.1億噸與去年同期7.79億噸比為成長3.97%

預計2015年世界鋼鐵產能將達到22.83億噸，遠遠超過16.63億噸的消費量

2013年中國鋼材出口量約為6000萬噸，
2014年中國鋼材的出口量約為8000萬噸，**相當於美國全年粗鋼產量。**

只有提升高附加價值產品　才能有競爭力

表1. 2012年國內鋼筋廠產能利用率

類型	家數	2012年產量（單位：千公噸）	市佔率	年產能（單位：千公噸）	產能利用率
電爐廠	10	4,048	70%	5,436	74%
單軋廠	15	1,731	30%	3,630	48%
合　計	25	5,779	100%	9,066	64%

資料來源：台灣鋼鐵工業同業公會

表4. 鋼筋用途別鋼種尺寸分佈表

用途別	選樣工程數	鋼種分佈比(%)					尺寸分佈比(%)		
		SD280	SD280W	SD420	SD420W	其他	D10~D13	D16~D22	D25~
公共工程	37	11	3	18	68	—	10	42	48
營建業	40	15	6	48	30	1	36	25	39

資料來源：豐興鋼鐵(股)公司提供

中拉鋼筋(SD280/SD280W)與高拉力鋼筋
(SD420/SD420W/其他如SD490)之銷售比例約為20%/80%

營建業與公共工程耗用量比例約為75%/25%

採自台灣鋼鐵五十年專刊

二、目前 CNS 標準及相關規範

《CNS 560鋼筋用規範》鋼筋鋼液之化學成份

種類	符號	C(%)	Mn(%)	P(%)	S(%)	Si(%)	C.E%
光面鋼筋	SR240	—	—	≦0.060	≦0.060	—	—
	SR300	—	—	≦0.060	≦0.060	—	—
竹節鋼筋	SD280	—	—	≦0.060	≦0.060	—	—
	SD420	≦0.32	≦1.80	≦0.050	≦0.050	≦0.50	≦0.570
	SD490	≦0.32	≦1.80	≦0.050	≦0.050	≦0.50	≦0.570
	SD280W	≦0.30	≦1.50	≦0.040	≦0.040	≦0.50	≦0.550
	SD420W	≦0.30	≦1.50	≦0.040	≦0.040	≦0.50	≦0.550

C.E=C+Mn/6+Cu/40+Ni/20+Cr/10-Mo/50-V/10

CNS 鋼筋機械性質

種類	符號	機械性質							
		降伏點或降伏強度 N/mm²	抗拉強度 N/mm²	實際抗拉強度 / 實際降伏強度	試片	伸長率 (%)	彎曲角度		彎曲直徑
竹節鋼筋	SD 280	280～420	420以上	–	2號 14A號	14以上 17以上	180°		標稱直徑之4倍
	SD 280W	280～380	420以上	1.25以上					
	SD 420	420～540	620以上	–	2號 14A號	12以上 13以上	180°	未滿D19	標稱直徑之5倍
								D19以上	標稱直徑之6倍
	SD 420W	420～540	550以上	1.25以上	2號 14A號	12以上 13以上	180°	未滿D19	標稱直徑之3倍
								D19～D25	標稱直徑之4倍
								超過D25	標稱直徑之6倍
	SD 490	490～625	620以上	–	2號 14A號	12以上 13以上	90°	D25以下	標稱直徑之5倍
								超過D25	標稱直徑之6倍

三、未來高強度鋼筋相關規範

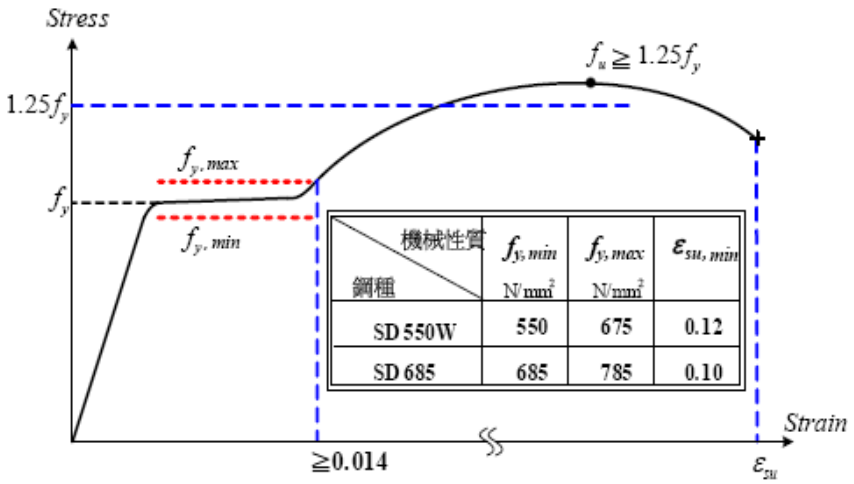
符號	化學成分 %					
	C	Mn	P	S	Si	C.E.(2)
SD 550W	0.33 以下	1.55 以下	0.045 以下	0.045 以下	0.55 以下	0.55 以下
SD 685	0.50 以下	1.80 以下	0.030 以下	0.030 以下	1.50 以下	--
SD 785	0.50 以下	1.80 以下	0.030 以下	0.030 以下	1.50 以下	--

註(2) C.E.（碳當量）% = $\left(C+\frac{Mn}{6}+\frac{Cu}{40}+\frac{Ni}{20}+\frac{Cr}{10}-\frac{Mo}{50}-\frac{V}{10}\right)$ %

符號	機械性質								
	降伏點或降伏強度(3) N/mm²	抗拉強度 N/mm²	實際抗拉強度/實際降伏強度	達降伏強度上限之應變(4)	試片		伸長率 %	彎曲角度	彎曲直徑
SD 550W	550~675	690 以上	1.25 以上	0.014 以上	2 號 14A 號		12 以上	90°	標稱直徑之 4 倍
SD 685	685~785	860 以上	1.25 以上	0.014 以上	2 號 14A 號		10 以上	90°	標稱直徑之 4 倍
SD 785	785 以上	930 以上	—	—	母材	2 號 14A 號	8 以上	180°	標稱直徑之 3 倍
					銲接處(5)	2 號 14A 號	5 以上	—	—

律己 愛智 樂觀

高強度鋼筋續接器續接性能規範



鋼種 \ 機械性質	$f_{y,min}$ N/mm²	$f_{y,max}$ N/mm²	$\varepsilon_{su,min}$
SD 550W	550	675	0.12
SD 685	685	785	0.10

律己 愛智 樂觀

四、強化理論

1. 晶格
2. 強化機構—主要有：

固溶強化—在鋼中添加溶質元素，由於溶質原子與鐵原子大小不同，會造成晶格扭曲，增加差排移動阻礙，而提高強度。

析出硬化—若添加的合金元素，與C、N的結合力很強，會形成細小析出物，造成差排移動困難，而產生析出硬化，從而提高強度。

晶粒細化—是利用減小肥粒鐵晶粒尺寸，提高晶界面積，當發生塑性變形，差排移動遭到晶界的阻礙，而提高發生塑性變形所需的應力，是以強化。

認識鋼鐵

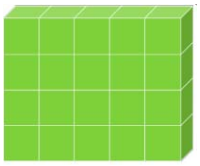
鋼鐵不單只是包含鐵與合金構成的固體。

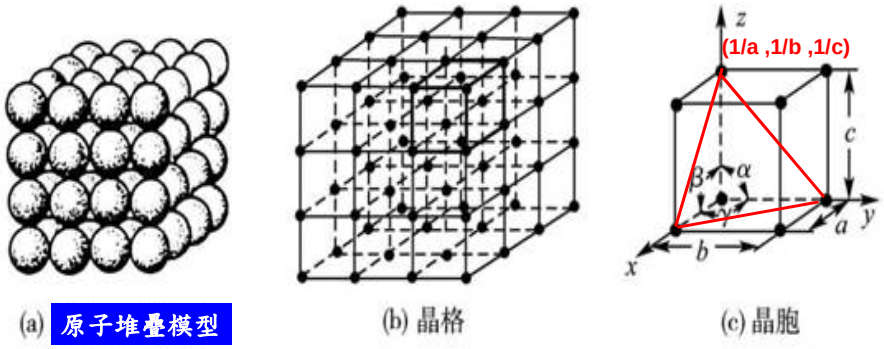
鋼鐵具有分子晶格、明確的結構以及晶粒，就像是鑽石與水晶一樣。

鋼鐵中最常見的分子晶格型式是以鐵原子構成的體心立方。

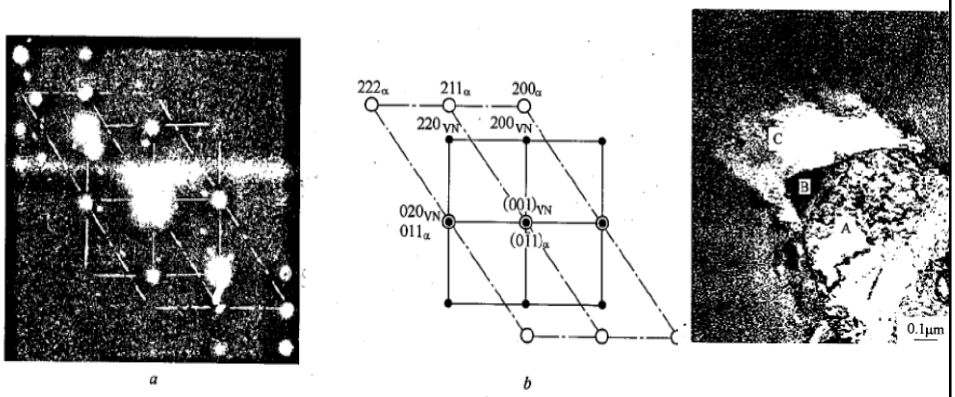
純鐵是完全以鐵原子構成的體心立方所構成的結構，鋼鐵則還帶有一些非純鐵的分子晶格，例如 Pearlite。

分子晶格堆積起來成為鋼鐵的體積。



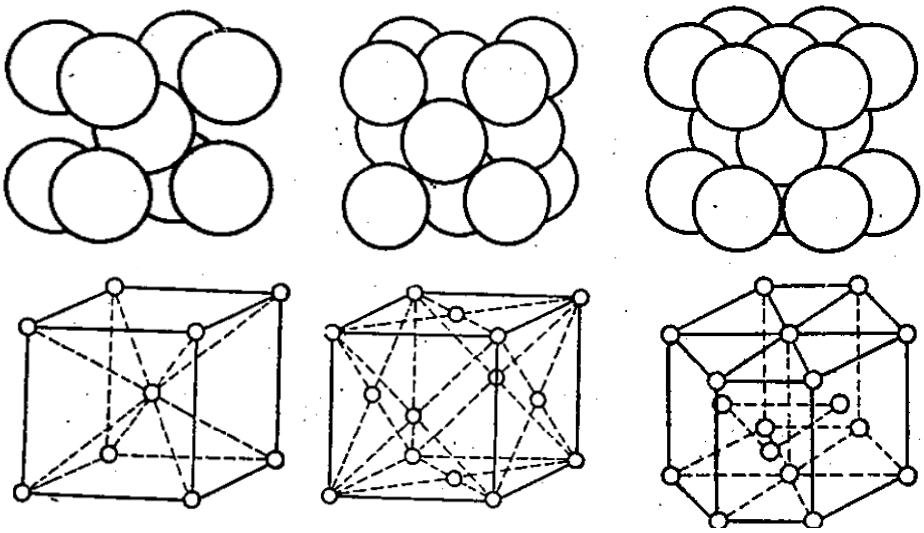


晶體中原子排列示意圖



中碳氮釩鋼淬火
SEM圖

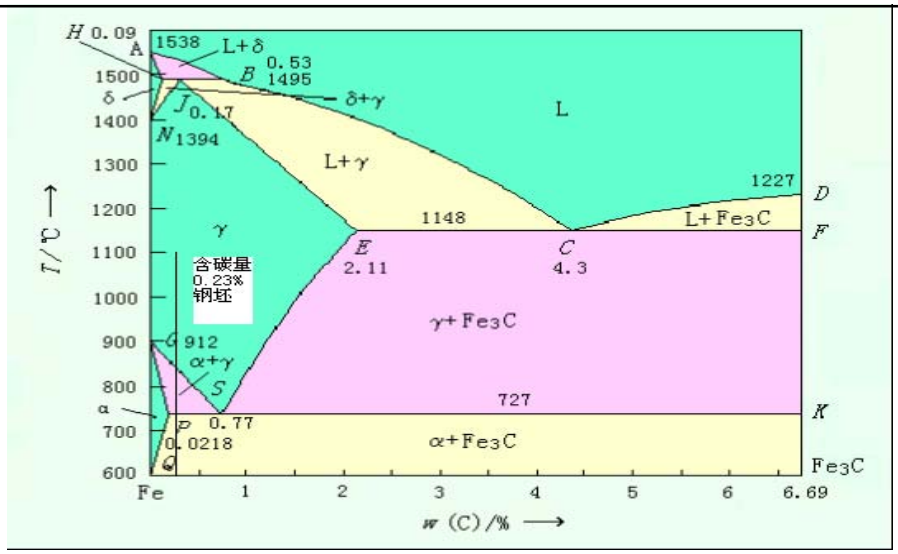
右圖B和C區域 TEM 圖及 標定

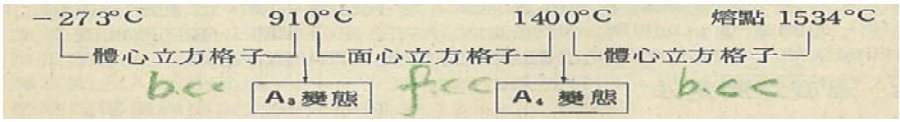


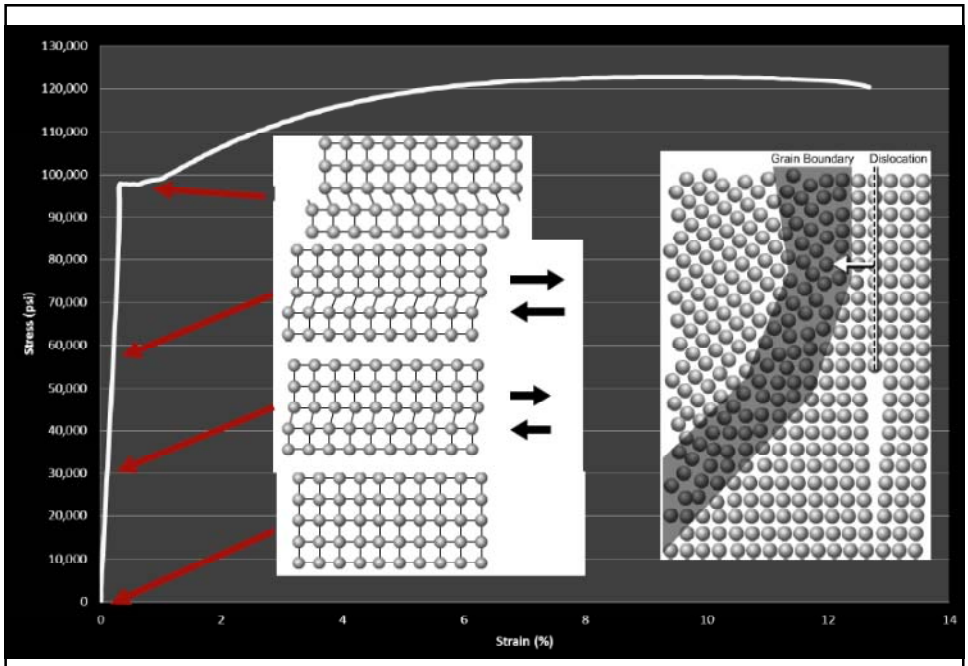
體心立方 BCC
肥粒鐵

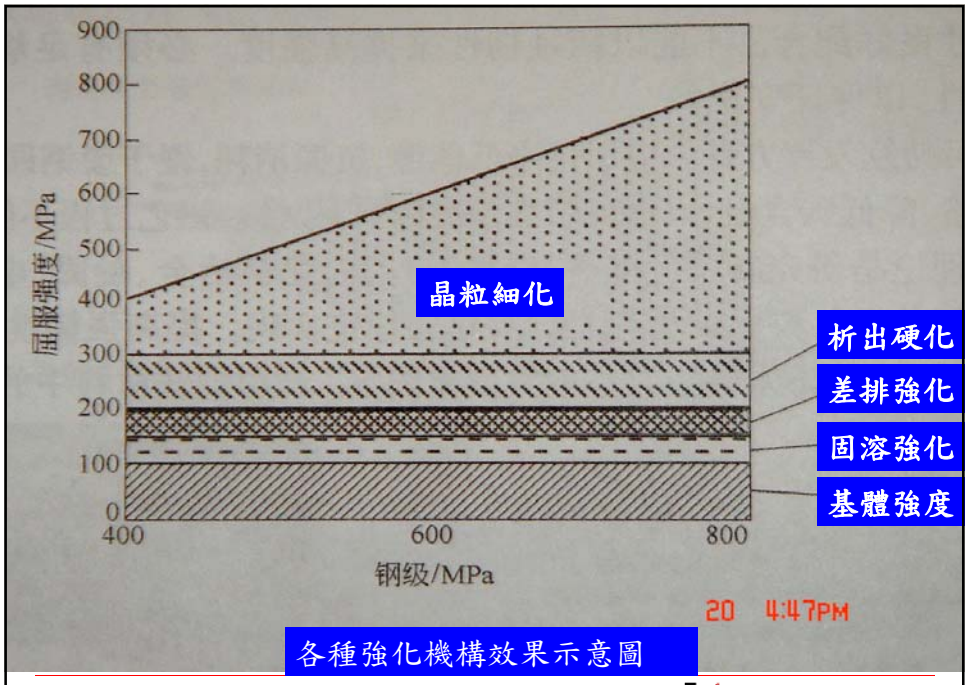
面心立方FCC
沃斯田鐵

六方最密堆積HCP





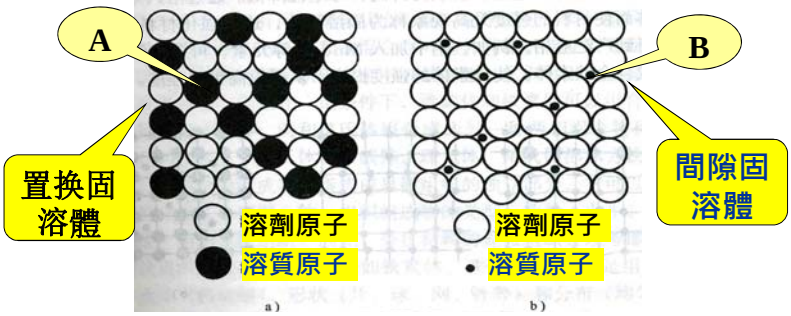




各種強化機構效果示意圖

置換固溶體 ：溶質原子和溶劑原子尺寸相差較小，形成固溶體時溶質原子替換了溶劑晶格中的一部分原子，就形成了置換固溶體。

如：Fe與Mn、Si 、Al 、Cr 、Ti 、Nb等形成置換固溶體

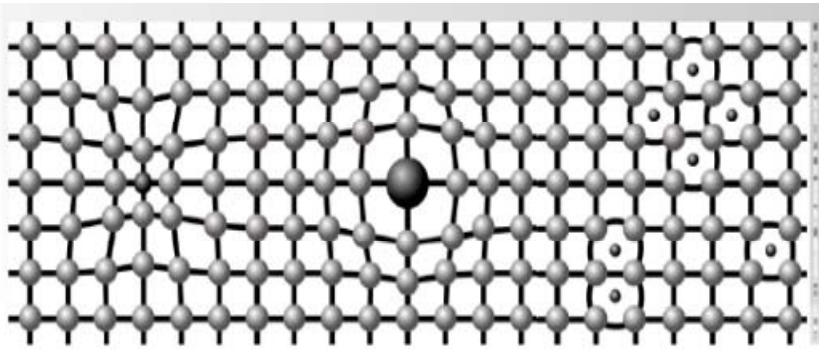


間隙固溶體 ：溶質原子和溶劑原子直徑相差較大，溶質原子處於溶劑晶體結構的間隙位置上，則形成間隙固溶體。

如：Fe 與 C ，N ，O ，H 形成間隙固溶體

固溶強化—Solid Solution Strengthening

置換型固溶強化　　間隙型固溶強化



固溶強化

固溶強化

碳（Carbon）

錳（Manganese）

關於元素對普通碳鋼機械性質的影響，許多經驗式可作一般的參考。其中一經驗式，如下所述。

含碳量在0.20～0.50%的範圍之鋼材，

其抗拉強度（kgf/mm²）＝30.5＋43 ×C＋60 ×C ×Mn

而伸長率※（%）＝17.6＋380／C－0.0022 ×C ×Mn

※ C ＝C% ×100 ， Mn ＝Mn% ×100

微合金化元素

析出硬化

晶狀細化

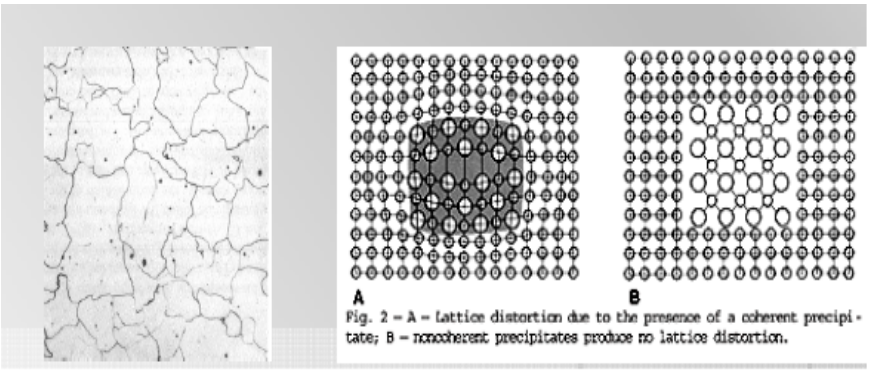


Fig. 2 – A – Lattice distortion due to the presence of a coherent precipitate; B – noncoherent precipitates produce no lattice distortion.

晶粒細化

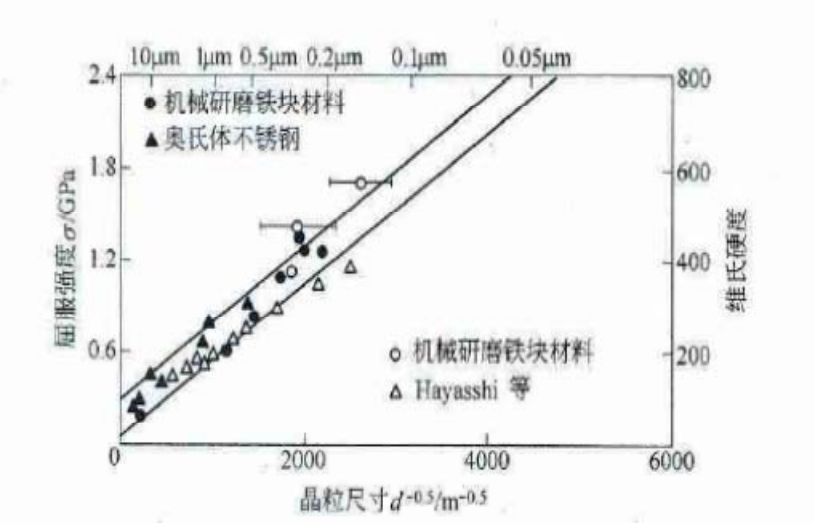
晶粒細化

- 釩（Vanadium）
- 鈮（Columbium）
- 溫度
- 軋延比

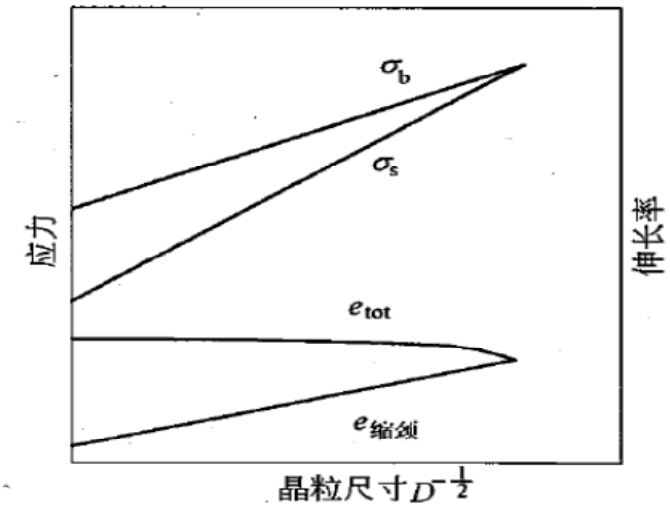
鋼材之晶粒愈細，強度愈高早為人們所習知，Hall及Petch進一步研究晶粒和強度之關係，而發現強度和晶粒之-1/2次方成正比，此即非常著名之Hall-Petch關係式

$$\sigma = \sigma_0 + k \cdot d^{-\frac{1}{2}}$$

此處σ0為鋼材之本質強度，d為晶粒大小（單位為mm），K為常數。一般常用之晶粒細化手段是利用添鈮、釩、鈦等元素進行來完成。



不同晶粒度與降伏強度的關係



晶粒尺寸對力學性能之影響

析出強化

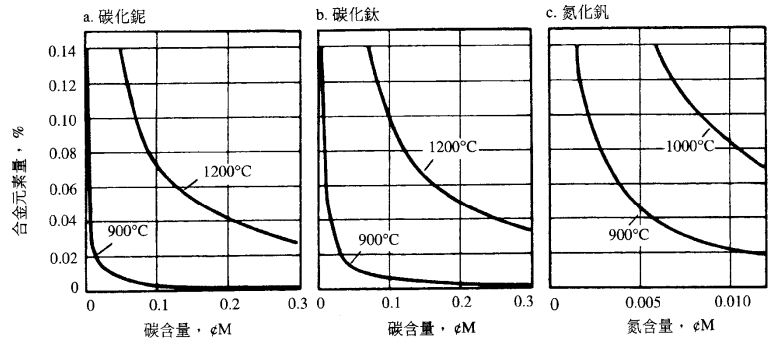
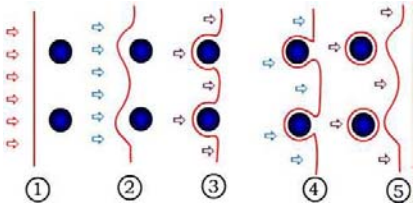


圖 7　碳化鈮(b)碳化鈦及(c)氮化釩在高溫之固溶狀態

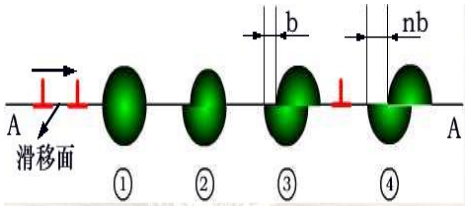
析出強化（第二相硬質點強化）

析出強化的機制是差排和顆粒之間的相互作用。

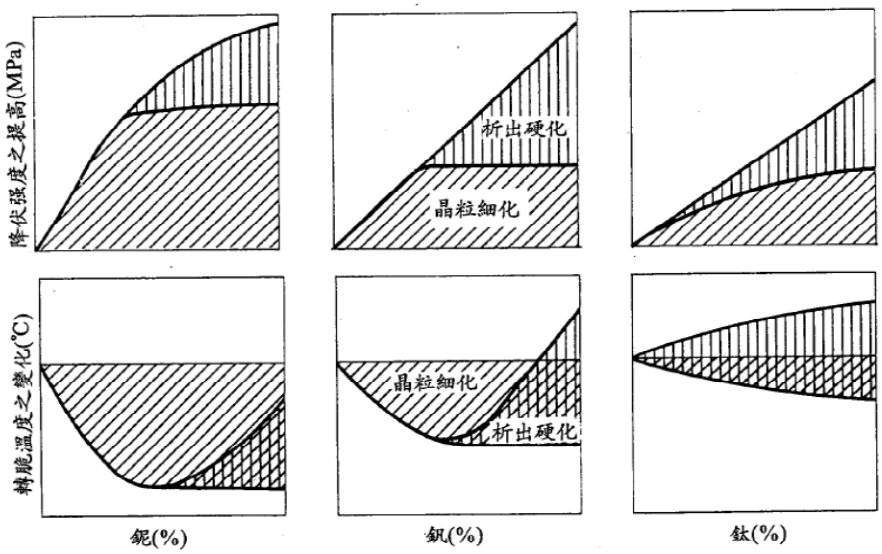
在外力作用下，運動差排遇到第二相硬質點時的運動方式有兩種，(1)對提高強度有積極作用的繞過過程； (2)對提高強度作用較小的切割/剪切過程。它們都會增加運動阻力，可以提高材料的強度。



繞過機制



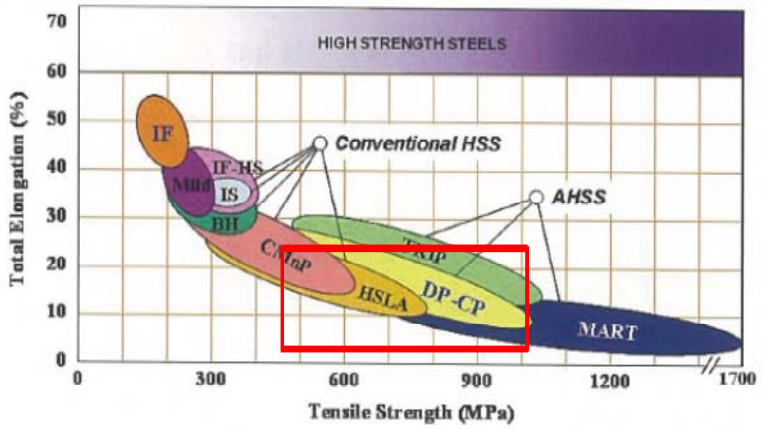
切割機制

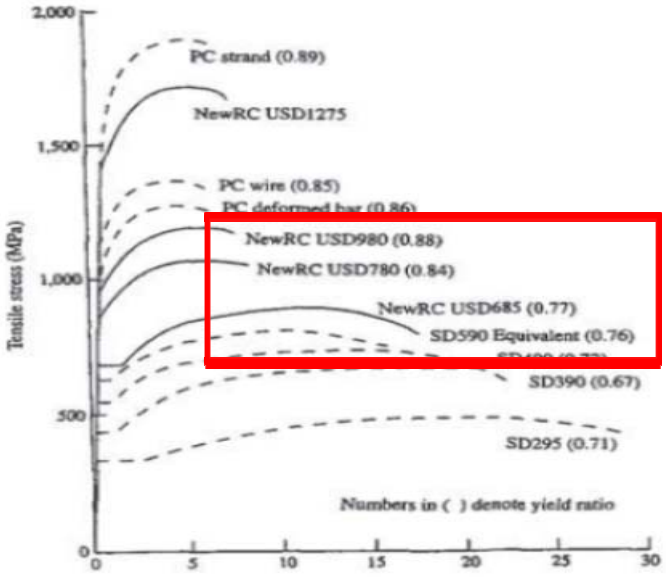


不同合金所造成之强化效果及對韌性之影響

五、490提升到→685→785

高強度鋼可分為傳統高強度鋼及先進高強度鋼(圖10)，介紹如下：





Stress-strain relationships of steel with different strength.

五、490提升到→685→785

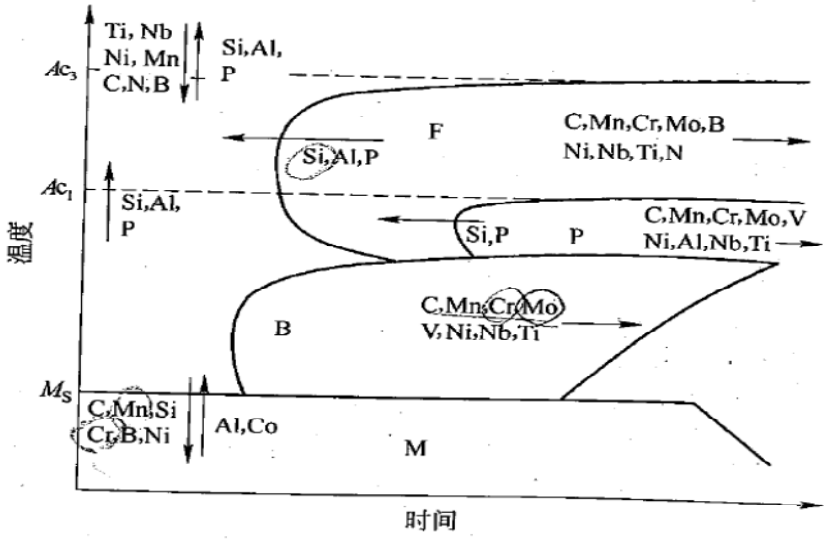
1. 材料技術-合金化設計及微合金化
2. 設備發展-設備改造/自動控制系統
3. 操作技術-
   - 控軋控冷
   - 低溫軋製
   - 超快冷技術

表 1 控轧控冷技术专利优先权国统计 项

日本	中国	世界知识产权组织	韩国	美国	欧洲专利局	德国	俄罗斯	加拿大	印度	英国	法国	芬兰	意大利	苏联
3 881	2 432	843	697	304	131	129	44	26	23	22	20	15	14	13

区分			サイズ	主な化学成分 wt%								機械的性質					
				C	Si	Mn	P	S	$C+\frac{Mn}{6}$	V	Nb	降伏点 N/mm²	引張強度 N/mm²	伸び %	降伏棚の長さ	降伏比 %	曲げ性能
開発鋼	S-SD 685	実績	D51	0.30	0.99	1.61	0.018	0.015	0.57	0.31	0.04	708	905	16	3.4	78	GOOD
		規格		0.38 以下	1.50 以下	1.80 以下	0.030 以下	0.030 以下	0.68 以下	—	—	685～755	860 以上	10 以上	3.0 以上	80 以下	公称直径の 2 倍 曲げ角度 90 度
比較鋼	JIS G3112 SD490	実績	D51	0.26	0.46	1.41	0.022	0.018	0.50	0.15	—	509	702	13	—	—	GOOD
		規格		0.32 以下	0.55 以下	1.80 以下	0.040 以下	0.040 以下	0.60 以下	—	—	490～625	620 以上	12 以上	—	—	公称直径の 2 倍 曲げ角度 90 度

[6] 神戸製鋼技報 **Vol. 48 No. 1 (1998）p.85**



合金元素對CCT曲線之影響

五、490提升到→685→785

1. 材料技術-合金化設計及微合金化

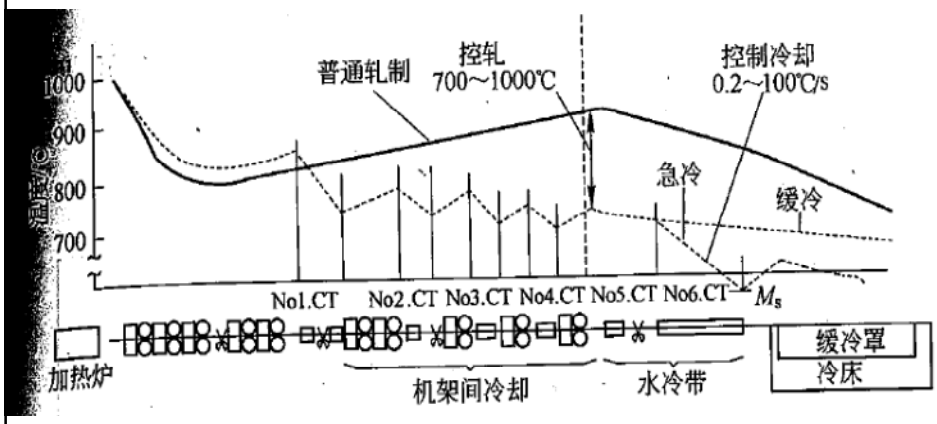
2. 設備發展-設備改造/自動控制系統

3. 操作技術-

控軋控冷

低溫軋製

超快冷技術



軋製棒材對溫度冷卻之參數

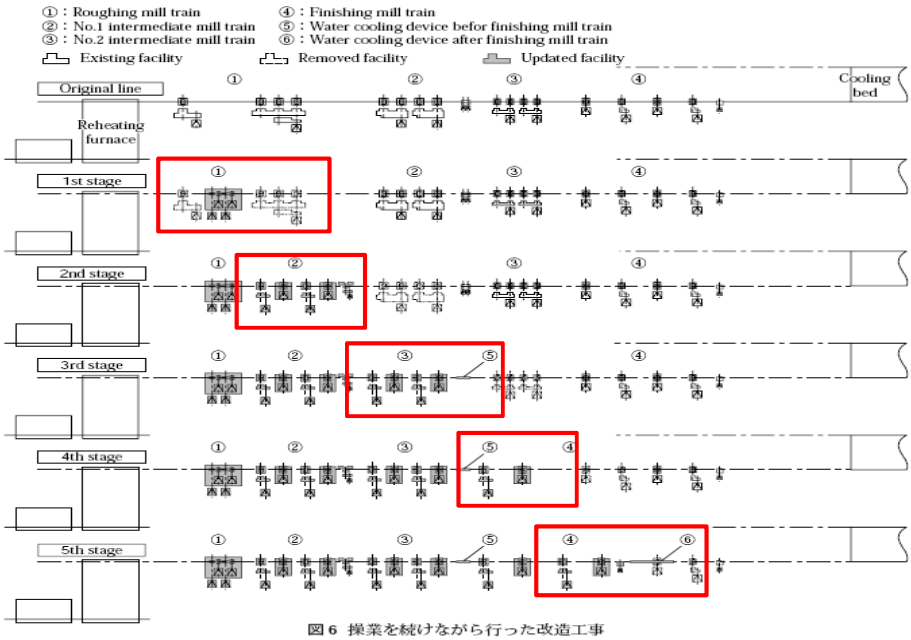
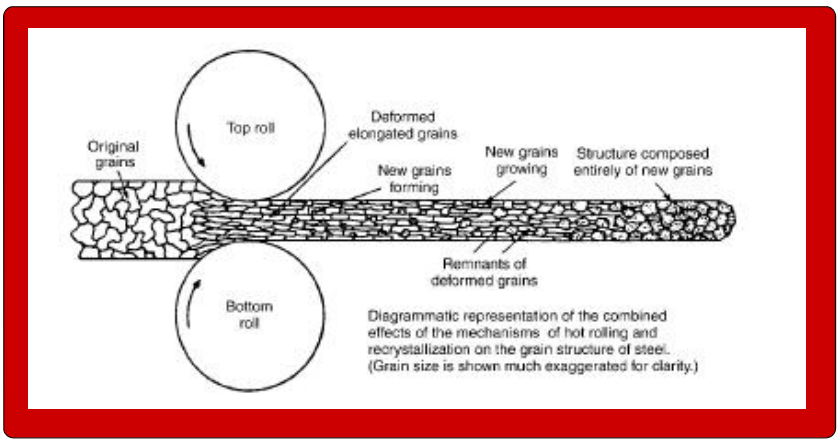
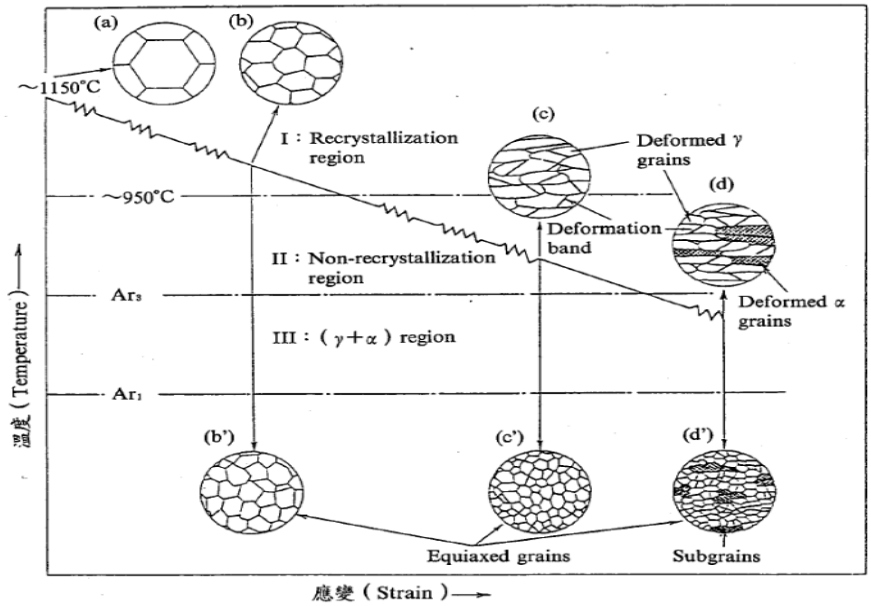


図 6 操業を続けながら行った改造工事

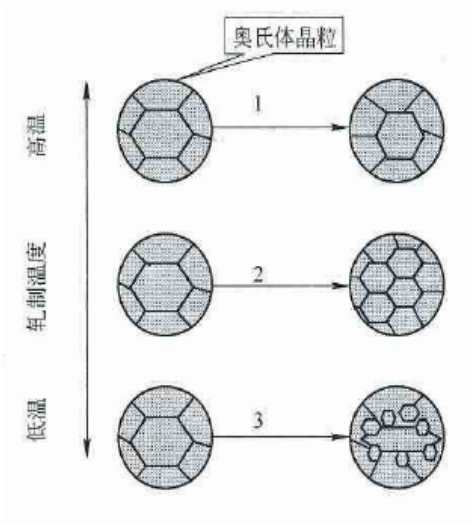
Fig. 6 Schematic drawings of revamping construction without stopping operation

晶粒細化（機械加工）





控制軋延的軋製示意圖



高溫軋製

再結晶軋製

部分再結晶軋製

再結晶軋製示意圖

a.原始沃斯田鐵組織

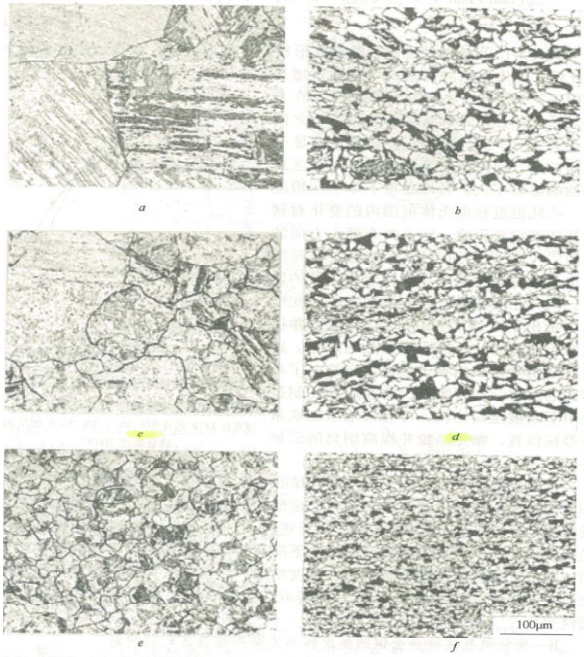
b.變形60%

c.部分再結晶沃斯田鐵

d.變形60%

e. 再結晶沃斯田鐵組織

f.變形60%



含鈮微合金鋼沃斯田鐵和軋製後的相變組織

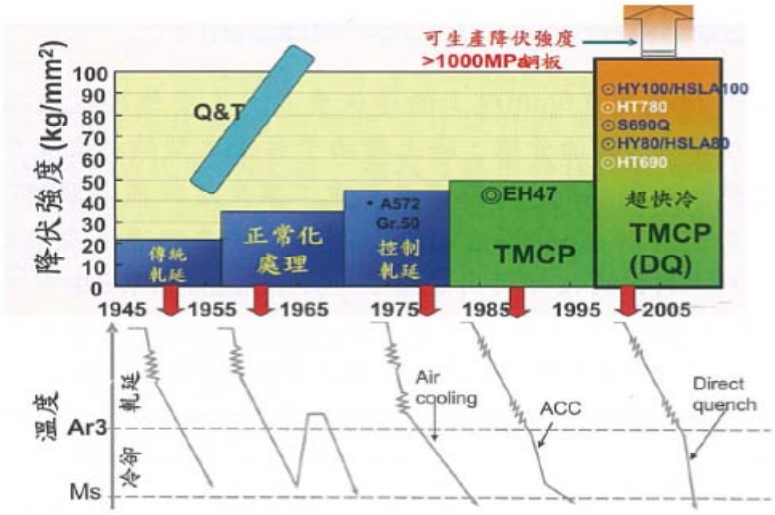
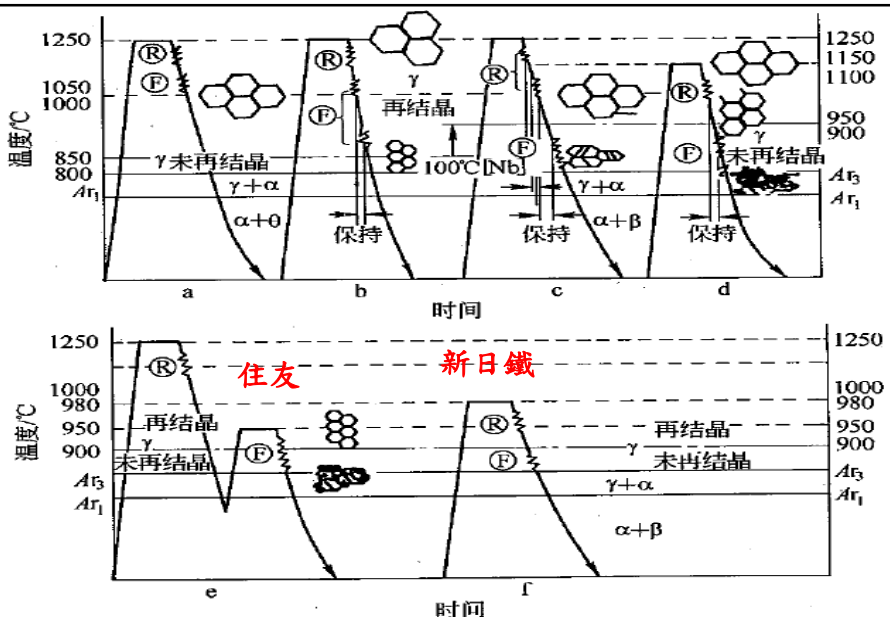
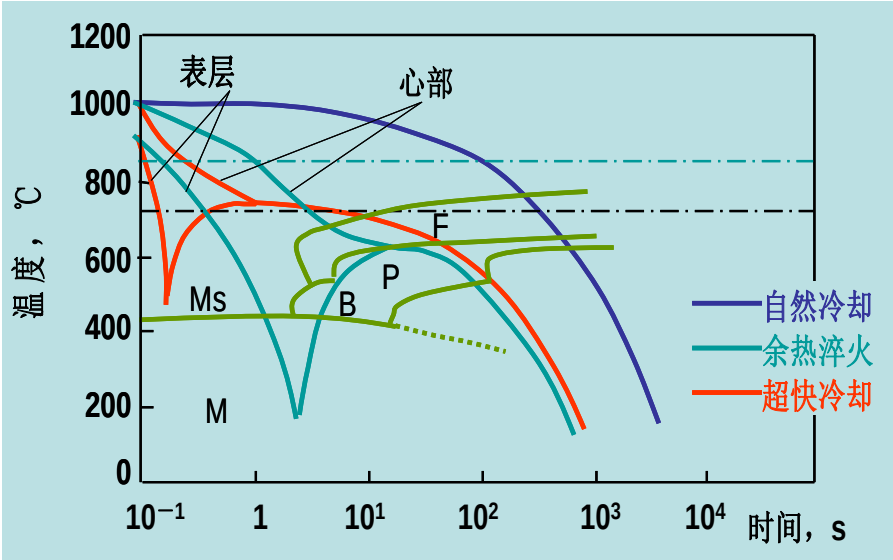


圖2　鋼板生產技術發展示意圖



控制軋製的發展



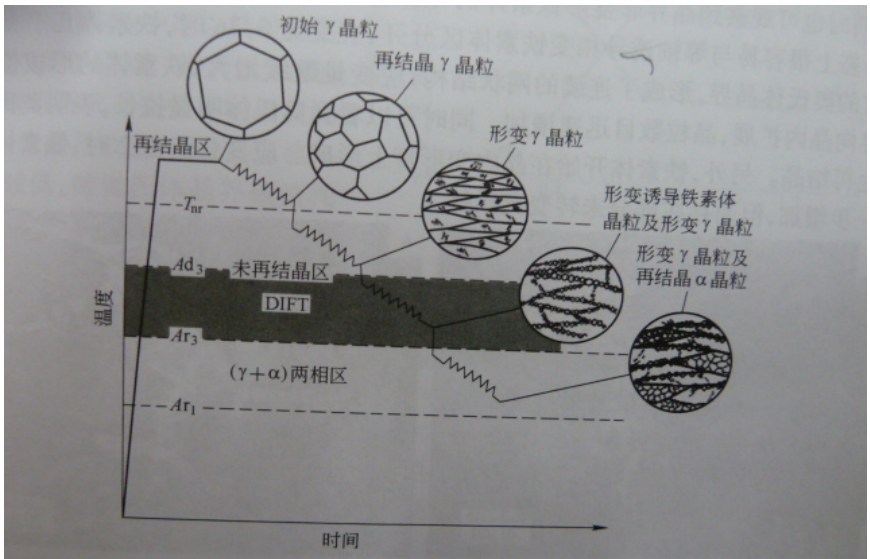
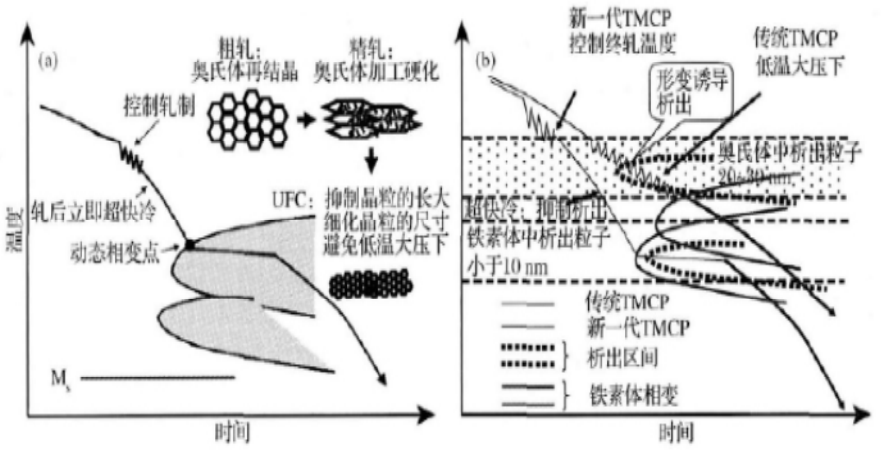


图 1-3-1 形变诱导铁素体相变与传统 TMCP 的关系

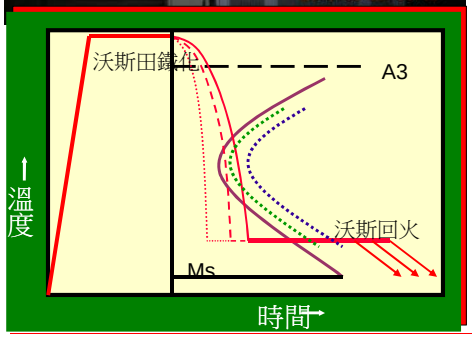


(a) 新一代 TMCP 细晶强化机制; (b) 新一代 TMCP 析出强化机制。

图 5 新一代 TMCP 技术与传统 TMCP 技术强化机制的区别

Fig. 5 Distinctions of strengthening between NG-TMCP and conventional TMCP

六.測試與研究—續接器之 ADI 熱處理



步驟:

1.沃斯田鐵化(850~950˚C)

2.急冷

3.恒溫變態(Ms點以上)

4.冷卻至常溫

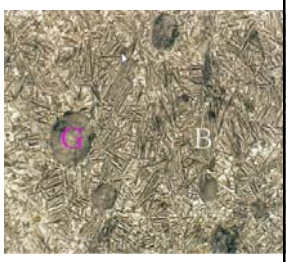
next

ADI熱處理設備

加熱爐

鹽浴爐

ADI金相



球墨鑄鐵 沃斯回火 變韌肥粒鐵+殘留沃斯田鐵

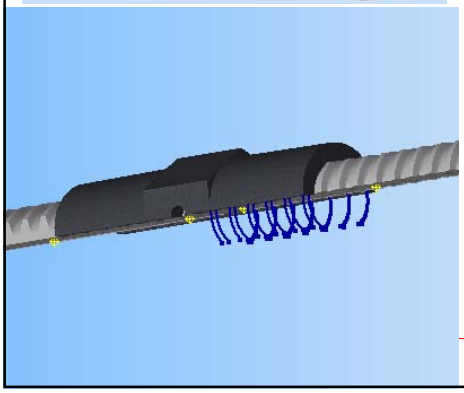
波來鐵+肥粒鐵 → （沃斯肥粒鐵）

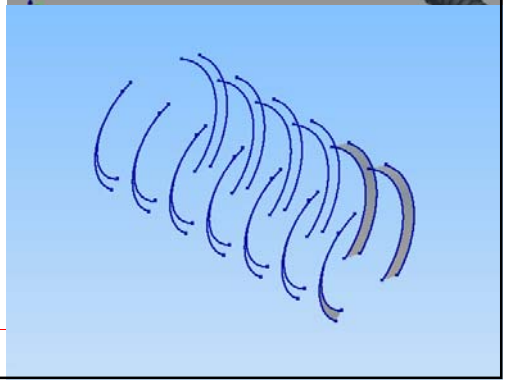
沃斯回火球墨鑄鐵螺旋型鋼筋續接器

專利證書號157047

球墨鑄鐵晶沃斯回火後基地上的白色小隙縫:柔韌的沃斯田鐵. 此相提供ADI高韌性,吸震性.工件表皮受磨擦或撞擊時轉變成HRC60的麻田散鐵.其相變化機制與高錳鋼同,石墨內可蓄積油膜,很適合需高韌性, 耐磨耗性,又需吸震的地方.例:齒輪 刀把.... 應用性很廣.

◆尺寸鋼筋所需鎖接長度規定-範例D25

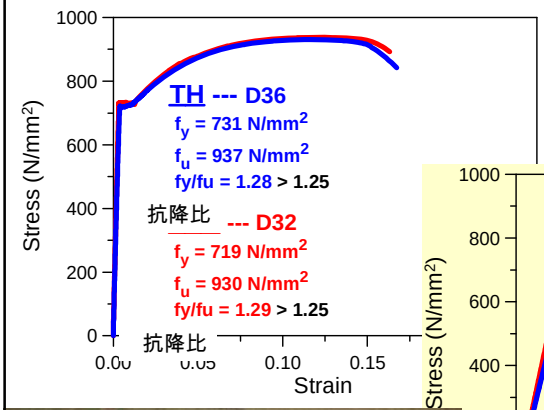




◆尺寸鋼筋所需鎖接長度規定-範例D25

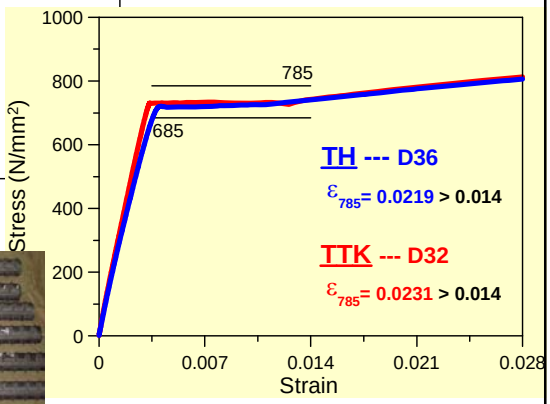
- 螺紋鋼筋與續接器組裝3D面積分析
- 單邊需4牙
- 設計安全係數1.5
- 螺紋鋼筋鎖入續接器長度設計6牙
- 其它尺寸同此模式設計

六、測試與研究--高強度鋼筋拉伸試驗



TH D36 符合

SD 685鋼筋規格



東和鋼鐵 SD 685 D43 鋼筋拉伸試驗結果

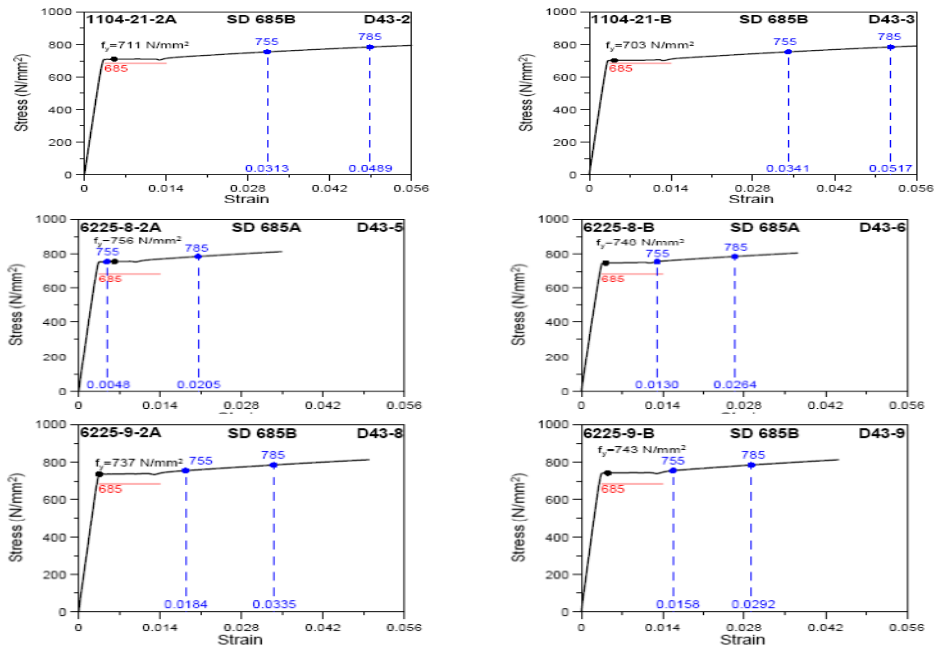
台灣世曦楊梅材料實驗室　　試驗日期：2009/9/4

試樣編號	鋼筋編號	鋼筋種類	降伏強度 N/mm²	抗拉強度 N/mm²	抗降比	彈性模數 N/mm²	ε_{755}	ε_{785}	伸長率 %	單位重 %
							>0.014			
D43-1	1104-21-1A	SD 685 V+Nb	697	896	1.29	—			16	11.29
D43-2	1104-21-2A	SD 685	711	903	1.27	234,981	0.0313	0.0489	16	11.36
D43-3	1104-21-B	SD 685	703	902	1.28	233,216	0.0341	0.0517	15	11.42
D43-4	6225-8-1A	SD 685 V	729	949	1.30	—	—	—	14	11.30
D43-5	6225-8-2A	SD 685	756	950	1.26	235,131	0.0048	0.0205	13	11.26
D43-6	6225-8-B	SD 685	748	944	1.26	223,075	0.0130	0.0264	波斷在標點外	11.29
D43-7	6225-9-1A	SD 685 V	705	929	1.32	—	—	—	15	11.37
D43-8	6225-9-2A	SD 685	737	933	1.27	224,657	0.0184	0.0335	13	11.33
D43-9	6225-9-B	SD 685	743	941	1.27	224,657	0.0158	0.0292	15	11.47

註：D43-1、D43-4、D43-7 未架延伸計。

律己 愛智 樂觀

東和鋼鐵企業股份有限公司 TUNG HO STEEL ENTERPRISE CORPORATION



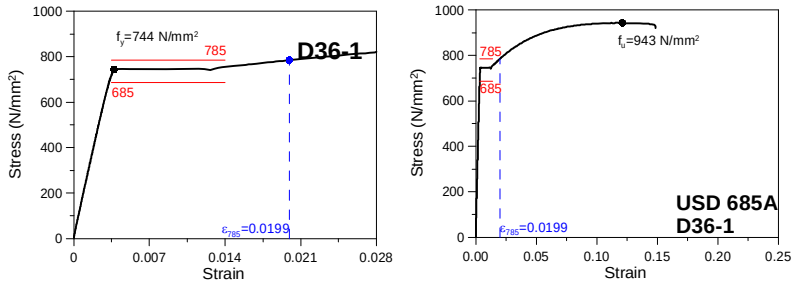
律己 愛智 樂觀

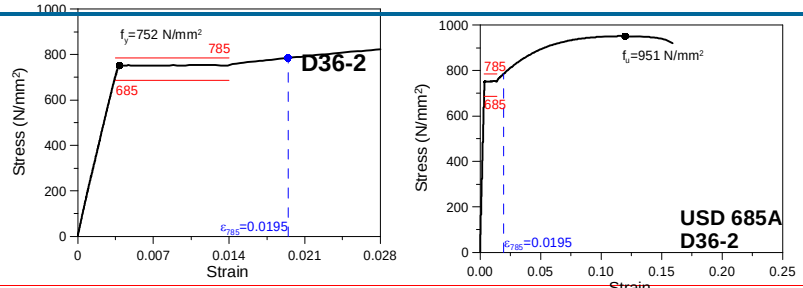
東和鋼鐵企業股份有限公司 TUNG HO STEEL ENTERPRISE CORPORATION

D36鋼筋試片試驗結果

編號	降伏強度 N/mm²	抗拉強度 N/mm²	抗降比	ε 785	破壞應變
	685~785	808以上	1.18以上	0.014以上	0.10以上
D36-1_1104-4-1-A	744	943	1.27	0.0199	0.149
D36-2_1104-4-1-B	752	951	1.26	0.0195	0.159
D36-3_1104-4-1-B	731	937	1.28	0.0219	0.163
D36-4_1104-3-A	723	912	1.26	0.0272	0.116
D36-5_1104-3-B	724	915	1.26	0.0271	0.127
D36-6_1104-3-B	723	912	1.26	0.0270	0.161

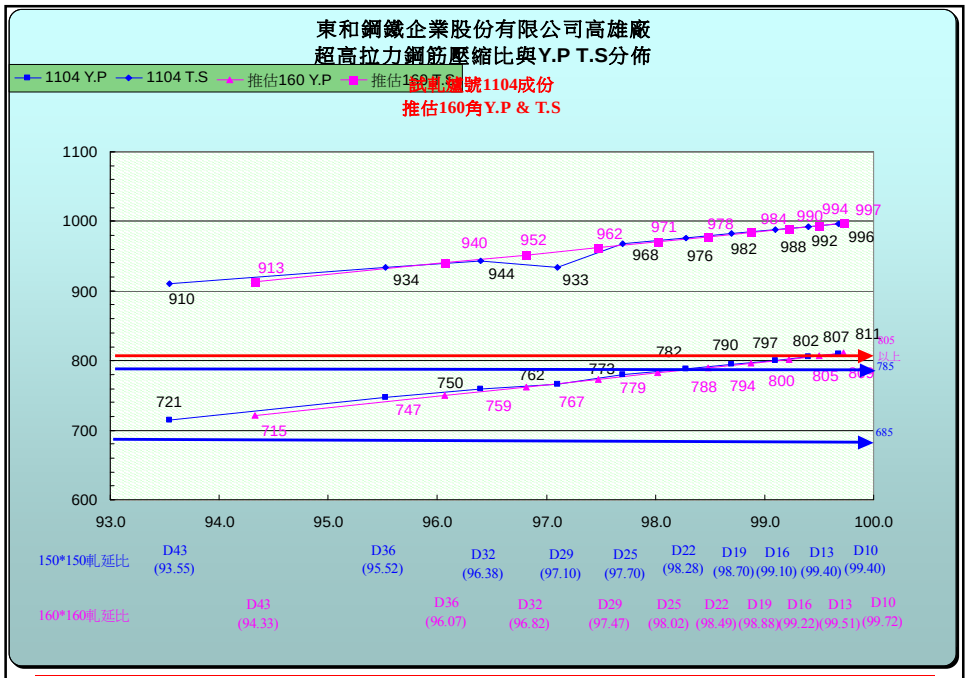
D36-1_1104-4-1-A
D36-2_1104-4-1-B





D16鋼筋試片試驗結果

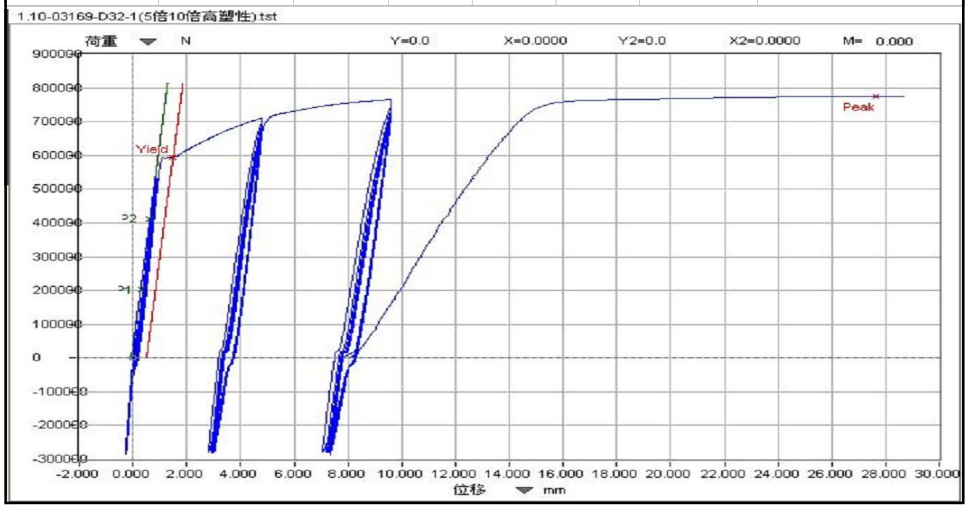
編號	降伏強度 N/mm²	抗拉強度 N/mm²	破壞應變
	785以上	930以上	0.80
D16-1_1101-1-2-1	778*	968	0.141
D16-2_1101-1-2-2	819	1003	0.116
D16-3_1101-1-2-2	846	1030	0.0773*
D16-4_1101-5-2-1	790	963	0.122
D16-5_1101-5-2-3	777*	951	0.097
D16-6_1101-5-2-3	792	970	0.125



與續接器接合測試數據(SD685)

※ SD685 D32高塑性反覆載重試驗數據(土木401-86)

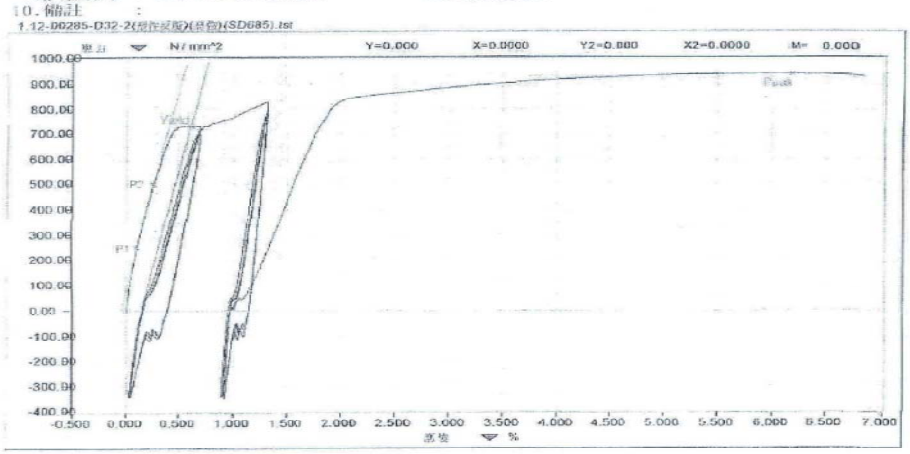
試體標號	續接器本體長度(mm)	標點長度(mm)	滑移量(mm)			抗拉強度 N/mm2	備註
			16c	24c 5εy	32c 10εy		
10-03169-D32-1	240	340	0.194	0.323	0.438	953	D43 車製D32螺紋

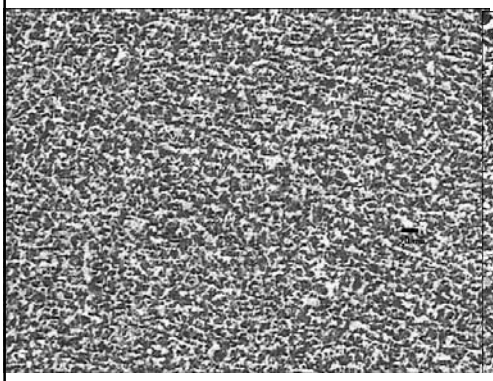


螺紋鋼筋與續接器接合測試 (SD685)

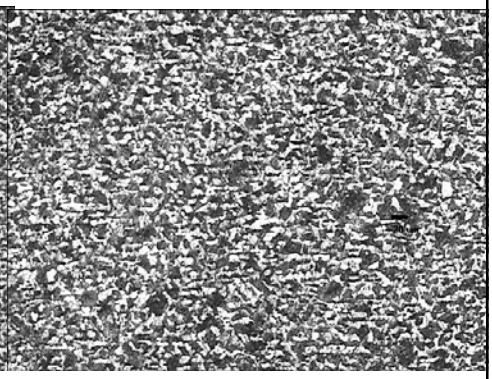
#10 經螺紋

1.製造廠商 :
2.操作者 :
3.試驗批號 : 12-00285-D32-2(塑性反覆)(高強)(SD685)
4.試驗日期 : 2012年3月9日
5.試驗時間 : 上午 10:17:53
6.試驗溫度 : 25C
7.試驗速度 : 20.0 N / mm^2・S then -2.0 %/min and 2.0 %/min
8.試驗型態 : 高塑性反復載重試驗
9.試驗標準 : 土木401-86 附錄乙
10.備註 :
11.報告編號 : 12-00285-D32-2(塑性反覆)(高強)(SD685)
12.續接器形式 : 套筒
13.續接性能等級 :
14.鋼筋規格 : SD685
15.續接器本體長度 : 238mm
16.測試件數 : 1 pcs
17.母材標稱降伏強度Fyn : 685.00N / mm^2
18.母材實際降伏應變εy : 0.343%
19.母材實際彈性模數E : 200000N / mm^2
20.規範要求 :





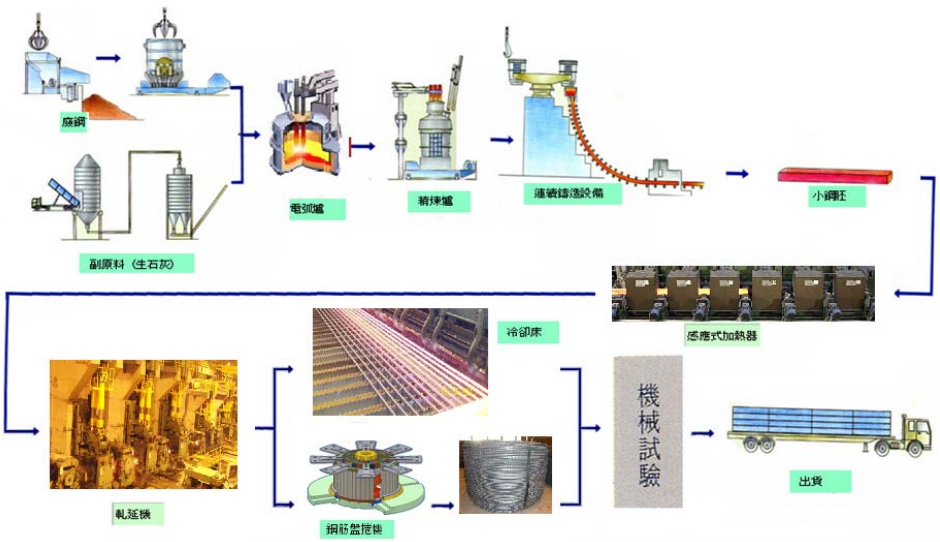
SD685-(D32)1104-6表面100倍 N=9~10



一般(D32)SD420W表面100倍 N=5~6

七、目前具體作法
桃園廠新設備--最先進之製程

桃園廠生產流程簡介(含VCC)



傳統單軋廠製程:

冷鋼胚 → 加熱爐 > 軋鋼 > 鋼筋

傳統煉軋鋼廠製程:

煉鋼 > 鋼胚 → 加熱爐 > 軋鋼 > 鋼筋

熱進軋延

東鋼直接軋延新製程:

煉鋼 > 鋼胚 → 軋鋼 > 鋼筋

直接軋延

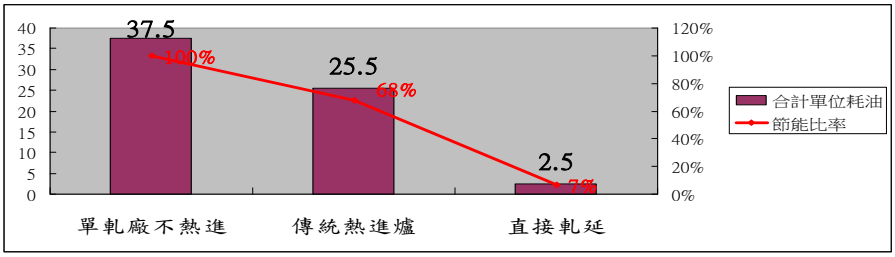
優點:

- **沒有設置加熱爐**避免硫氧化物與氮氧化物產生。
- **大幅降低能源耗用**

  一年可節省71,000 噸的CO2排放量，

  相當於190座大安森林公園一年吸收之CO2量。

項目	單位	單軋廠不熱進	傳統熱進爐	直接軋延
鋼胚進爐溫度	℃	30	400~600	900~1000
鋼胚開軋溫度	℃	1200	1200	1200
燃油耗用	L/ton	37.5	25.5	0
電熱相當油耗	L/ton	0	0	2.5
合計單位耗油	L/ton	37.5	25.5	2.5
燃油耗用比率	%	100%	68%	7%



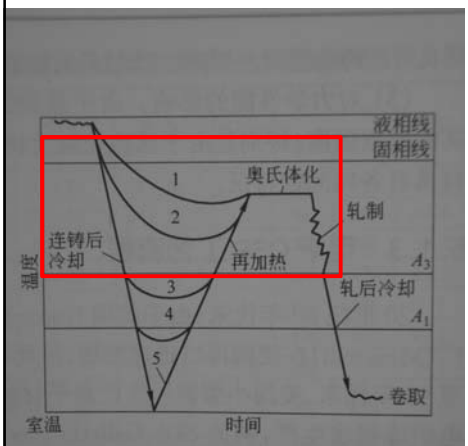


图 1-5-1　连铸连轧方式示意图

1—连铸-直接轧制方式;2—连铸-直接热装炉方式;3—普通热装炉方式;4—最易实现的一种热装炉方式;5—普通工艺方法

(1) 连铸坯直接轧制,简称 CC - DR,分类为 I型。连铸坯在 1100 ℃条件下不经加热炉,在输送过程中通过边角补热装置直接送入轧机轧制。铸坯轧前未经过忙 $\gamma \rightarrow \alpha \rightarrow \gamma$ 相变再结晶过程,仍保留铸态粗大的奥氏体晶粒,微量元素 Nb、V 等无常规冷装炉的析出、再溶解过程,因而需开发新的轧制工艺来得到晶粒细化的组织,这对微合金化钢来说,更能充分发挥 Nb 等微合金化元素的作用。

(2) 连铸坯热直接轧制,简称 CC - HDR,分类为 II型。连铸坯温度在 1100 ℃以下,A_3 以上,铸坯不经加热炉,在输送过程中通过补热和均热,使钢坯达到可轧温度,直接送入轧机轧制。铸坯的金属学特征基本与 I 型相同,仅一些微量元素有少量析出和再溶解,相应的轧制工艺与 I型相似。

八、未來努力方向

1. 提升鋼液清淨度
2. 控軋控冷的改善
3. 軋後緩冷—堆疊保溫
4. 節型的設計改善

九、結論

- 敬請指教

15000psi 混凝土的產製與施工管理

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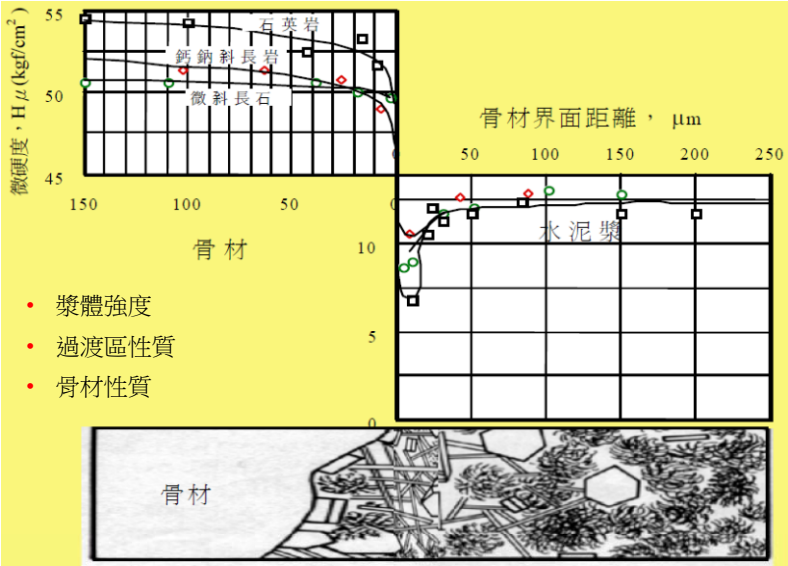
2014.12.05

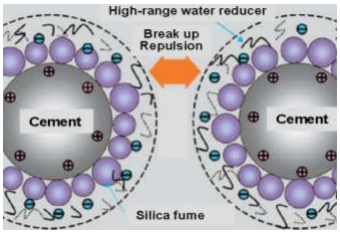
生產高強度混凝土控制因素



- 漿體強度
- 過渡區性質
- 骨材性質

漿體強度

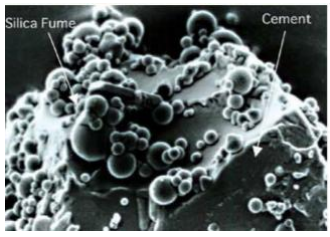
- **使用新型高性能減水劑**
  - 由於超高強度混凝土中的用水量較一般普通混凝土降低許多，爲了維持適當的工作性，此類新型的超高效減水劑具有良好的流動性、高減水率（減水率大於30%）、坍保效果佳。目前以羧酸系與氨基磺酸鹽系兩大類減水劑為主。



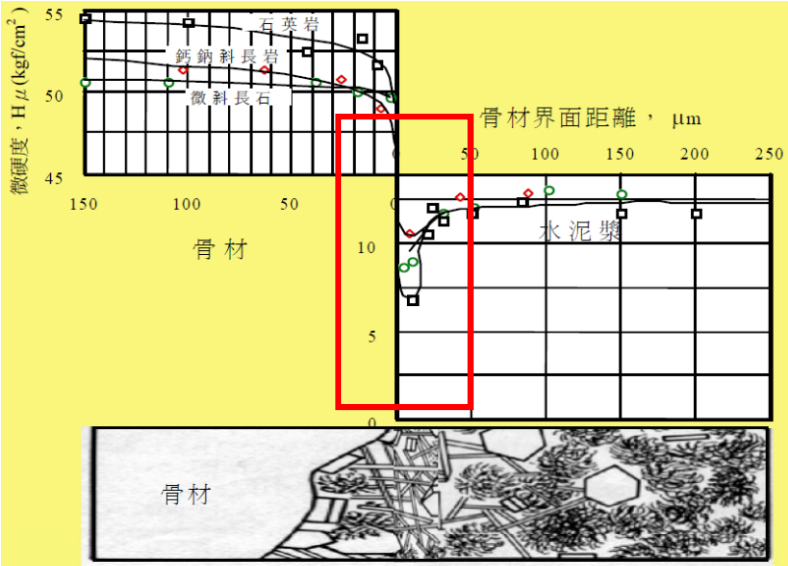
- **降低水膠比**
  - 爲了盡量減少混凝土中孔徑50nm的數量，水膠比將控制在0.38以下，當製產10000psi混凝土時，水膠比更需要控制在0.3以下。

漿體強度

- **使用超細礦物摻料**
  - 目前超細礦物摻料的比表面積可從6000cm2/g到15000cm2/g以上，有矽灰、120級以上超細爐石粉、超細粉煤灰、超細沸石粉、超細稻殼灰、超細石灰石粉等。此類礦物摻料因為具有卜作嵐活性反應，顆粒粒徑小，具有極大的比表面積，故能填充混凝土中細微的空隙，並快速增加C-H-S膠體數量，降低混凝土中孔隙率，增加緻密度。

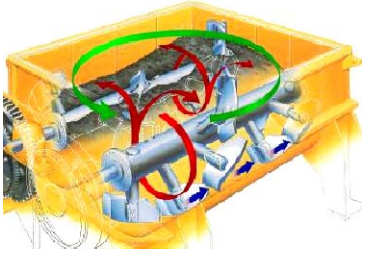


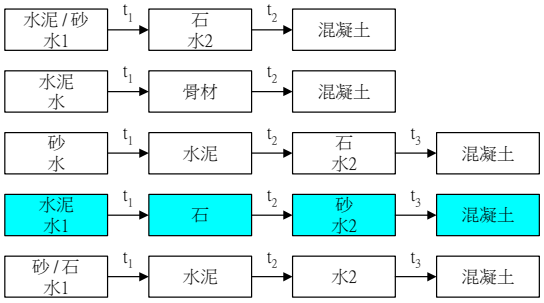
過渡區性質的強化



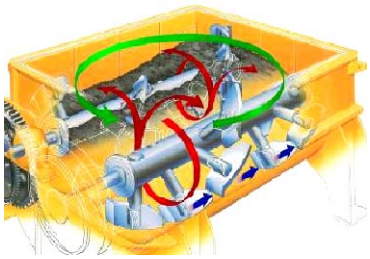
過渡區性質的強化

- 當攪拌機可轉速調節時，宜採用先拌砂漿法。該作法攪拌40s的混凝土強度比傳統攪拌40s、50s、68s時分別提高24%、16%、4%，且攪拌速度可根據負荷變化調節。
- 當設備採用複式攪拌機時，宜採用水泥裹石法，該技術攪拌50s的混凝土強度比傳統攪拌50s 、68s時分別提高16.8%和4.4%，且由於上下兩台攪拌機可同時攪拌，部分攪拌時間的重疊使攪拌週期縮短，效率提高。
- 當臥式攪拌機為機械傳動時，宜用單速兩次投料攪拌。該技術攪拌30s的混凝土強度比雙速二次攪拌40s和傳統攪拌40s時分別提高4.4%、29.6%，且設備結構簡單。
- 二次攪拌製程的混凝土介面過渡區的粘結強度得到了改善。





要求拌合機拌合參數



實際生產的案例

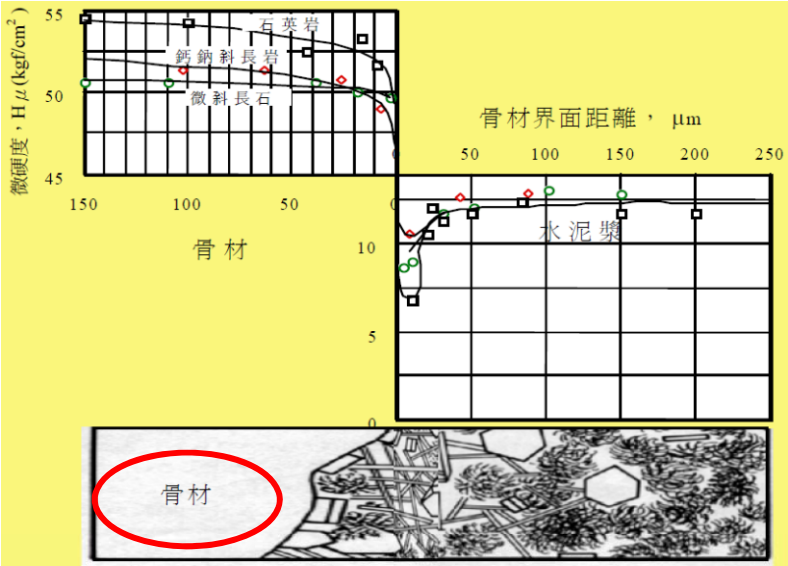
測試編號	水泥 (kg/m3)	爐石 (kg/m3)	飛灰 (kg/m3)	矽灰 (kg/m3)	粗骨材 (kg/m3)	細骨材 (kg/m3)	用水量 (kg/m3)	強塑劑 (kg/m3)	膨脹劑 (kg/m3)
12K-20	400	200		50	851	692	165	14.3	20
12K-15	400	200		50	851	692	165	14.3	15
12K-10	400	200		50	851	692	165	14.3	10
12K-5	400	200		50	851	692	165	14.3	5
12K-0	400	200		50	851	692	165	14.3	
10k-20	400	170		50	851	720	165	13.6	20
10k-0	400	170		50	851	720	165	13.6	
8k-20	350	140	70		838	777	165	8.96	20
8k-0	350	140	70		838	777	165	8.96	

與一般SCC生產的差異

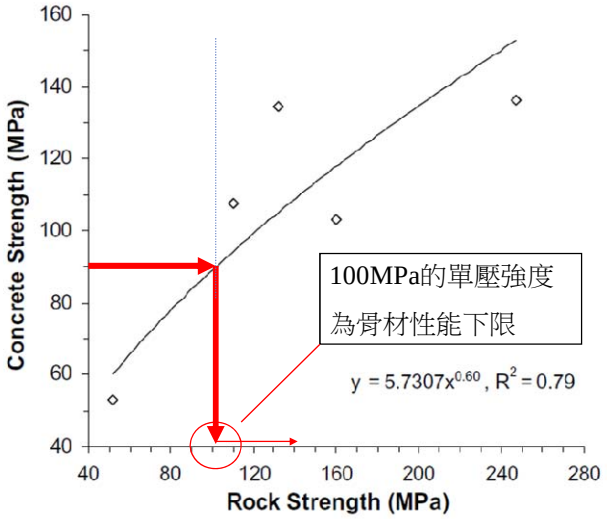
1. 拌合時間220~240秒（較一般SCC長60秒）
2. 穩定安培值為34安培（較一般SCC高2A）
3. 每小時可供應混凝土量24m3（較一般SCC低約10m3）
4. 量產無虞。

組別	時間	混凝土溫度(℃)	流度（cm）	空氣含量（%）	U型充填（cm）	V漏斗時間（sec）
1	08:51	25	65	2.6	33.5	15
2	11:19	27	65	2.5	33.5	16.8
3	13:04	28	60.5	2.1	33	18.4
4	14:41	28.5	64.5	2.4	33.5	16.2
5	15:58	28	66	2.6	33.5	17.7
6	17:13	26	65	2.7	33	14.4

骨材性質要求



骨材強度與混凝土抗壓強度關係圖

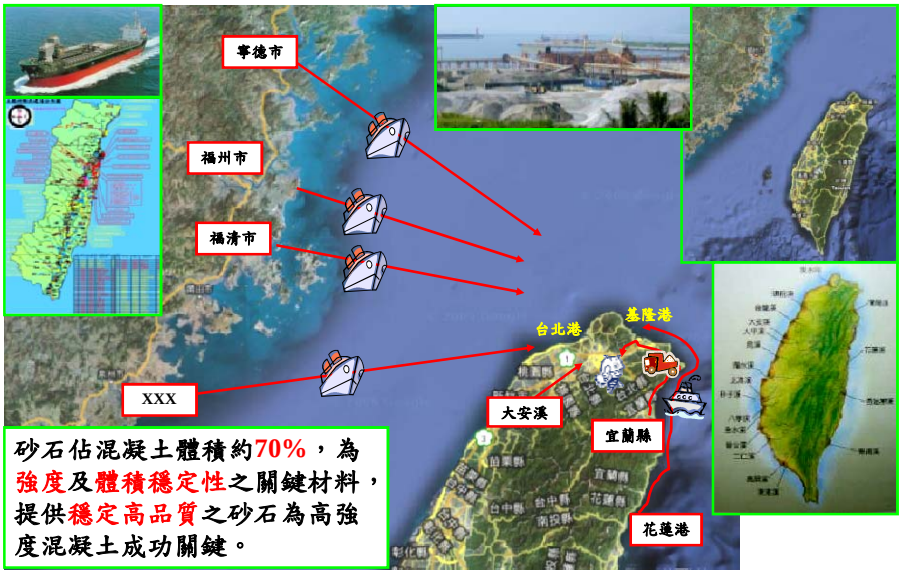


資料來源：A. Kilic, C.D. Atus, A. Teymen, O.Karahan, F.Ozacn,C.Bilim,M.Ozdemir,"The influence of aggregate type on the strength concrete",Cement and Concrete Research",P290-296,2008.

硬質砂岩的性能可以合乎需求

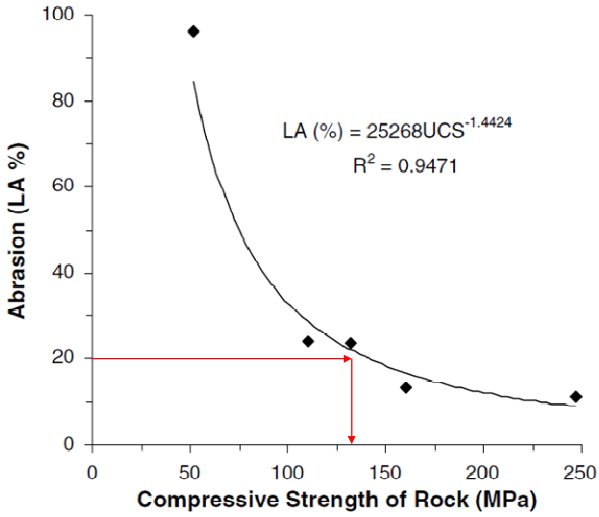
種類	表乾密度 (g/cm^3)	吸水率 (%)	压縮強度 (N/mm^2)
安山岩	2.62	2.55	190
硬質砂岩	2.65	0.44	315
硬質砂岩	2.72	0.68	164
石灰岩	2.68	0.92	113
石灰岩	2.69	0.49	116
石英片岩	2.61	0.92	262

砂石供應來源



砂石佔混凝土體積約70%，為強度及體積穩定性之關鍵材料，提供穩定高品質之砂石為高強度混凝土成功關鍵。

料源管制-骨材洛杉磯磨損率與骨材單壓強度關係



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料源管制-以化性確認來源

化學成分

- 不同區域確實有差異
- 用檢驗以管制料源
- 每兩週要求混凝土供應商取樣送驗，建立粒料化性資料庫。
- 提供粒料開始供應時比對參考。

Sample:	力泰樣品	
Compound	Value	Unit
SiO2	84.917	%
Al2O3	13.029	%
Fe2O3	5.45	%
CaO	0	%
MgO	1.396	%
SO3	0.349	%
K2O	1.602	%
Na2O	0.721	%
P2O5	0.064	%
TiO2	0.643	%
SrO	0.007	%
MnO	0.06	%

Sample:	國產樣品 3/25	
Compound	Value	Unit
SiO2	83.988	%
Al2O3	13.961	%
Fe2O3	7.107	%
CaO	0	%
MgO	1.731	%
SO3	0.346	%
K2O	1.633	%
Na2O	0.757	%
P2O5	0.054	%
TiO2	0.74	%
SrO	0.022	%
MnO	0.133	%

Sample:	國產樣品 2/12	
Compound	Value	Unit
SiO2	85.826	%
Al2O3	11.542	%
Fe2O3	4.438	%
CaO	0	%
MgO	1.802	%
SO3	0.377	%
K2O	1.388	%
Na2O	0.957	%
P2O5	0.054	%
TiO2	0.54	%
SrO	0.058	%
MnO	0.097	%

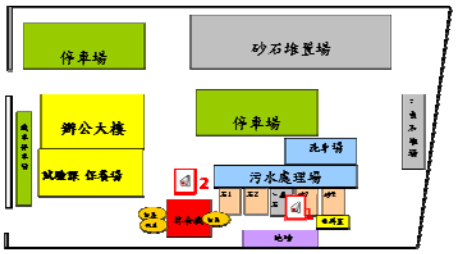
Sample:	RUENTEX 3/27	
Compound	Value	Unit
SiO2	9.23	%
Al2O3	2.458	%
Fe2O3	0.234	%
CaO	86.254	%
MgO	0.892	%
SO3	0.184	%
K2O	0.119	%
Na2O	0.095	%
P2O5	0.036	%
TiO2	0.082	%
SrO	0.111	%
MnO	0.026	%

生產管理-要求獨立料艙

- 混凝土供應商以獨立料倉於施工前入廠堆置，由駐廠人員取樣檢驗：

力泰建設企業股份有限公司汐止廠
平 面 配 置 圖

國產實業建設股份有限公司台北廠
平面配置圖



1

進料監視系統管制:

- 全程監視系統監控
- 控制盤定位進料

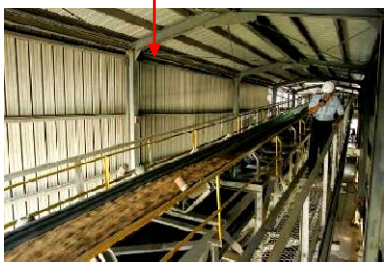
2

出料監視系統管制:

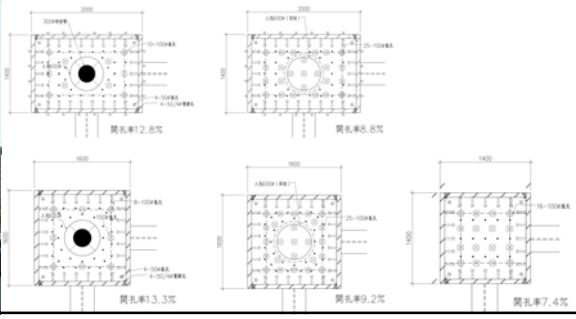
- 配比由電腦設定料倉
- 使用量每日回饋

生產管理-要求獨立料艙

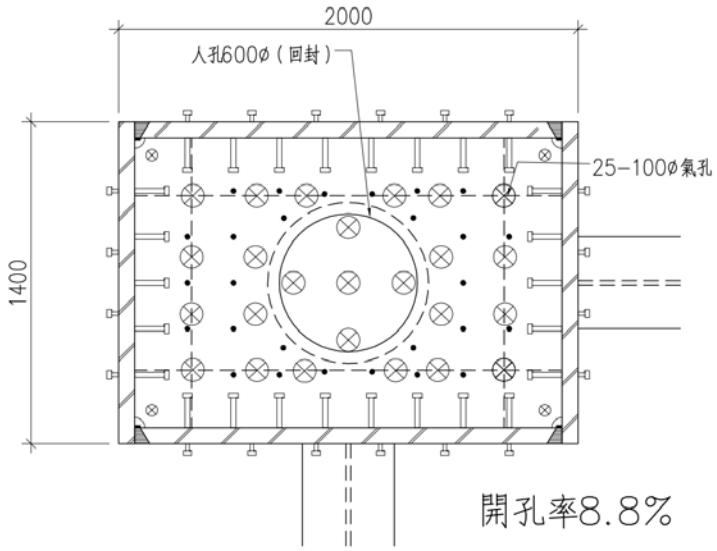
- 砂石品質分析
- 使用噴霧設備，保持含水量穩定
- 砂石分倉儲存



南山廣場案例-
鋼柱內填充無收縮自充填混凝土



巨柱泵送逆打：需具有似SCC的工作度



巨柱內混凝土塑性沈降問題：

-限制沈降量與泌水率

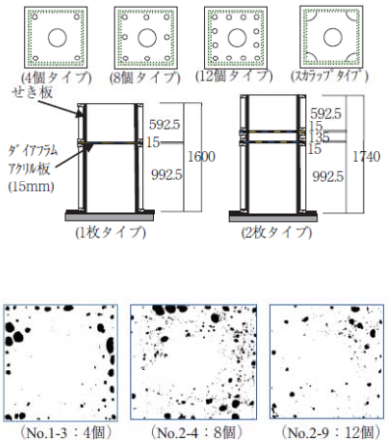
1. 參考JASS-5T-503:2009，「混凝土塑性沈陷量測方法」進行。
2. 置於大氣環境，量測至24小時止。
3. 泌水率以CNS1235方式為之。

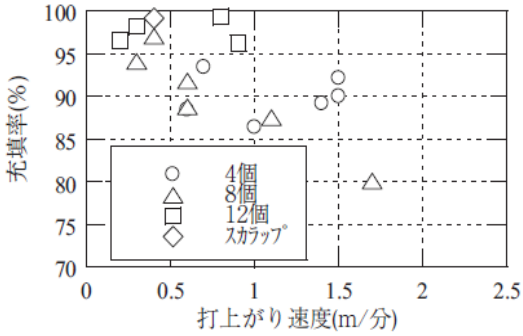
巨柱內混凝土自體收縮問題：

-添加膨脹劑

1. 參考ASTM C341方法。
2. 包封，阻止水分交換，並置於標準溫度環境。
3. 七天齡期前，每日量測；七天齡期後，每週量測。

逆打速率與充填率：限制逆打速率





$$充填率(\%) = \frac{(試験体内法面積 - 気泡面積) \times 100}{試験体内法面積} \quad (1)$$

柱內灌漿混凝土的性質：配比性能

2.1　材料

2.1.1　鋼柱內填充無收縮自充填混凝土之硬固混凝土性質要求：

試驗項目		品質要求	試驗方法
91 天齡期試體抗壓強度	12000 等級	>840kg/cm^2	CNS C1232
	10000 等級	>700kg/cm^2	
91 天齡期試體膨脹率		(0.04±0.02) %	CNS C14603
24 小時泌水率		<0.02cm^3/cm^2	CNS 1235
24 小時試體塑性沈降量		2mm 以下	JASS 5T-503:2009

2.1.2　本工程使用無收縮特性之自充填混凝土，其新拌混凝土的物理特性，應滿足 CNS 14841 粉體系之 R2 等級要求。

柱內灌漿混凝土的性質：使用原物料

2.1.3　本工程使用無收縮特性自充填混凝土配比之組成物料除應滿足國家標準外，特別要求如下：

A. 使用 CNS 61 之卜特蘭 II 型水泥。

B. 使用矽灰之 SiO2 成分應大於 90%、LOI 應小於 3%、比表面積應大於 20000 m2/kg，其用以取代部分水泥量，相對於水泥之使用量至少 5%，使用矽灰之性質可詳見附件。

C. 配比使用三分石為粗骨材，其來源應為大雪山系之硬質砂岩；使用骨材之洛杉磯磨耗測試數值應 20%以下、硫酸鈉健度試驗損失量應為 10%以下，供應商應就使用骨材送交本司確認後方得使用，使用骨材性質可參考附件。

D. 細骨材細度模數 FM=2.9±0.2。

E. 本工程使用之無收縮添加劑，應以鈣鎂石為主要成分，不同配比使用之無收縮添加劑用量可參考附件，並應以試拌結果確認之。

F. 化學摻料應符合 CNS12833 流動化混凝土用化學摻料。

G. 空氣含氣量小於 3%。

H. 配比用水量不超過 170kg/m3。

柱內灌漿混凝土的性質：廠試拌驗證業

2.1.5 供料商應以試拌方式，進行無收縮自充填混凝土之廠試拌，用以確認性質符合需求，廠試拌過程，除 2.1.1、2.1.2 節之測試項目外，應統計配比單位重供參。

柱內灌漿混凝土的性質：施工前抽驗

3.1.1 每次供應無收縮自充填混凝土前，供料商應以獨立料艙方式，進行骨材原料的儲放，以供駐廠人員抽驗。

3.1.2 施工前，施工承商應確認無收縮化學摻料之添加計畫，內容至少包括每次施工前後無收縮添加劑應進行數量的檢算供駐廠人員確認，以防止無收縮化學摻料添加過量或不足。

3.1.3 施工單位應於施工前擬具施工泵送計畫，內容含括泵送佈設、泵送配管與管徑、泵送設備性能等資料由工程司審核後方可施工。

柱內灌漿混凝土的性質：施工中管制

3.2 施工管理

3.2.1 柱內灌漿時，施工單位應控制澆置鋼管內無收縮自充填混凝土升高的速率應於 0.5m/每分鐘以下。

3.2.2 每次無收縮自充填混凝土出貨單上，應載明出廠時間；送抵工地後，應交由現場監造人員確認無收縮自充填混凝土送抵時間，兩者時間不得超過 1 小時，否則予以退車處理。

3.2.3 施工時，無收縮自充填混凝土供應商，應以每 50m3 取樣乙次之取樣頻率，配合進行強度、坍流度的施工檢驗。

3.2.3 若現場檢驗之坍流度值、坍流時間不符規範要求，則依不足比例進行扣款。

3.2.4 若 φ15×30 試體之沈降量大於[1mm]或泌水率超過[0.1cm3/cm2]，則施工單位應即警戒並通知混凝土供應商進行配比的確認與檢討。

從原料到施工：

原料料源

- ◆ 水泥。
- ◆ 矽灰。
- ◆ 無收縮添加劑。
- ◆ 化學摻料。
- ◆ 水。
- ◆ 粒料。

粗粒料進廠管制

- ◆ 產地。
- ◆ 專用料倉。
- ◆ 設置監控錄影機。
- ◆ 分批進場，分批檢驗
  1. S.S.D比重。
  2. 吸水率。
  3. 洛杉磯磨損率。
  4. 硫酸鈉健度試驗。
  5. 扁平、細長、扁平率。
  6. 篩分析。
- ◆ 粒料使用量控管紀錄。

生產管制

- ◆ 配比設計、廠試拌。
- ◆ 供料前一天(駐廠)
  1.取樣作化性。
  2.留樣作物性。
- ◆ 供料當天(駐廠)
  1. 每車次進行坍流度試驗。
  2. 每車次B50坍流時間。
  3. 塑性沈降量量測<1mm。
  4. 每100m3製作膨脹率試體。

施工管制

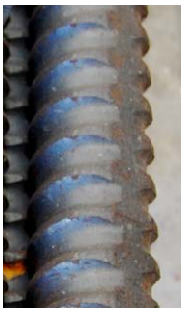
- ◆ 每50m3進行坍流度試驗。
- ◆ 每50m3進行B50坍流時間。
- ◆ 每100m3製作強度試體。
- ◆ 每天製作一組體積膨脹試體。
- ◆ 進場時間不能超過1小時。
- ◆ 柱內灌漿上升速度0.5m/min以下。

結論：

1. 混凝土材料是**量身定做**的。
2. 柱內灌漿混凝土除強度性能外，**塑性沈降**、**泌水率**以及**泵送能力的漿體量**，是重要的指標，一般SCC的規範無法含括。
3. 南山柱內灌漿混凝土規範，是國內第一次針對柱內灌漿需求所研擬的規範，可供未來工程參考。
4. 南山案針對**原物料的穩定性**問題，所規劃的留樣方法，符合統計的意義，應在後續工程中予以推動落實。
5. NewRC構造物所使用的混凝土材料，也應特別量身定做，訂定出符合需求的項目。

高強度鋼筋混凝土結構桿件設計研討會
NCREE,12/5, 2014

螺紋節鋼筋在高強度鋼筋混凝土之應用與握裹設計

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December 05, 2014

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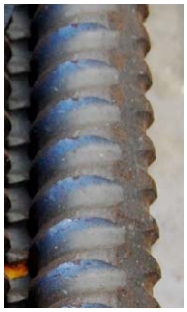
螺紋節鋼筋與竹節鋼筋

螺紋節鋼筋

竹節鋼筋


NCREE

Contents

- Background
- Objectives
- Geometric Pattern of Threaded Bars
- Experimental Program
- Strength Results
- Splitting Results
- Summary

NCREE

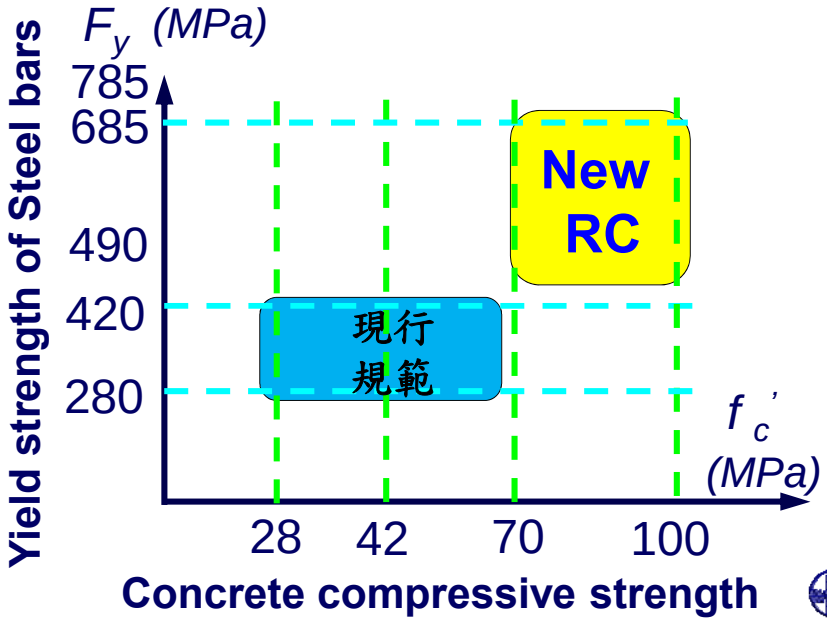
Contents

- Background
- Geometric Pattern of Threaded Bars
- Demand of Bond Stress of Bar
- Experimental Program
- Strength Results
- Splitting Results
- Summary

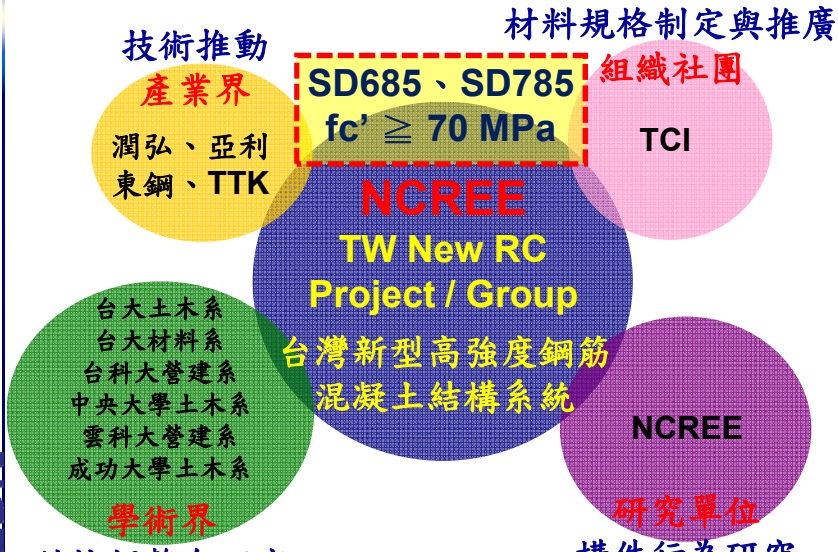
NCREE

日本New RC建築經驗

New RC 材料強度應用範圍



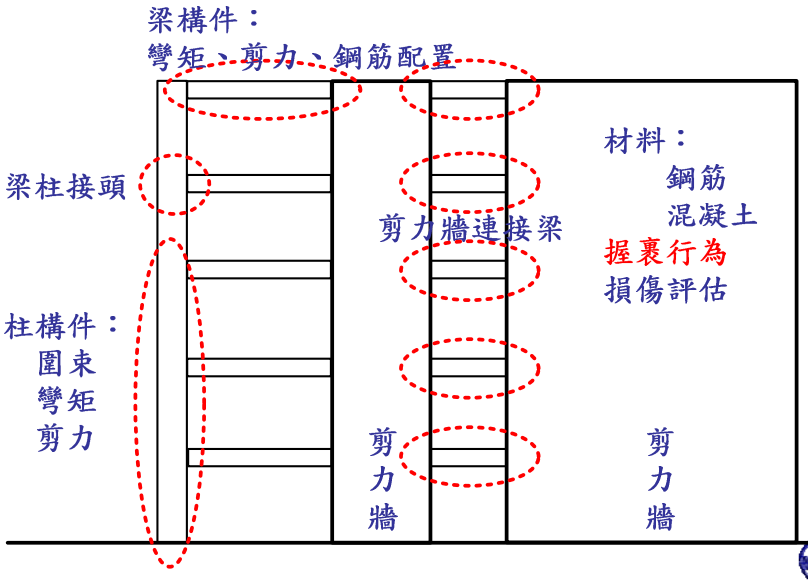
Taiwan New RC Group 2008~



台灣新型高強度鋼筋混凝土結構系統研發計畫與團隊(Taiwan New RC)

材料研發 [經濟部科專計畫] [台灣混凝土學會]	構件行為研究 [國科會整合型計畫]	結構分析、設計與施工 [國震中心專案計畫] [台灣混凝土學會]
工作內容 ● SD685、SD785 鋼筋研發 ● 鋼筋續接器與端錨器研發 ● 100MPa 混凝土配比研發與基本性質識別 ● 建立高強度鋼筋混凝土握裹與錨定特性 ● 含纖維超高強度混凝土 ● 高強度混凝土的乾縮、潛與耐火性質	● 高強度鋼筋混凝土 T 形梁耐震行為研究 ● 梁柱接頭耐震性能研究及資料庫建構 ● 高強度鋼筋與混凝土柱主筋降伏後剪力強度衰減模型建構 ● 高強度鋼筋混凝土構件之損傷評估與生命週期修復性能研究 ● 高強度纖維混凝土剪力行為研究 ● 高強度鋼筋混凝土柱圍束作用之研究 ● 剪力牆連接梁力學行為研究 ● 高強度鋼筋混凝土之握裹行為研究	● 現行規範採用之分析與設計模型適用性檢討 ● 建立彈性與非彈性兩階段分析法 ● 建立各構件非線性分析模型的 ● 建立柱構件非線性分析模型 ● 損傷評估不含纖維 ● 建立完整分析設計示範例 ● 提出分析與設計之指針或準則 ● 研擬施工建議與規範 ● 提出構件設計與施工之細節與指針

主要構件行為研究



Critical Technology and Devices

- 超高強度鋼筋不允許銲接與製作標準彎曲加工。
- 採用螺紋節鋼筋
- 配合預鑄工法的應用

- 灌漿式鋼筋續接套筒
- 灌漿式鋼筋端部錨定裝置

Contents

NCREE

Objectives

ACI 318-11 and Taiwan (2011) Codes

Straight Development length in tension (unit : SI)

$$l_d = \frac{A_b \cdot f_s}{\pi \, u \, d_b} \Rightarrow l_d = 0.9 \frac{f_y}{\sqrt{f_c'}} \frac{1}{\left(\frac{c_b + k_{tr}}{d_b}\right)} d_b$$

Limitations for Equation

- $f_y \leqq 412$ MPa
- $f_c' \leqq 70$ MPa

$$1.0 \le \left(\frac{c_b + K_{tr}}{d_b}\right) \le 2.5$$

$$K_{tr} = \frac{40 A_{tr}}{sn} \quad \text{(ACI 318-11)}$$

$$K_{tr} = \frac{A_{tr} f_{yt}}{10.3 sn} \quad \text{(Taiwan, 2011)}$$

12.1.2 — The values of $\sqrt{f_c'}$ used in this chapter shall not exceed 100 psi.

From 1989

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Contents

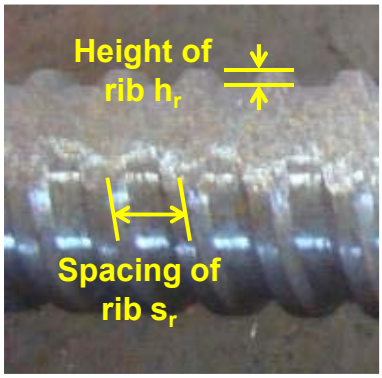
NCREE

Types of Rebar for Tests

Thread Rebar
Hot roll

Thread Rebar
Milled

Plain Rebar



Relative Area of Rebar

$$R_r = \frac{\text{Height of rib}}{\text{Spacing of rib}} = \frac{h_r}{s_r}$$

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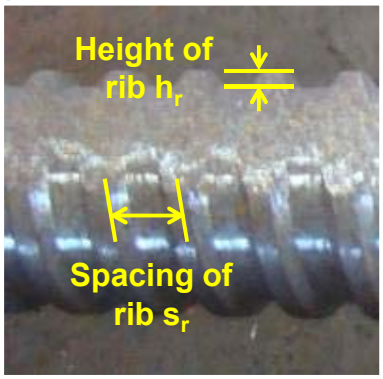
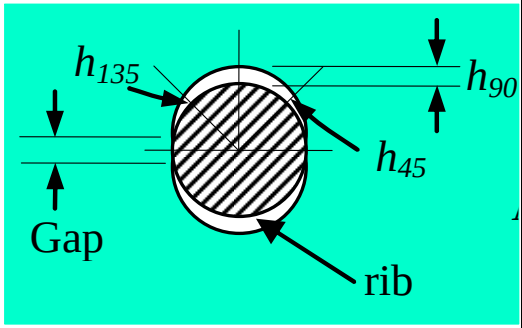
Parameter of Threaded Rebar

Relative Rib Area R_r





$$R_r = \frac{bearing\ area}{shear\ area} \approx \frac{h_r}{s_r}$$

Max. $h_r = h_{90}$

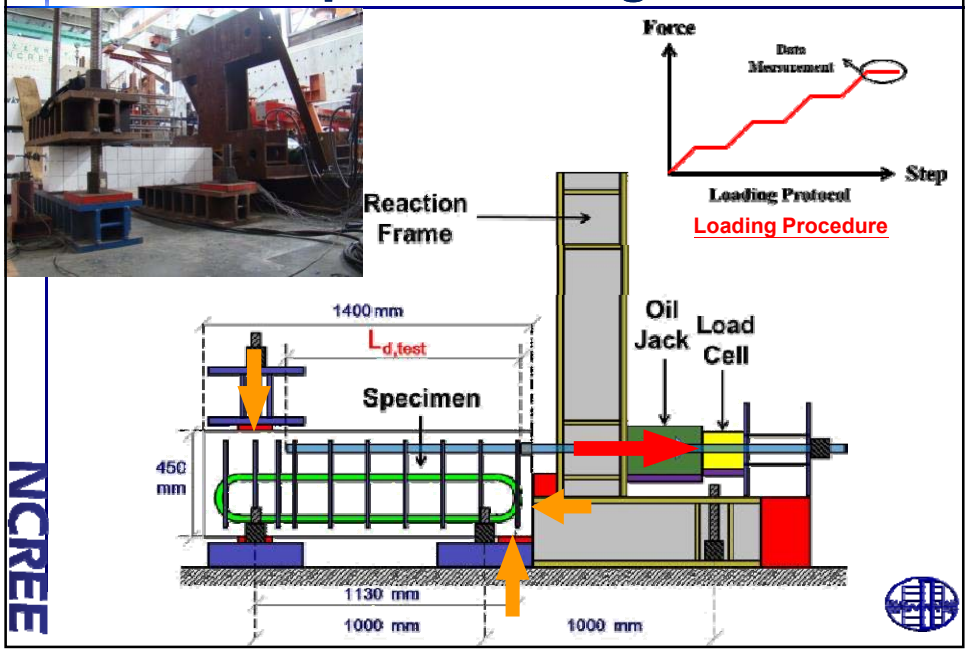
Avg. $h_r = (h_{90}+h_{45}+h_{135})/3$

NCREE

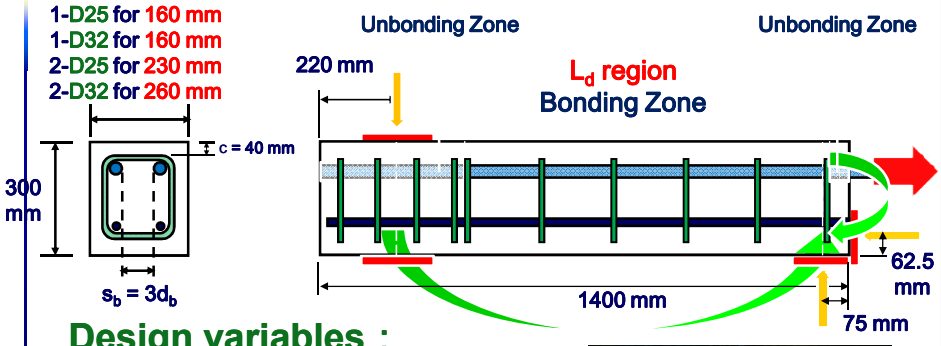
Contents

NCREE

Setup of Bonding Tests



Design Variables of Specimens (I)



Design variables :

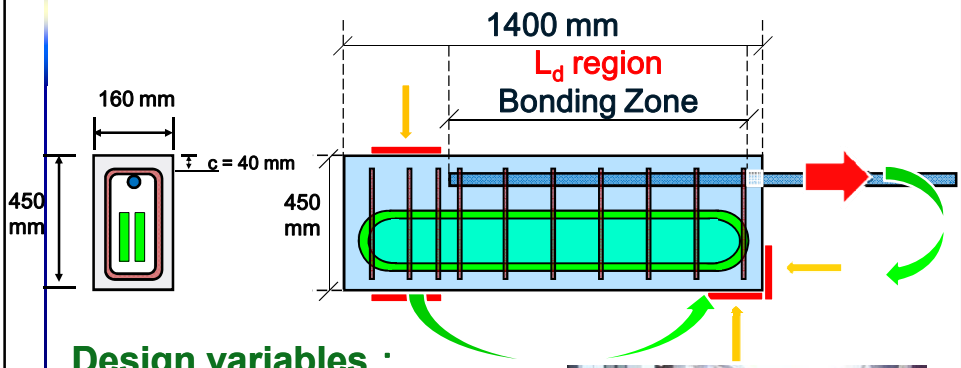
1. Bar size : D25, D32
2. Number of bars : 1, 2-($3d_b$)
3. $L_{d,test}$ / $L_{d,dem}$: 0.6, 0.8, 1.0
4. h_r / s_r : 0.126, 0.143
5. f_c' : 70 MPa

Design of Specimens (I)

Spec.	Width (mm)	$d_{b,b}$ (mm)	$d_{b,s}$ (mm)	f_c' (MPa)	f_y (MPa)	c (mm)	n_b	s_b (mm)	S_s (mm)	K_{tr} (mm)	D. Length		Ratio
											$L_{d,dem}$ (mm)	$L_{d,test}$ (mm)	$\frac{L_{d,test}}{L_{d,dem}}$
BE1	160	25	12.7	70	685	40	1	0	200	25.4	762	750	0.98
BE2	160	25	12.7	70	685	40	1	0	200	25.4	762	600	0.79
BE3	160	25	12.7	70	685	40	1	0	200	25.4	762	450	0.59
BE4	230	25	12.7	70	685	40	2	76.2	100	50.7	762	750	0.98
BE7	160	32	12.7	70	685	40	1	0	200	25.4	966	950	0.98
BE8	160	32	12.7	70	685	40	1	0	200	25.4	966	750	0.78
BE9	160	32	12.7	70	685	40	1	0	200	25.4	966	600	0.62
BE10	260	32	12.7	70	685	40	2	96.6	100	50.7	966	950	0.98

Depth : 300 mm

Design Variables of Specimens (II)



Design variables :

1. Bar size : D32, D39
2. Geometry : Thread, Plain
3. f_c' : 70, 100 MPa
4. f_{yt} : 280, 420, 785 MPa
5. $R_r = h_r / s_r$: 0, 0.13, 0.16, 0.18

Design of Specimens (II)

Spec.	Width (mm)	$d_{b,b}$ (mm)	$d_{b,s}$ (mm)	f_c' (MPa)	f_y (MPa)	f_{yt} (MPa	c_b (mm)	S_s (mm)	K_{tr} (mm)	D. Length		Ratio
										$L_{d,dem}$ (mm)	$L_{d,test}$ (mm)	$\frac{L_{d,test}}{L_{d,dem}}$
SP1	160	32	13	70	685	420	77.8	180	28.2	945	950	1.00
SP2	160	32	13	100	685	420	77.8	180	28.2	945	950	1.00
SP3	160	32	13	70	685	420	77.8	150	33.8	945	500	0.53
SP4	160	32	13	100	685	420	77.8	150	33.8	945	500	0.53
SP5	160	32	13	70	685	420	77.8	175	29.0	945	750	0.79
SP6	160	32	13	100	685	420	77.8	175	29.0	945	750	0.79
SP7	160	39	13	70	490	420	77.6	188	27.0	813	800	0.98
SP8	160	39	13	100	490	420	77.6	188	27.0	813	800	0.98
SP9	160	32	13	70	490	420	77.8	183	27.6	676	600	0.89
SP10	160	32	13	100	490	420	77.8	183	27.6	676	600	0.89
SP11	160	39	13	70	490	280	77.6	180	28.2	813	950	1.17
SP12	160	39	13	100	490	280	77.6	180	28.2	813	950	1.17
TD1	160	32	13	70	685	785	77.8	170	29.8	945	950	1.00
TD2	160	32	13	70	685	785	77.8	170	29.8	945	950	1.00

Depth : 450 mm

Actual R_r of Rebar for All Spec.

Specimen	$d_{b,b}$ (mm)	Geometric of Rebar	h_r (mm)	s_r (mm)	R_r
BE1-BE4	25	Thread Rebar Hot roll	1.44	10.0	0.143
BE7-BE10	32		1.50	11.9	0.126
SP1-SP2	32		1.56	11.88	0.131
SP3-SP4	32		2.21	12.06	0.183
SP5-SP6	32	Thread Rebar Milled	2.21	12.01	0.184
SP7-SP8	39	Thread Rebar Hot roll	2.37	14.8	0.160
SP9-SP10	32	Plain Rebar	X	X	X
SP11-SP12	39	Thread Rebar Hot roll	2.37	14.8	0.160
TD1-TD2	32	Thread Rebar Hot roll	2.22	12.34	0.180

Contents

NCREE

Actual Concrete Strength

Specimen	f_c', Testing Day (MPa)
BE1~BE3	64.3
BE4	62.7
BE7~BE9	90.9
BE10	91.8
SP1	80.4
SP2	110.6
SP3	84.8
SP4	110.7
SP5	90.7
SP6	106.2

Specimen	f_c', Testing Day (MPa)
SP7	93.8
SP8	109.0
SP9	86.6
SP10	106.4
SP11	91.4
SP12	102.3
TD1	71.8
TD2	71.8

Only BE1-BE4
$f_c' \leqq 70$ MPa

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Actual Reinforcement Strength

Group	Bar Size	Relative of Area R_r	Type	Yield f_{ya} (MPa)	Ultimate f_{ua} (MPa)
Series (I)	D25	0.143	SD685 Hot Roll	738	932
	D32	0.126	SD685 Hot Roll	742	966
Series (II)	D25	0.131, 0.183	SD685 Hot Roll	713	928
		0.184	SD685 Milled	777	971
		X	SD490 Plain	503	698
	D39	0.160	SD490 Hot Roll	534	701

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Test Results - Strength

Spec	Development Length (mm)				Length. Ratio			Results (kN)		
	$L_{d,dem}$	$L_{da,lmt}$	$L_{da,unlmtt}$	$L_{da,test}$	$\frac{L_{da,test}}{L_{d,dem}}$	$\frac{L_{da,test}}{L_{da,lmt}}$	$\frac{L_{da,test}}{L_{da,unlmt}}$	P_{ya}	P_{Max}	$\frac{P_{Max}}{P_{ya}}$
SP1	945	1000	923	825	0.87	0.83	0.89	578.18	560.68	0.97
SP2	945	1000	787	745	0.79	0.75	0.95	578.18	516.73	0.89
SP3	945	1007	906	310	0.33	0.31	0.34	583.06	296.44	0.51
SP4	945	1007	793	328	0.35	0.33	0.41	583.06	277.69	0.48
SP5	945	1093	951	563	0.60	0.51	0.59	632.74	442.68	0.70
SP6	945	1093	879	545	0.58	0.50	0.62	632.74	504.44	0.80
SP7	813	903	773	605	0.74	0.67	0.78	628.14	513.05	0.82
SP8	813	903	717	613	0.75	0.68	0.85	628.14	510.08	0.81
SP9	676	708	630	600	0.89	0.85	0.95	409.61	121.57	0.30
SP10	676	708	569	600	0.89	0.85	1.06	409.61	141.27	0.35
SP11	813	903	783	735	0.90	0.81	0.94	628.14	586.52	0.93
SP12	813	903	740	755	0.93	0.84	1.02	628.14	591.70	0.94
TD1	945	990	968	890	0.94	0.90	0.92	573.26	654.64	1.14
TD1	945	990	968	885	0.94	0.89	0.91	573.26	608.07	1.06

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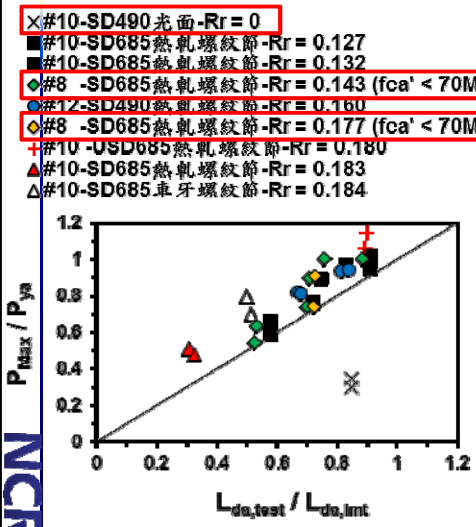
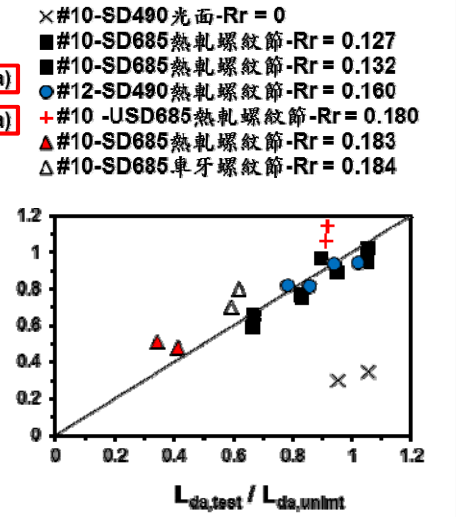
Test Results - Strength



Limit for $f_c' \leqq 70$ MPa



No Limit for $f_c' \leqq 70$ MPa

Bond Stress

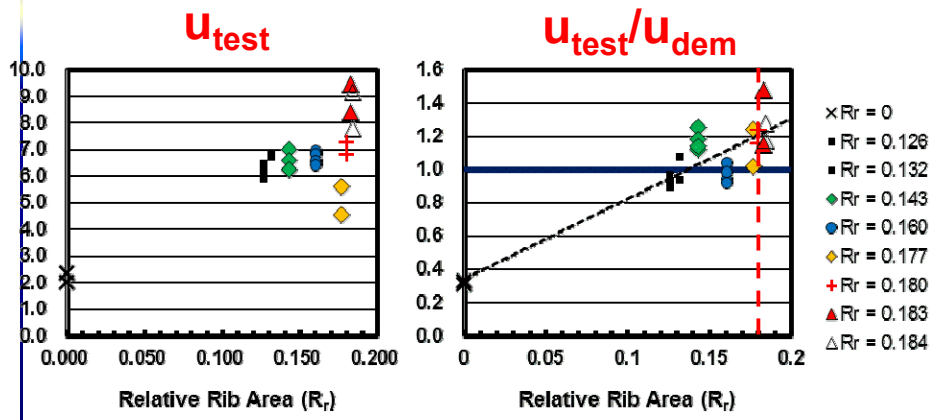
$$u_{dem} = \frac{A_b \times f_s}{\pi \times d_b \times \ell_d} = \frac{\frac{\pi d_b^2}{4} \times f_s}{\pi \times d_b \times \frac{0.9 f_s \times d_b}{\sqrt{f_{ca}'}\left(\frac{c_b + K_{tr}}{d_b}\right)}}$$

$$= \frac{\sqrt{f_{ca}'}\left(\frac{c_b + K_{tr}}{d_b}\right)}{3.6} = \frac{(1 \sim 2.5)\sqrt{f_{ca}'}}{3.6}$$

Calculation for Bond Stress

Material types	Max. Rib height (h_r) (mm)	Avg. Rib height (h_r) (mm)	Rib spacing (s_r) (mm)	Relative rib area ($R_r = h_r / s_r$)		$(C_b+K_{tr})/d_b$ Top Side	f_{ca}' (MPa)	u_{test} (MPa)	u_{dem} (MPa)	R_{eff} (u_{test}/u_{dem})
				Max.	Avg.					
#8-SD685 Hot Rolled	1.65	1.44	10.02	0.165	0.143	3.64	64.3	6.24	5.56	1.12
						4.22	64.3	6.98	5.56	1.25
							64.3	6.58	5.56	1.18
							62.7	6.26	5.50	1.14
#10-SD685 Hot Rolled (1)	1.67	1.50	11.88	0.141	0.126	2.94	90.9	5.96	6.62	0.90
						3.29	90.9	6.15	6.62	0.93
							90.9	5.92	6.62	0.89
							91.8	6.45	6.65	0.97
#10-SD685 Hot Rolled (2)	1.54	1.56	11.88	0.129	0.132	3.36	80.4	6.72	6.22	1.08
						3.43	110.6	6.86	7.30	0.94
#10-SD685 Hot Rolled (3)	2.35	2.21	12.06	0.194	0.183	3.36	84.8	9.45	6.39	1.48
						3.43	110.7	8.38	7.30	1.15
#10-SD685 Hot Rolled (4)	2.49	2.22	12.34	0.202	0.180	3.34	71.8	7.27	5.88	1.24
						3.41	71.8	6.79	5.88	1.15
#10-SD685 Milled	2.27	2.21	12.01	0.189	0.184	3.36	90.7	7.78	6.61	1.18
						3.43	106.2	9.15	7.15	1.28
#12-SD490 Hot Rolled	2.46	2.37	14.8	0.166	0.160	2.72	93.8	6.98	6.72	1.04
						2.78	109	6.85	7.25	0.95
							91.4	6.56	6.63	0.99
							102.3	6.45	7.02	0.92
#10-SD490 Plain	X	X	X	X	X	3.27	86.6	2.00	6.46	0.31
						3.35	106.4	2.33	7.16	0.33

Bond Stress



$$u_{dem} = \frac{A_b \times f_s}{\pi \times d_b \times \ell_d} = \frac{\frac{\pi d_b^2}{4} \times f_s}{\pi \times d_b \times \frac{0.9 f_s \times d_b}{\sqrt{f_{ca}'}\left(\frac{c_b + K_{tr}}{d_b}\right)}} = \frac{\sqrt{f_{ca}'}\left(\frac{c_b + K_{tr}}{d_b}\right)}{3.6} = \frac{(1 \sim 2.5)\sqrt{f_{ca}'}}{3.6}$$

Contents

NCREE

Test Results – Splitting (I)

Specimen (n_b = 1)	$\frac{c_b + K_{tr}}{d_b}$		Splitting	
	Top cover	Side cover	Top cover	Side cover
BE1 – BE3	3.64	4.22	■	—
BE7 – BE9	2.94	3.29	■	■

BE1 – BE3

BE7 – BE9

$\left(\frac{c_b + K_{tr}}{d_b}\right) \approx 3.65$ is a upper bound for splitting

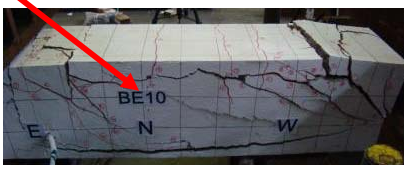
NCREE

Test Results – Splitting (I)

Specimen (n_b = 2)	$\frac{c_b + K_{tr}}{d_b}$			Splitting	
	Top cover	Side cover	Bar spacing	Top	Side
BE4	4.64	4.63	4.03	–	–
BE10	3.73	3.65	3.58	–	■

BE4

BE10

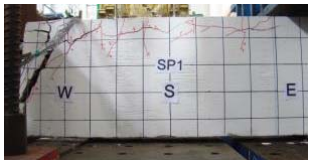


$\left(\frac{c_b + K_{tr}}{d_b}\right) \approx 3.65$ is a upper bound for splitting

Test Results – Splitting (II)

Specimen	$\frac{c_b + K_{tr}}{d_b}$		Splitting	
	Top cover	Side cover	Top cover	Side cover
SP1 – SP6	3.36	3.43	■	■
SP7 – SP8 SP11 – SP12	2.72	2.78	■	■

SP1-SP6



SP7-SP8, SP11-SP12

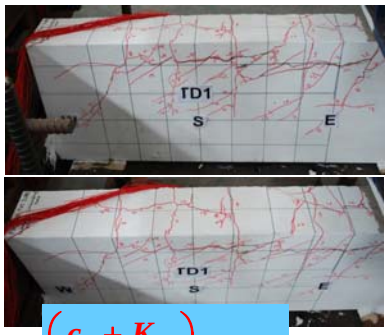
$\left(\frac{c_b + K_{tr}}{d_b}\right) \approx 3.65$

is a upper bound for splitting

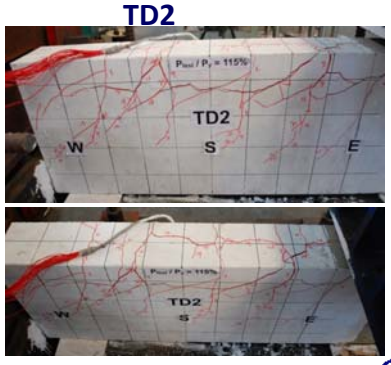
Test Results – Splitting (II)

Specimen	$\frac{c_b + K_{tr}}{d_b}$		Splitting	
	Top cover	Side cover	Top cover	Side cover
TD1 – TD2	3.34	3.41	■	■

TD1



TD2



$\left(\frac{c_b + K_{tr}}{d_b}\right) \approx 3.65$ is a upper bound for splitting

Proposed Bond Model of Threaded Rebar within HRC

Development length in tension following ACI 318 code (unit : SI)

$$l_d = 0.9 \frac{f_y}{\sqrt{f_c'}} \frac{1}{\left(\frac{c_b + k_{tr}}{d_b}\right)} d_b$$

$$R_r = \frac{h_r}{s_r} \geq 0.18$$

Limitations for Bond Equation

- $f_y \leqq 685$ MPa
- $f_{yt} \leqq 412$ MPa
- $f_c' \leqq 100$ MPa

$$1.0 \leq \left(\frac{c_b + K_{tr}}{d_b}\right) \leq 2.5$$

$$K_{tr} = \frac{A_{tr} f_{yt}}{10.3sn}$$

Contents

NCREE

Summary (I)

- 試驗結果顯示，本研究所採用螺紋節鋼筋，當採用規範建議之握裹模型，且混凝土強度受70MPa上限限制時，均能發揮模型之預期握裹性能。

$$l_d = 0.9 \frac{f_y}{\sqrt{f_c'}} \frac{1}{\left(\frac{c_b + k_{tr}}{d_b}\right)} d_b$$

$$f_c' \le 70\ MPa$$

$$f_c' > 70\ MPa$$

- 螺紋節鋼筋之表面相對節面積平均R_r值為0.18者，在應用規範建議之握裹模型時，可不受混凝土強度70MPa上限之限制。

NCREE

Summary (II)

◆ 本研究建議鋼筋拉力直線發展長度公式為

$$l_d = 0.9 \frac{f_y}{\sqrt{f_c'}} \frac{1}{\left(\frac{c_b + k_{tr}}{d_b}\right)} d_b$$

$$f_c' \le 100\ MPa$$
$$f_y \le 685\ MPa$$

但是否能不受混凝土強度上限100 MPa之限制，為未來研究方向。

◆ 上述模型建立在螺紋節鋼筋R_r值為0.18以上之基礎。

NCREE

Summary (II)

社團法人 台灣混凝土學會
Taiwan Concrete Institute

鋼筋混凝土用鋼筋
Steel bars for Concrete Reinforcement
—SD550W、SD685、SD785

新高強度鋼筋混凝土技術委員會報告

主任委員：黃世建
副主任委員：詹穎雯
幹　　事：張宏印 林克強
委　　員：十鳴成 王燈凱 朱　剛
[illegible] 何長慶 [illegible]
[illegible] 陳正誠 陳[illegible]
陳[illegible] 夏[illegible] 趙文成
[illegible] [illegible]
研究助理：陳智軒（[illegible]）
2014年29月審定版

4. 形狀、尺度、質量及許可差

4.5 鋼筋之節，其高度最小值及最大值依表2之規定。為確保鋼筋在混凝土之握裹強度，**竹節鋼筋稱號D22以上**，其節之高度平均值不得小於其節距平均值之**10%**；**螺紋節鋼筋稱號D22以上**，其節之高度平均值不得小於其節距平均值之**18%**。

表4　螺紋節鋼筋之標示代號、單位質量、標稱尺度及節之尺度

鋼筋 稱號	標示代號	單位質量 (W) kg/m	標稱直徑 (d) mm	標稱剖面積 (S) mm²	標稱周長 (ℓ) mm	節之尺度			
						節距平均值最大值 (P) mm	節之高度(a) 最小值 (h) mm	節之高度(a) 最大值 (H) mm	單一間隙寬度最大值 (b) mm
D13	4	0.994	12.7	126.7	40	6.4	0.8	1.3	5.0
D16	5	1.56	15.9	198.6	50	8.0	1.0	1.7	6.2
D19	6	2.25	19.1	286.5	60	9.6	1.2	2.0	7.5
D22	7	3.04	22.2	387.1	70	11.1	1.3	2.2	8.7
D25	8	3.98	25.4	506.7	80	12.7	1.6	2.6	10.0
D29	9	5.08	28.7	646.9	90	14.4	1.7	2.8	11.3
D32	10	6.39	32.2	814.3	101	16.1	1.9	3.2	12.6
D36	11	7.90	35.8	1007	113	17.9	2.2	3.6	14.1
D39	12	9.57	39.4	1219	124	19.7	2.4	4.0	15.5
D43	14	11.4	43.0	1452	135	21.5	2.5	4.2	16.9
D50	16	15.5	50.2	1979	158	25.1	3.0	5.0	19.7
D57	18	20.2	57.3	2579	180	28.7	3.5	5.8	22.5

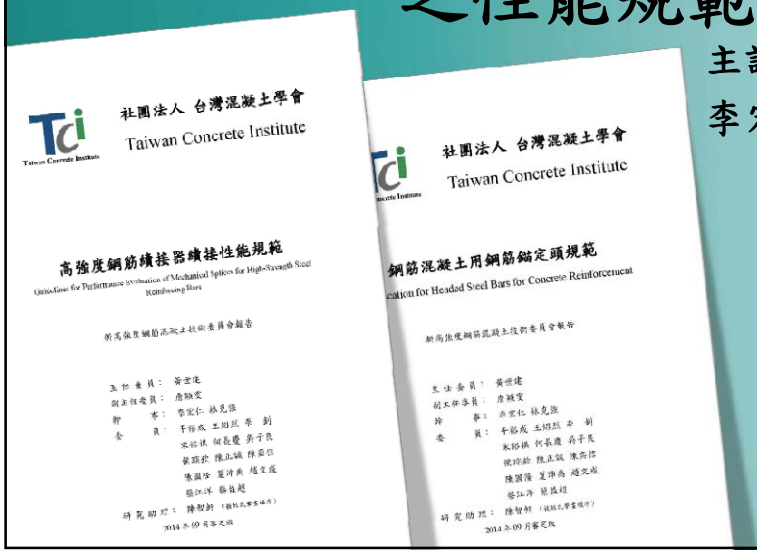
NCREE

Summary (III)

- 試驗結果顯示，螺紋節鋼筋之相對節面積R_r值愈大，其鋼筋所能提供之握裹應力越大。
- 鋼筋實際握裹應力與 $\sqrt{f_c'}$ 之比值，約與R_r值成正比。
- 試驗結果顯示，發生劈裂破壞之 $\left(\frac{c_b + k_{tr}}{d_b}\right)$ 最大值約為3.65。

NCREE

高強度鋼筋續接器及錨定頭之性能規範



主講人：
李宏仁 副教授

簡報大綱

一. <u>規範背景與目的</u>

二. 鋼筋機械式續接及錨定工法介紹

三. 續接器續接性能規範及測試

四. 錨定頭規範及測試

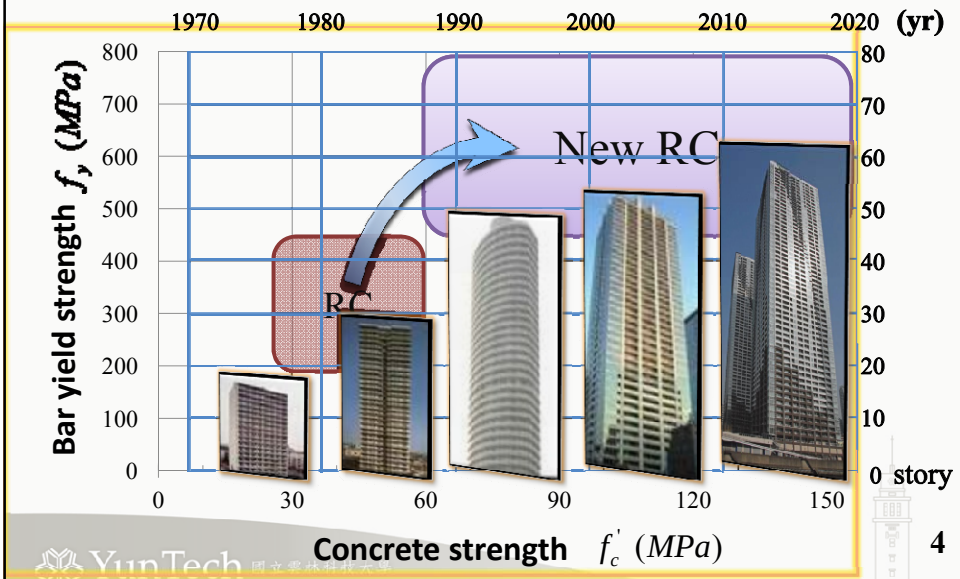
五. 結語

背景

時間	21層以上建築
2009	109
2010	90
2011	71
2012	142
2013	160
2014 前三季	161

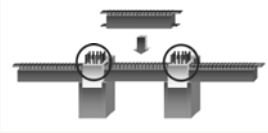
資料來源：內政部營建署

日本 New RC 建築技術發展



參考日本超高層建築發展

高強度鋼筋混凝土
預鑄工法



鋼筋組立工法之改良

最經濟

經濟、快速、品質

工法 / 比較	傳統現場組立	預組工法	預鑄工法
混凝土/模板	現場一體澆置	現場一體澆置	工廠預鑄構件 現場澆置接合部
鋼筋續接	搭接 螺紋式續接	~~搭接~~ 螺紋式續接	套管式續接 螺紋式續接
材料	中	低	高
人力	多	中	少
工期	慢	中	快

TOTAL COST DOWN !! 總成本降低

續接器用螺紋鋼筋

- 簡稱螺紋節鋼筋
- 德國發明、日本改良、引進台灣

Germany

SAH Stahlwerk Annahütte

TTK 東京鉄鋼 Japan

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

預組工法-柱範例

(東京鉄鋼提供)

Prefabricated steel cages

(on- or off-site)

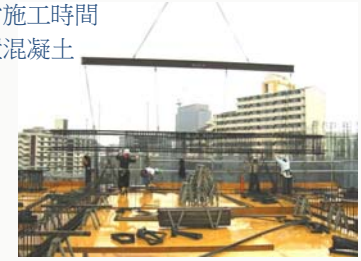
- 在建築現場或是工廠內先將鋼筋籠組合
- 以機具協助組立、節省施工時間
- 現場組模、支撐、澆置混凝土

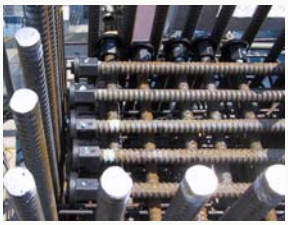
預組工法-梁範例

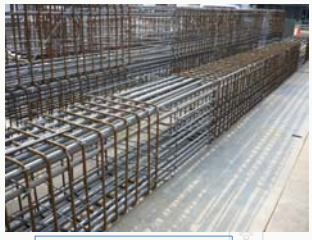
(東京鉄鋼提供)

Prefabricated steel cages
(on- or off-site)

- 在建築現場或是工廠內先將鋼筋籠組合
- 以機具協助組立、節省施工時間
- 現場組模、支撐、澆置混凝土







預組工法-梁

預鑄工法-柱範例

(東京鉄鋼提供)

Precast Construction Method

◆預鑄工法

- 專為預鑄工法設計之續接套管，置於預鑄柱構件內，現場組裝時注射特製砂漿續接
- 快速、高品質

Precast Construction Method

預鑄工法-梁範例

◆預鑄工法

- 專為預鑄工法設計之續接套管，現場對接梁主筋時注射特製砂漿完成續接
- 現場只澆置接合部混凝土(及部分樓板)
- 半乾式工法
- 快速、高品質

(東京鉄鋼提供)

①預鑄樑到接合處

②續接器接合

③螺栓固定

④砂漿注入

鋼筋規格及續接器、錨定頭規範

	Grade	#6 D19	#7 D22	#8 D25	#9 D29	#10 D32	#11 D36	#12 D39	#14 D43
	685								
	550								
	490								
	420								

表 11　鋼筋之機械性質

符號	機械性質								
	降伏點或降伏強度 (3) N/mm²	抗拉強度 N/mm²	實際抗拉強度 / 實際降伏強度	達降伏強度上限之應變 (4)	試片		伸長率 %	彎曲角度	彎曲直徑
SD 550W	550~675	690 以上	1.25 以上	0.014 以上	2 號 14A 號		12 以上	90°	標稱直徑之 4 倍
SD 685	685~785	860 以上	1.25 以上	0.014 以上	2 號 14A 號		10 以上	90°	標稱直徑之 4 倍
SD 785	785 以上	930 以上	–	–	母材	2 號 14A 號	8 以上	180°	標稱直徑之 3 倍
					銲接處 (5)	2 號 14A 號	5 以上	–	–

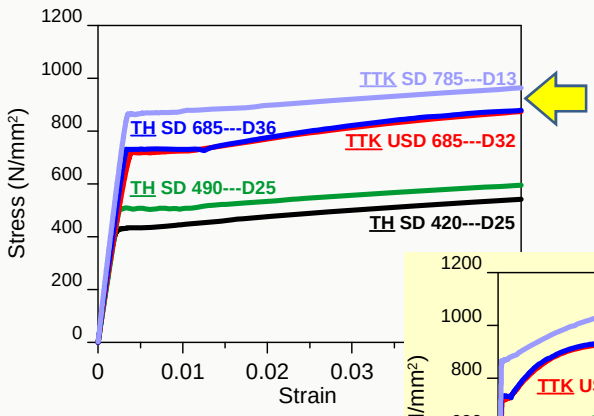
註(3) 參考 CNS 2111，降伏點不明顯時以 0.2 %橫距法測定。

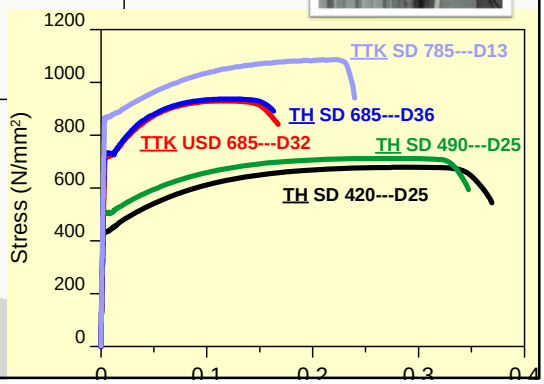
註(4) 參考 CNS 2111 及 2112，拉伸試驗時，以位移計求出伸長量除以標點距離，可求得軸向平均應變。SD 550W 鋼筋應力達 675 N/mm² 時，SD 685 鋼筋應力達 785 N/mm² 時，其對應之軸向應變值均不得少於 0.014。限定此變形範圍內鋼筋應力不超過降伏強度上限，以符合結構設計者所期待的耐震結構整體降伏機制。

註(5) 鋼筋 SD 785 直線對銲處於施工及試驗時均不得彎曲。

備考： 表 11 之鋼筋伸長率規定值適用於稱號 D32 以下，若稱號為 D36 時，則依表 11 之規定值減 1%， D39 時則依表 11 之規定值減 2%，D43 以上時依表 11 之規定值減 3%。

各等級鋼筋應力-應變曲線之比較





簡報大綱

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四. 錨定頭規範及測試

五. 結語

機械式續接-鋼筋經過加工具有螺紋接頭

鋼筋車牙

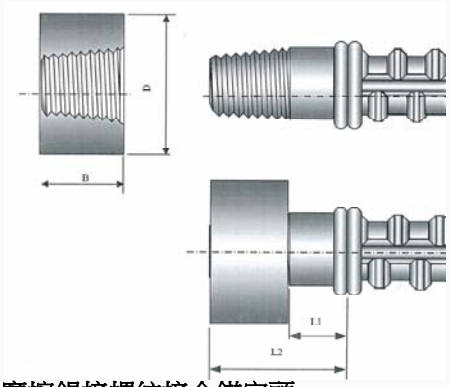
摩擦銲接三件式

摩擦銲接二件式

摩擦銲接四件式

資料來源：蘭州-詮新工程
http://www.cens.com/cens/html/zh/supplier/supplier_home_26668.html

機械式錨定-鋼筋經過加工具有螺紋接頭

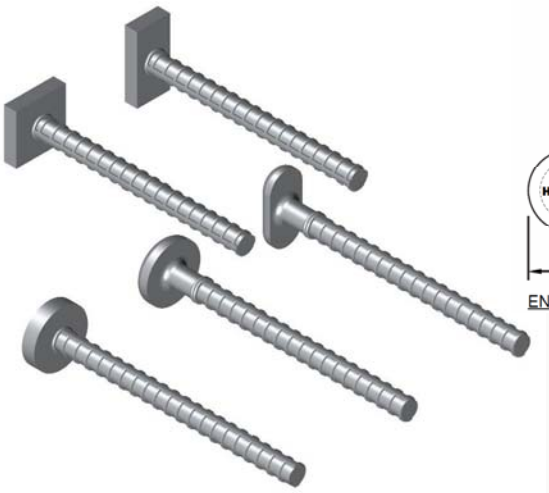


摩擦銲接螺紋接合錨定頭
(蘭州/詮新工程)

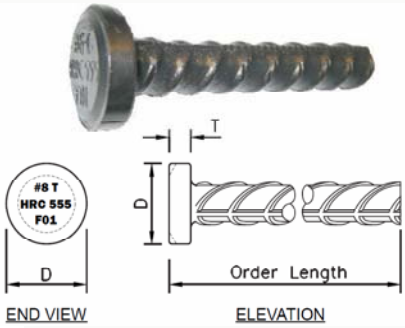
摩擦銲接螺紋接合錨定頭
(嘉山鋼鐵)

機械式錨定-非螺紋式、零滑動

- 摩擦壓接錨定頭鋼筋
(friction-welded headed bar)



- 鍛造錨定頭鋼筋(forged headed bar)



(HRC USA)

材料進場取樣?　　已組裝試體

照片：新北市江翠國小忠孝樓新建工程

工地抽驗? 取樣? 工地組立試體!

扭力試驗：鋼筋經車牙、滾牙、或摩擦銲接具有螺牙之接頭，應依製造商建議之扭力值在工地現場鎖緊，在箍筋及繫筋未綁紮固定之前，由監造人員以扭力板手抽驗其扭力值應大於製造商之建議值，抽驗數量不得低於該批產品數量之15%，不合格部份須鎖緊至扭力值之外，另再加倍抽驗直到合格為止。

工地組立試體：施工過程中，在工地自已組立之鋼筋籠切下之續接器接合試體。每滿200~300個取樣1個…

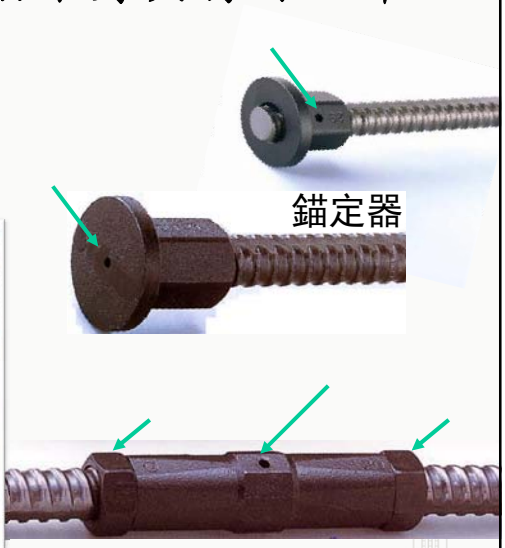
照片：新北市江翠國小忠孝樓新建工程

鋼筋綁紮完成才抽驗、不正確

機械式續接-鋼筋本身具有螺紋節

續接簡易 快速 降低施工成本!

- ✓ 熱軋成型
- ✓ 不須再加工螺紋
- ✓ 任何切點皆可接續、錨定
- ✓ 可對應各種施工型式，施工簡易



ACE JOINT

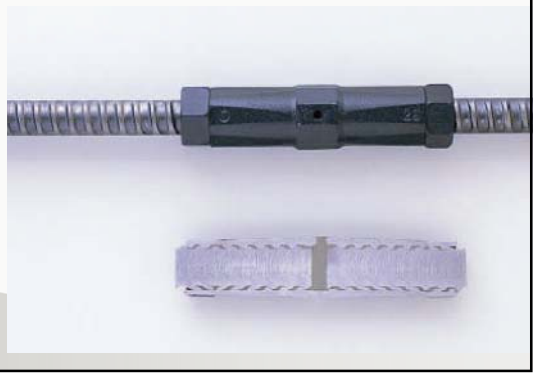
螺紋鋼筋砂漿填充續接器

TTK Grout 600 無機填充材

配比：W/G ~ 36-40%

5 kg包裝填充材 +

1900±100 cc的水

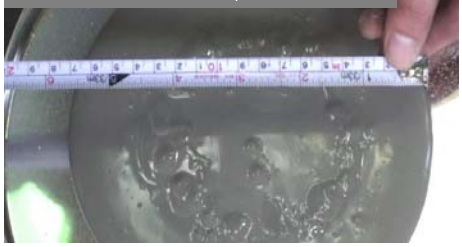


水泥砂漿拌合程序與流度試驗

於附件鐵桶內加入**1900cc±100cc**的清水。

再加入高強度無收縮水泥砂漿粉末，使用攪拌機拌合**3分鐘**。

高強度無收縮水泥砂漿流度須在**120mm~200mm**之間。



拌合完成後可利用鐵蓋施作流度試驗。

圓柱試體製作與注射準備

高強度無收縮水泥砂漿圓柱試體製作分兩層，每層搗實25下。

確實將氣泡釋出，再以塑膠袋封模。

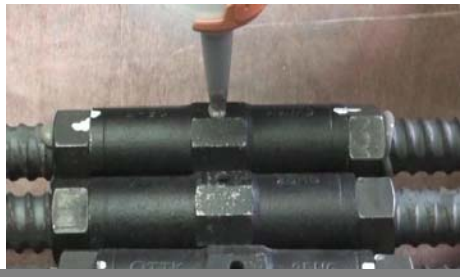
可利用細鐵絲夾於管壁，以利塞上蓋子時排氣。

之後，將高強度無收縮水泥砂漿倒入廠商所附之注射器管中。

注射水泥砂漿於鋼筋續接器

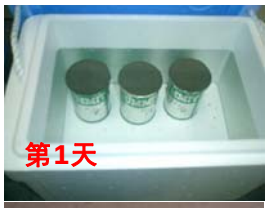
注射器管前端可旋下利用其尖端將管內保護膜刺破，再將其組合回去。

注射前先將其前端空氣排出。



使用注射器將高強度無收縮水泥砂漿注入續接器中至兩旁螺絲滿漿流出即可。

東京鉄鋼之砂漿試體抗壓試驗結果



2.圧縮強度試験

(1)養生温度別圧縮強度(水グラウト比 40%)　(単位:N/mm²)

養生温度	0℃	20℃	40℃
材齢 1 日	4.1	60.2	63.6
材齢 3 日	43.4	70.6	72.7
材齢 7 日	60.2	76.9	79.5
材齢 14 日	66.9	82.7	88.3
材齢 28 日	72.6	87.6	92.4

注)赤線は圧縮強度の規格範囲を図示している。

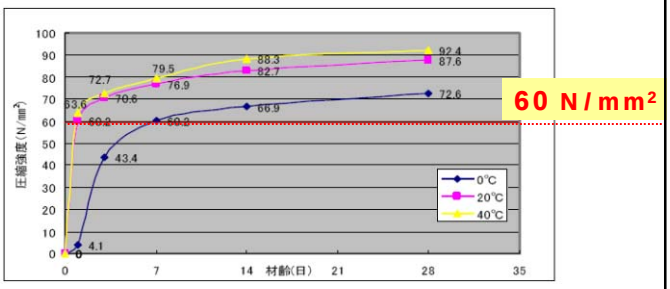


図 2.　養生温度別圧縮強度(水グラウト比 40%)

➢圓柱試體7天齡期強度須大於60 N/mm²

填充材圓柱試體抗壓試驗結果

齡期（天）	抗壓強度（N/mm²）	平均（N/mm²）
1	67.3	70.0
	69.9	
	72.8	
7	90.9	83.8 大於60 N/mm² OK!
	87.2	
	73.2	
28	86.6	92.8
	95.8	
	95.8	

養護溫度恆溫為20度。

TTK製品-螺紋節鋼筋續接器(預組預鑄用)

螺紋鋼筋砂漿填充續接器

ACE JOINT

SD390 ~ SD685

螺紋鋼筋樹脂填充續接器

EPOCH JOINT

SD390 ~ SD685

螺紋鋼筋砂漿填充續接器

FREE JOINT

SD390 ~ SD590

續接簡易

- ✓ 可以同一斷面續接
- ✓ 鋼筋未加工、無銲道
- ✓ 施工不受天候影響
- ✓ 施工容易、單人操作
- ✓ 手工具、不需要機具

TTK製品-砂漿填充式續接器(預鑄用)

TOPSJOINT

BOLTOPS

特點

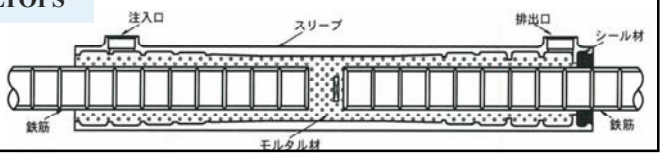
- 可與各種變型鋼筋接續
- 減少接續部分的重疊鋼筋
- 傾斜方式也可以接續
- TTK特製砂漿品質穩定

TOPSJOINT 預鑄用套管續接器

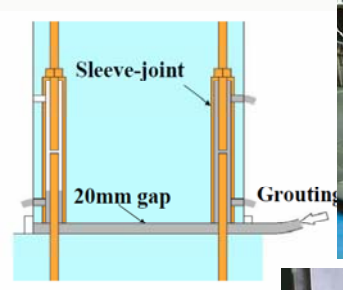
- 內部的凹凸設計，可接續不同的變型鋼筋，注入材為無收縮性TTK特製砂漿，簡易接續施工

BOLTOPS 螺栓固定砂漿填充續接器

- 允許誤差40mm，可以簡單接續各種變型鋼筋



TOPS JOINT 在預鑄柱之應用



SHIMIZU CORPORATION SHMZ

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

BOLTOPS 在預鑄梁之應用

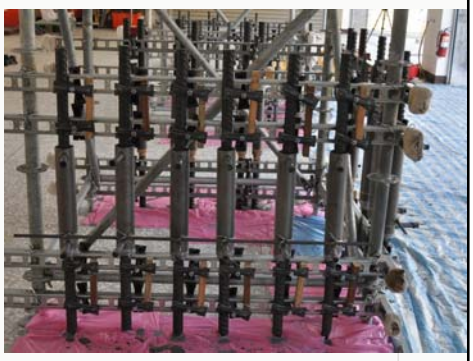
- 預鑄柱 + 預鑄梁下半部及接頭
- BOLTOPS套管續接避開梁塑鉸區

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

工地抽驗? 取樣? 工地組立試體!

➢ 預鑄或鋼筋預組工法之工地組立試體應採用同批材料依相同程序於工地現場同時組立試體。

ACE JOINT 試體
(水平 or 直立式注射)



TOPS JOINT 試體
(直立式壓力灌漿)

TTK製品-高強度無收縮水泥砂漿

for TOPS JOINT and BOLTOPS

Grout	拌合水/水泥 (kg / kg)	坍流度 (mm)	7日養護抗壓強度 (N/mm^2)
TTK 水泥砂漿H	2.8 / 25	220-260	120 以上
TTK 水泥砂漿150	2.8 / 25	220-260	150 以上

TOPS JOINT 試體

TTK 實驗室測試件固定架

YunTech 工廠測試件固定架
(鋼管施工架組搭)

- 續接套管實際施工容許偏心
- 但是試體兩段母材鋼筋若未對準造成軸線不重合(misalignment)可能影響試驗結果時應視為無效試體

TOPS JOINT 續接套管之壓力灌漿

簡報大綱

一. 規範背景與目的

二. 鋼筋機械式續接及錨定工法介紹

三. <u>續接器續接性能規範及測試</u>

四. 錨定頭規範及測試

五. 結語

http://www.concrete.org.tw/3931838913/tci-tci-hs001

鋼筋續接器性能等級

日本等級	續接性能評估基準	試驗項目
SA級	強度、變形、延展性及韌性等性能與母材鋼筋相近	4項 (標準較高)
A級	強度及變形與母材鋼筋相近，其他性能較母材鋼筋為劣	4項 (標準較低)
B級	強度與母材鋼筋相近，其他性能較母材鋼筋為劣	2項
C級	所有性能較母材鋼筋為劣	1項
我國現行	**續接性能評估基準**	**試驗項目**
SA級	強度、變形、延展性及韌性等性能與母材鋼筋相近	3項
B級	強度及變形與母材相近，其他性能較母材鋼筋為劣	2項

續接器性能分級與使用條件

TCI					ACI
續接性能	強度	變形能力	伸長率韌性	鋼筋可能降伏區	分類
SA級	○	○	○	**無條件使用**	第二類
SA級	○	○	*構件試驗*	**有條件使用**	第二類
B級	○			**禁止**	第一類

○ 表示性能與母材鋼筋相近

Type 1: 抗拉強度 $\geq 1.25 f_y$
Type 2: 抗拉強度 $\geq 1.25 f_y$ & f_u

雖然美國、日本以及我國相關規範，皆准許第二類機械式續接可以使用於構材可能的降伏區，但前提是續接器施工品質要受到有效的監督和查驗。

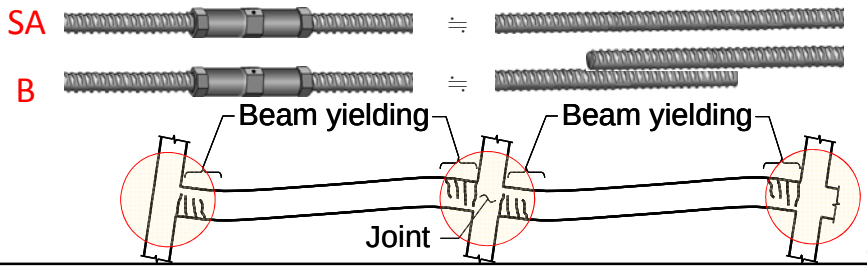
本規範參考日本規範訂定SA級續接允許使用於任何位置，並沒有錯置之規定。B級續接則不允許使用於鋼筋可能降伏區域。

高強度鋼筋續接器續接性能規範

- **第一章 通則**
  - 1.1 適用範圍
  - 1.2 用語釋義
  - 1.3 符號說明
- **第二章 鋼筋續接器續接性能評估基準**
  - 2.1 鋼筋續接器續接性能等級與評估項目
  - 2.2 鋼筋續接器續接性能試驗項目
  - 2.3 試驗試體
  - 2.4 母材鋼筋拉力試驗
  - 2.5 強度、滑動量和伸長率之合格基準
  - 2.6 鋼筋續接器續接性能試驗方法
  - 2.7 鋼筋續接器檢驗
- **參考文獻**

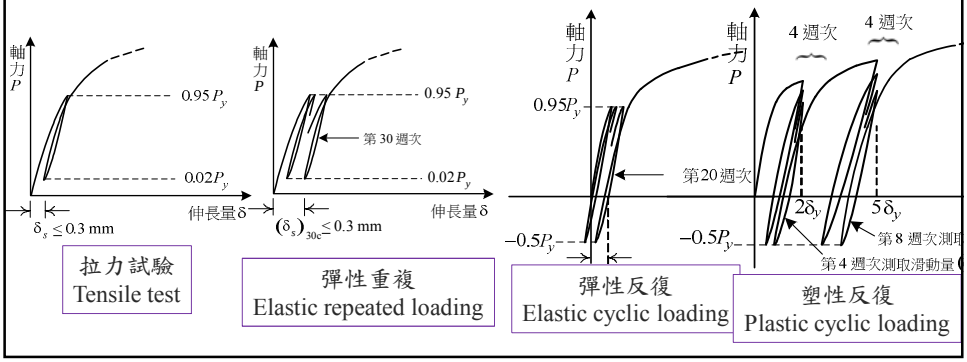
第一章 通則

- 本規範適用於鋼筋種類SD 550W及SD 685所規定鋼筋續接
- 依照設計圖說所規定之鋼筋續接器續接性能等級，選用合適之續接器型式。
- 續接器與續接器或續接器與鋼筋之淨距，不得小於混凝土粗粒料標稱最大粒徑之1.33倍，且提供續接施工所需之空間。
- 耐震構架內之構材，其縱向鋼筋採用鋼筋續接器續接時，B級鋼筋續接器續接不得使用於梁、柱接頭面或地震時鋼筋可能降伏處起算兩倍構材深度範圍內。SA級鋼筋續接器續接則准許使用於任何位置。



2.2 鋼筋續接器續接性能試驗項目

續接器接合試體					母材鋼筋
續接性能	拉力試驗	彈性重複載重試驗	彈性反復載重試驗	塑性反復載重試驗	拉力試驗
SA級	○		○	○	○
B級	○	○			○



拉力試驗
Tensile test

彈性重複
Elastic repeated loading

彈性反復
Elastic cyclic loading

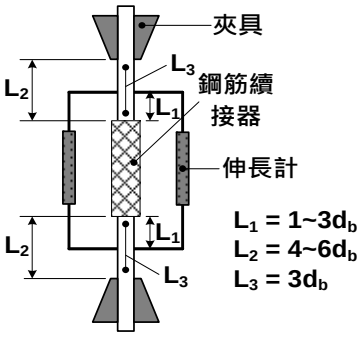
塑性反復
Plastic cyclic loading

2.3 試驗試體

- 鋼筋續接器續接性能試驗所用之試體，必須依據實際使用之材料與施工方法製作。
- 所有試體須使用性質相近之同一批次鋼筋，稱之為母材鋼筋。
- 母材鋼筋拉力試驗之試體為被續接之鋼筋；其餘試驗項目之接合試體，係以鋼筋續接器續接兩段母材鋼筋。
- 各試驗項目之試體數量須能代表該型續接器實際續接之平均性能，且至少 3 個試體為一組，取其平均值。
- 評估試體強度時，取 3 次試驗值之中最小值為其強度。評估滑動量及伸長率時，取 3 次試驗值之平均值。
- 接合試體在進行載重試驗前不得預拉。進行試驗時應先預拉至標稱零載重將伸長計讀數歸零後再開始加載，標稱零載重不得超過 4 N/mm^2 乘以鋼筋之標稱斷面積。

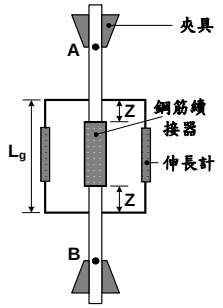
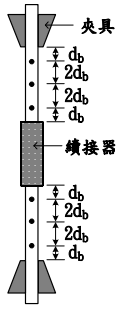
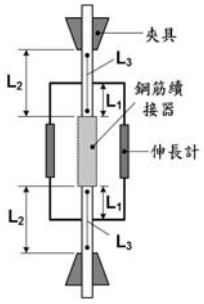
2.6 鋼筋續接器續接性能試驗方法

- 接合試體之軸向變形量測精確度至少須符合CNS 13723金屬材料拉伸試驗用伸長計標準等級2之規定
- 軸向變形量之檢測長度為自續接器兩端向外各加1倍至3倍鋼筋標稱直徑為原則
- 若檢測長度小於50 cm，則在無受壓挫曲之疑慮時最大可取50 cm，亦可用其他適當的替代方法直接量測續接器與鋼筋之滑動量
- 試驗裝置應避免試體受到軸壓力而挫曲。如發生試體挫曲之現象，該試驗視為無效而非試體不合格



- 伸長率：試驗前使用適當之衝子或劃線針於續接器兩側之鋼筋上分別標示兩個標記點，標記點距離以3倍至5倍鋼筋標稱直徑為原則，於拉力試驗破壞後分別量測標記點之間長度變化量，求得兩側鋼筋實際伸長率，取較大者。

韌性指標之比較

韌性指標			
	接合試體之延展性或伸長率（ε_{dc} or ε_{uc}）	接合試體之母材鋼筋直徑收縮率(R)	接合試體之母材鋼筋伸長率
優缺點	● ε_{dc} or ε_{uc}會包含滑動量。 ● 破斷位置會影響延展性。 ● 量測延展性，伸長計必須全程架設。	● 鋼筋拔出，則兩段鋼筋R皆需量測取大，量測繁瑣。 ● 破斷位置落於R量測點時，人為量測誤差大。 ● 鋼筋拔出時，量測得到R遠不及規範要求，另一形式要求必須為母材破斷。	● 實驗室易於執行。 ● 兩側母材鋼筋伸長率取大值，適用於各種破壞模式及異徑及異等級續接。 ● 量測完滑動量，即可拆掉伸長計，降低試驗風險及費用。

2.5 強度、滑動量和伸長率之合格基準

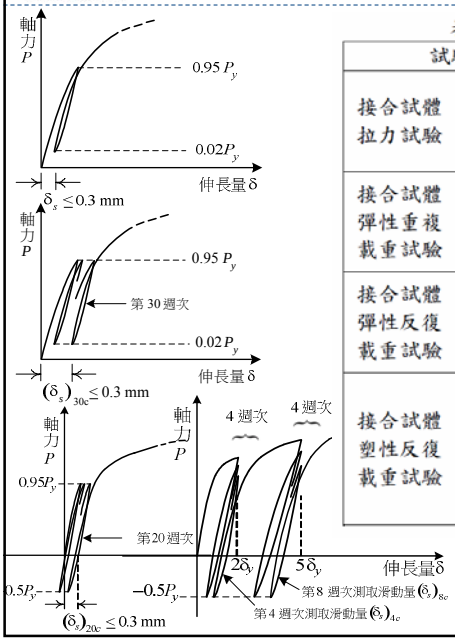
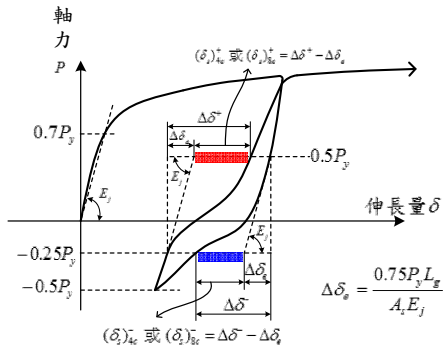


表 2.5.1 鋼筋續接器續接性能合格基準

試驗項目		SA 級	B 級
接合試體拉力試驗	抗拉強度 f_{uc}	$\ge 1.25 f_y$ 且 $\ge f_u$	$\ge 1.25 f_y$ 且 $\ge f_u$
	滑動量 δ_s	≤ 0.3 mm	≤ 0.3 mm
	伸長率 ε_{uc}	≥ 0.06	≥ 0.02
接合試體彈性重複載重試驗	抗拉強度 f_{uc}	---	$\ge 1.25 f_y$ 且 $\ge f_u$
	滑動量 $(\delta_s)_{30c}$		≤ 0.3 mm
	伸長率 ε_{uc}		≥ 0.02
接合試體彈性反復載重試驗	抗拉強度 f_{uc}	$\ge 1.25 f_y$ 且 $\ge f_u$	---
	滑動量 $(\delta_s)_{20c}$	≤ 0.3 mm	
	伸長率 ε_{uc}	≥ 0.06	
接合試體塑性反復載重試驗	抗拉強度 f_{uc}	$\ge 1.25 f_y$ 且 $\ge f_u$	---
	滑動量 $(\delta_s)_{4c}$	≤ 0.3 mm	
	滑動量 $(\delta_s)_{8c}$	≤ 0.9 mm	
	伸長率 ε_{uc}	≥ 0.06	

塑性反復載重測試 滑動量計算公式

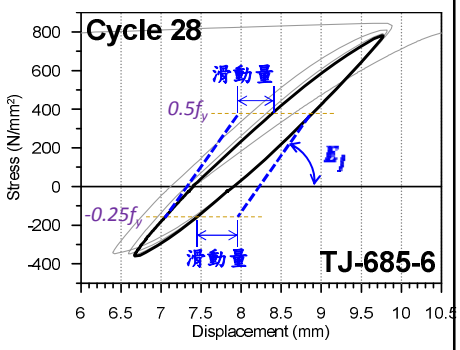
- 取載重在鋼筋規定降伏強度下限值50%拉力至25%壓力之間，由拉至壓及由壓至拉之相對軸向變形量扣除接該試體彈性變形後取平均值為當次滑動量
- 彈性變形以試體加載至鋼筋規定降伏強度下限值之70%之割線彈性模數計算



$$\Delta\delta_e = \frac{0.75 P_y L_g}{A_s E_j}$$

SA級續接器---第4週次 $(\delta_s)_{4c} = \frac{(\delta_s)^+_{4c} + (\delta_s)^-_{4c}}{2} \le 0.3\ mm$

第8週次 $(\delta_s)_{8c} = \frac{(\delta_s)^+_{8c} + (\delta_s)^-_{8c}}{2} \le 0.9\ mm$



試驗例

2.7 鋼筋續接器檢驗

- 鋼筋續接器施工前應先按本章規定執行一組續接性能試驗判定為SA級或B級。
- 鋼筋續接器施工期間之量產試驗數量及頻率可由買賣雙方約定之或按表2.7.1規定辦理。
- 取樣送驗結果不符合規定，加倍取樣複驗至合格，否則拒收。

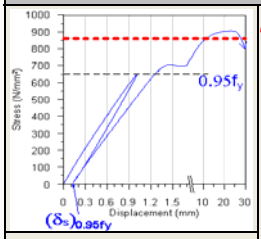
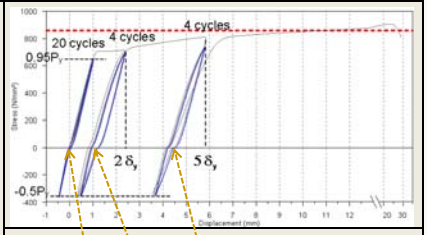
表2.7.1 鋼筋續接器施工現場取樣之參考頻率

續接器數量	接合試體拉力試驗
第一階段-2000個以內	每200個取樣1個已組裝試體以及1個工地組立試體，且至少須各取樣3個
第二階段-第2001個起	每300個取樣1個已組裝試體以及1個工地組立試體。
若取樣試驗結果有複驗或拒收情形發生，取樣頻率應回到第一階段。 已組裝試體：在工地自已完成加工之鋼筋及續接器中抽樣，並在工地依標準程序完成組裝之試體。 工地組立試體：在工地自已組立之鋼筋籠切下之續接器接合試體。預鑄或鋼筋預組工法之工地組立試體應採用同批材料依相同程序於工地現場同時組立固定。	

註：預鑄或鋼筋預組工法之工地組立試體應採用同批材料依相同程序於工地現場同時組立試體。

SD685 鋼筋續接器性能測試

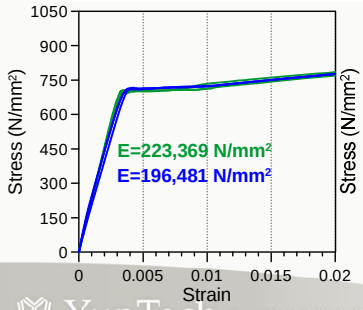
SD685鋼筋續接器性能測試(2013)

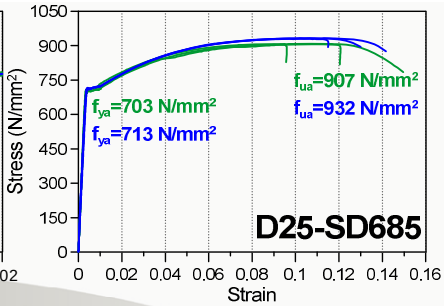
Tensile test		Elastoplastic cyclic loading test
	TS≧1.25f_y & f_u 20ε_y ductility or 4% elongation	
Slip≦ 0.3 mm		Slip≦ 0.3 0.3 0.9 mm

Grade	Couplers and Sleeves	Nov	Dec	Jan	Feb	Mar
SD685 D25	ACE JOINT 	●				
	EPOCH JOINT 	●				
	TOPS JOINT 	X	●			

SD685-D25母材鋼筋拉伸試驗

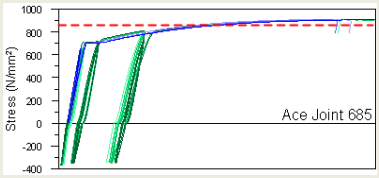
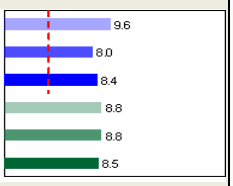
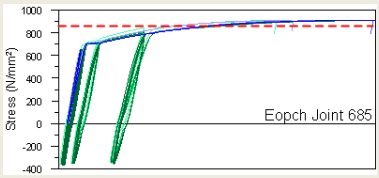
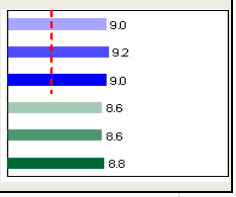
Grade, Size	Yield Strength (N/mm²)	Tensile Strength (N/mm²)	Elongation (%)	TS/YS	達降伏強度上限之應變
Criteria	685~785	≧860	≧10	≧1.25	≧0.014
SD685,D25	703	907	16	1.29	0.022
SD685,D25 (Retest)	713	932	15	1.31	0.0216





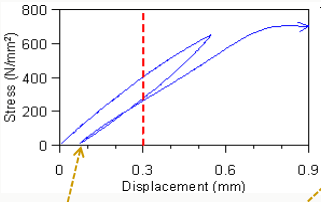
Tensile tests

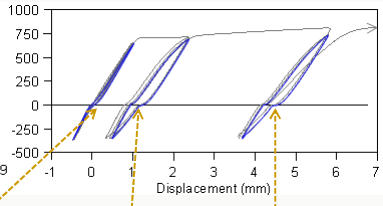
參考規範：TCI高強度鋼筋續接器續接性能規範+日本繼手指針

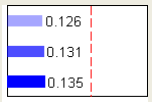
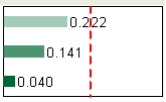
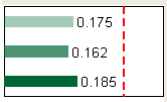
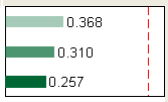
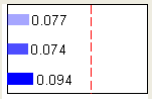
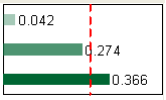
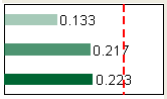
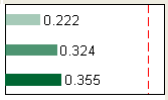
SA-Coupler Criteria	Tensile Strength ≧ f_u & 1.25f_y	Elongation ≧ 6%
ACE JOINT 685 (螺紋鋼筋砂漿填充續接器) 		
EPOCH JOINT 685 (螺紋鋼筋樹脂填充續接器) 		

✓ 抗拉強度及伸長率符合SA級要求

SD685 續接器滑動量(Slip)



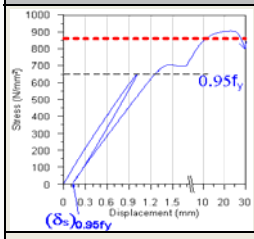
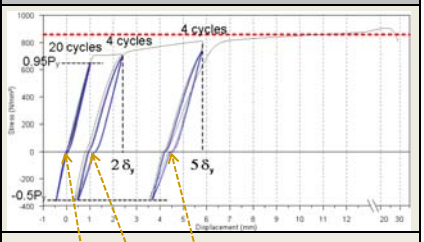


SA-Coupler Criteria	Tensile test	Elastoplastic cyclic loading test		
	$(\delta_s)_{0.9fy}$ ≦ 0.3 mm	$(\delta_s)_{20c}$ ≦ 0.3 mm	$(\delta_s)_{24c}$ ≦ 0.3 mm	$(\delta_s)_{28c}$ ≦ 0.9 mm
ACE JOINT 685 (螺紋鋼筋砂漿填充續接器) 				
EPOCH JOINT 685 (螺紋鋼筋樹脂填充續接器) 				

ACE JOINT 試驗前後

EPOCH JOINT 試驗前後

SD685鋼筋續接器性能測試(2013)

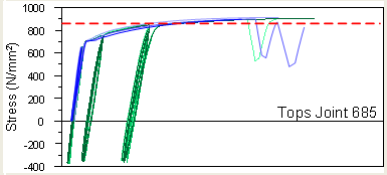
Tensile test		Elastoplastic cyclic loading test
	TS≧1.25f_y & f_u 20ε_y ductility or **4%** elongation	
Slip≦ 0.3 mm		Slip≦ 0.3 0.3 0.9 mm

Grade	Couplers and Sleeves	Nov	Dec	Jan	Feb	Mar
SD685 D25	ACE JOINT 	●				
	EPOCH JOINT 	●				
	TOPS JOINT 	X	●			

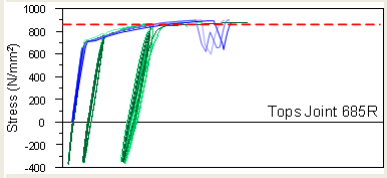
參考規範：TCI高強度鋼筋續接器續接性能規範+日本繼手指針

SA-Coupler Criteria	Tensile Strength ≧ f_u & 1.25f_y	Elongation > 6%

TOPS JOINT 685
(預鑄用套管式續接器)

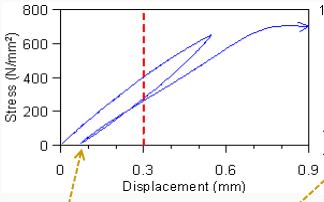


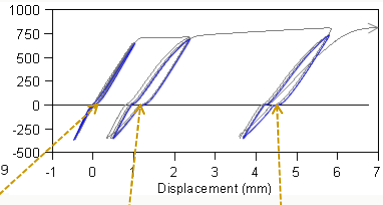
TOPS JOINT 685 Retest
(預鑄用套管式續接器)



Tensile Strengths > f_u=860 MPa & 1.25f_y

Bar pullout failure





SA-Coupler Criteria	Tensile test	Elastoplastic cyclic loading test		
	$(\delta_s)_{0.9}$ ≦ 0.3 mm	$(\delta_s)_{20c}$ ≦ 0.3 mm	$(\delta_s)_{24c}$ ≦ 0.3 mm	$(\delta_s)_{28c}$ ≦ 0.9 mm
TOPS JOINT 685 (預鑄用套管式續接器) 	0.342 0.123 0.144	0.299 0.380 0.307	0.107 0.201 0.257	0.297 0.145 0.393
TOPS JOINT 685 Retest (預鑄用套管式續接器) 	0.108 0.077 0.11	0.127 0.175 0.196	0.163 0.145 0.152	0.274 0.465 0.376


TOPS JOINT試驗前後


TOPS JOINT（Retest）試驗前後

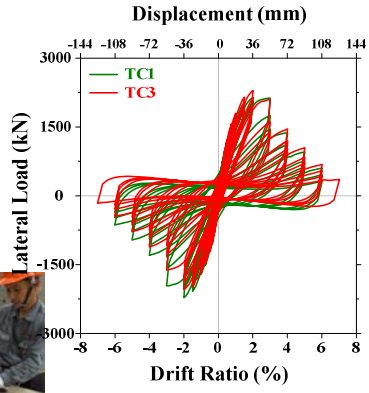
SD685鋼筋續接器性能測試結果(2013~2014)

Grade	Couplers and Sleeves	Nov	Dec	Jan	Feb	Mar	Apr	判定
SD685 D25	ACE JOINT 	●						SA
	EPOCH JOINT 	●						SA
	TOPS JOINT 	X	●					A~SA

強度及滑動量符合SA級規定
且經構件試驗證明其韌性符合耐震性能要求，亦可視為SA級續接器，或稱之為「有條件的SA級」，仍可考慮使用於塑鉸區。

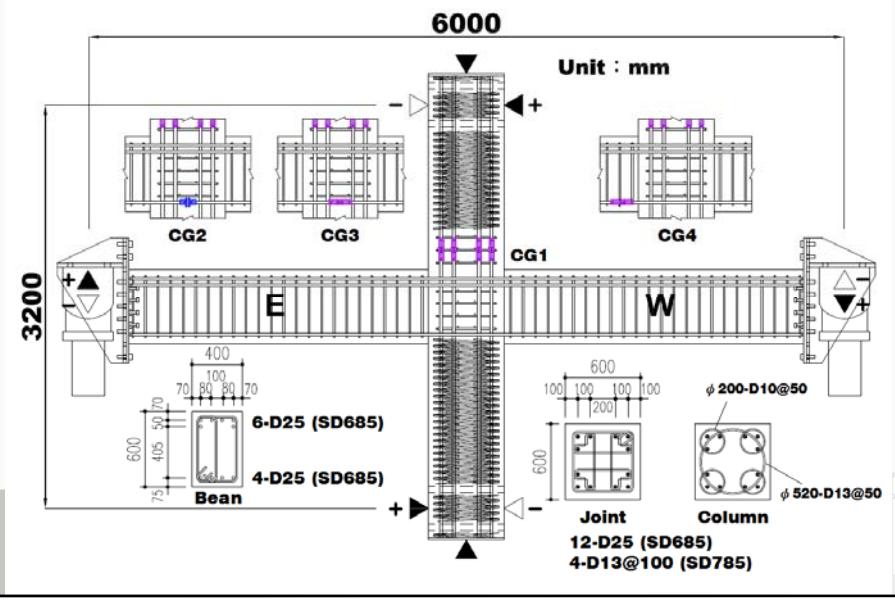
或經構件試驗證明其韌性符合要求

- 鋼筋續接器接合試體之強度及滑動量符合表2.5.1中SA級規定且經構件試驗證明其韌性符合耐震性能要求，亦可視為SA級續接器。

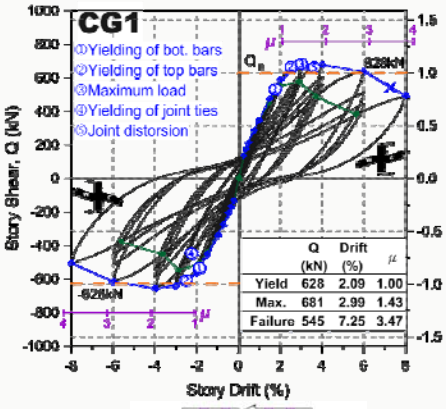


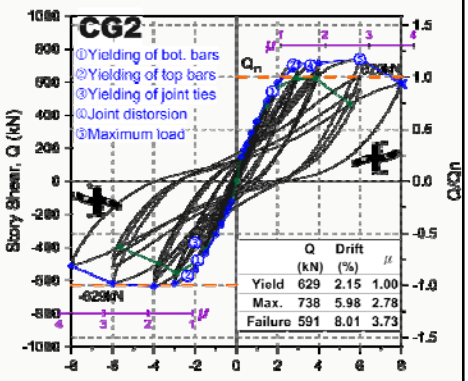
(台科大歐昱辰教授提供)

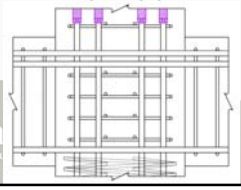
梁下層筋以不同型式續接錨定之預鑄梁柱接頭耐震性能-李宏仁等(2012)

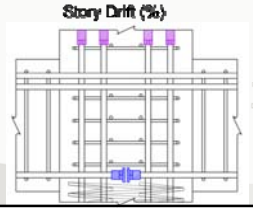


梁下層筋以不同型式續接錨定之預鑄梁柱接頭耐震性能-李宏仁等(2012)

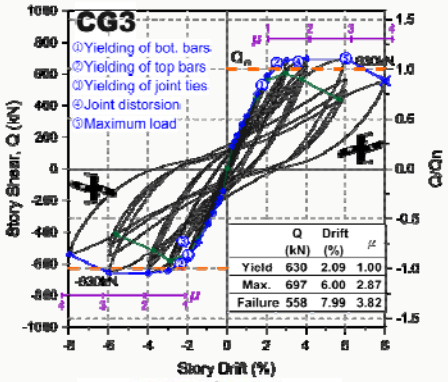


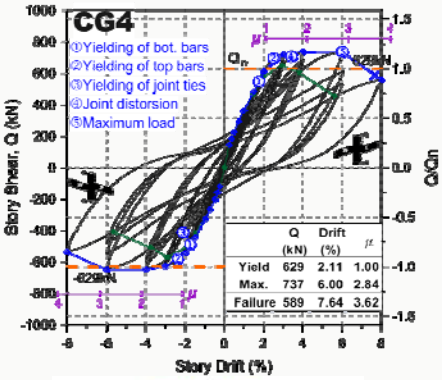


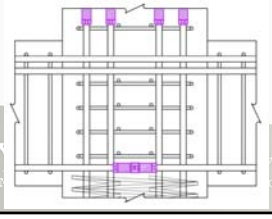


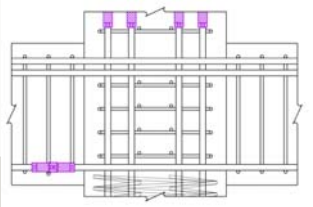


梁下層筋以不同型式續接錨定之預鑄梁柱接頭耐震性能-李宏仁等(2012)

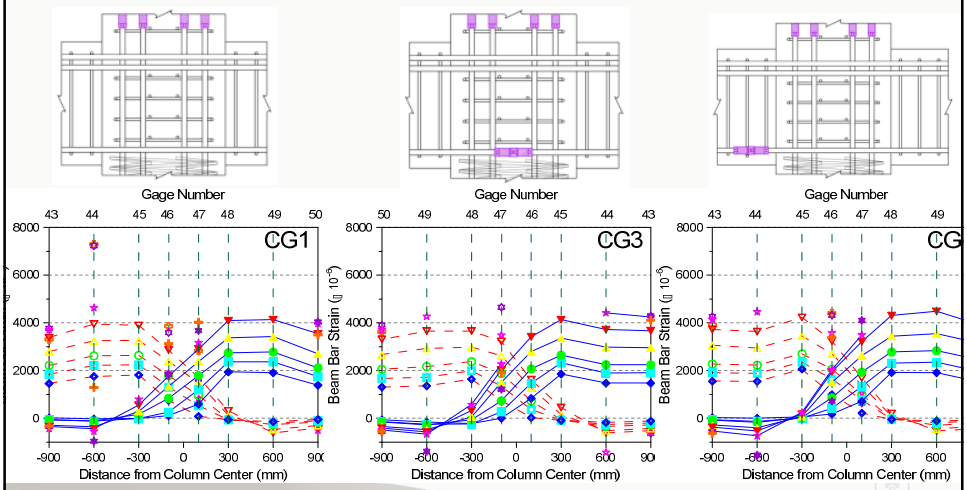








梁下層筋以不同型式續接錨定之預鑄梁柱接頭耐震性能-李宏仁等(2012)



彈性模數

續接器		E_{sa}	E_j	E_j/E_{sa}
ACE JOINT		226,076	216,844	0.96
EPOCH JOINT		226,076	224,174	0.99
TOPS JOINT		226,076	284,505	1.26
TOPS JOINT		212,084	297,543	1.40

簡報大綱

一. 規範背景與目的

二. 鋼筋機械式續接及錨定工法介紹

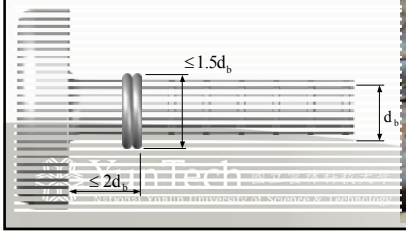
三. 續接器續接性能規範及測試

四. <u>錨定頭規範及測試</u>

五. 結語

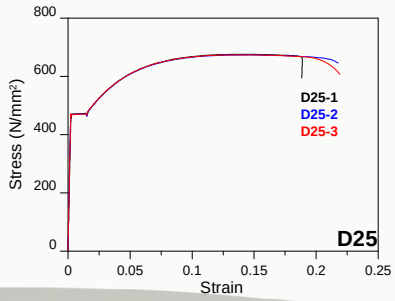
鋼筋錨定頭材料和製造

- 錨定頭與母材鋼筋之續接：摩擦銲接之母材鋼筋須符合CNS 560 標準之SD280W 或SD420W鋼筋，其它鋼筋不宜銲接，除非供應商經買方同意能提出相關試驗證明鋼筋銲接後性能符合要求
- 錨定頭之淨承壓面積不得少於鋼筋標稱剖面積之4 倍。錨定頭之淨承壓面積應由垂直鋼筋軸向之投影面積構成，鋼筋因摩擦銲接造成之表面隆起之障礙投影面積，應予以扣除。鋼筋表面之節因摩擦銲接而隆起之不平整範圍，距離承壓面不得超過2 倍鋼筋標稱直徑，直徑亦不得大於1.5 倍鋼筋直徑。



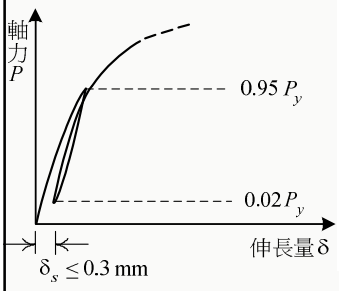
具錨定頭鋼筋之檢驗(1/2)

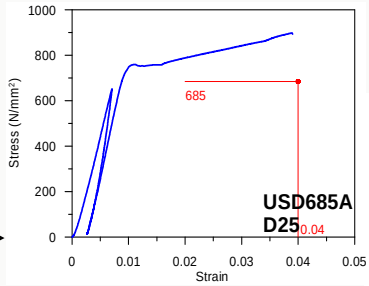
- 拉伸試驗
  - 每滿[300][　]個取樣[1個]，但各號數錨定頭鋼筋至少取樣[3個][　]
  - 其抗拉強度至少應達到鋼筋規定降伏強度下限值之1.25倍或鋼筋抗拉強度規定值。
  - 伸長率應達到母材鋼筋伸長率規定值以上。

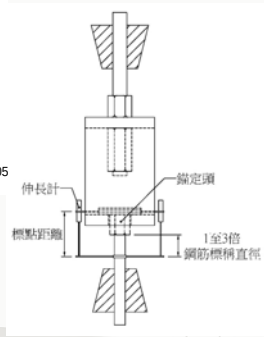


具錨定頭鋼筋之檢驗(2/2)

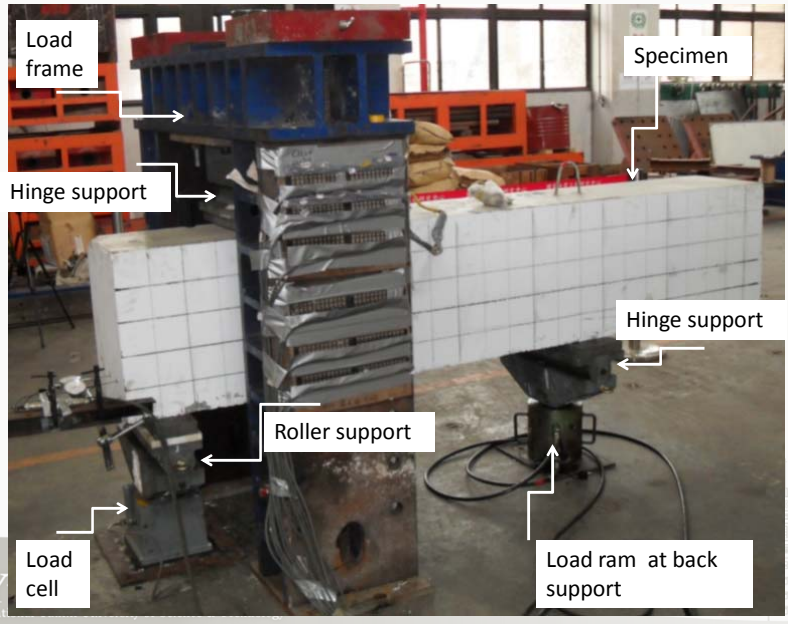
- 滑動量試驗
  - 以螺紋接合之錨定頭鋼筋於執行拉伸試驗時應同時量測接合部之殘留滑動量
  - 無滑動量疑慮之銲接或鍛造的錨定頭得省略滑動量試驗







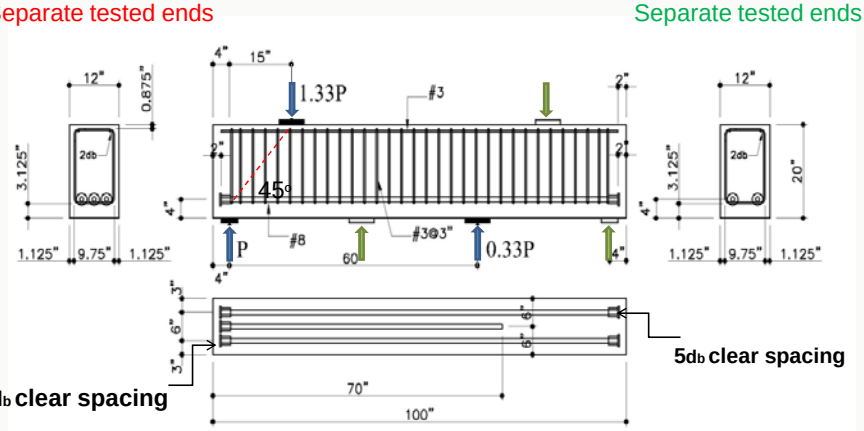
Test setup at NCREE



Confined Specimens (typ.)

Separate tested ends

Separate tested ends



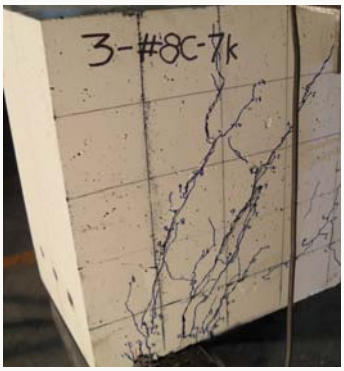
Effect of bar spacing

This comparison is true between **confined** specimens

Specimens	*f'c* (psi)	Peak load (kips)	Stress (*fs*) ksi	Bar area (As)	failure mode
3-#8C-7k	7073	326.0	137.5	2.37	Tie Yielding
2-#10C-7k	7189	301.9	122.7	2.46	Tie Yielding

Effect of bar spacing

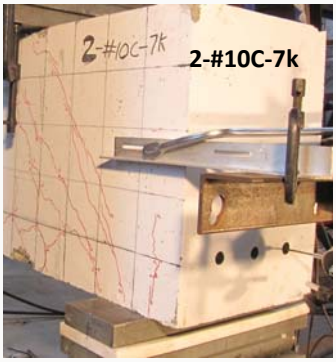
- *2db* clear spacing



Failed at higher load 326kips

(As 2.37 in²)

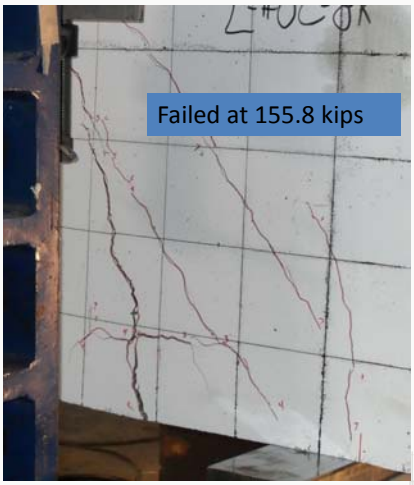
- *5 db* clear spacing



Failed at lower loads 301.9 kips

(As 2.46 in²)

Heads with or without grouting



Failure more of a pullout type, at ½ the load capacity of similar specimen

簡報大綱

一. 規範背景與目的

二. 鋼筋機械式續接及錨定工法介紹

三. 續接器續接性能規範及測試

四. 錨定頭規範及測試

五. 結語

Taiwan Concrete Institute

新高強度鋼筋混凝土技術委員會報告

主任委員：黃世建　國立台灣大學土木工程系　教授
執行幹事：李宏仁　國立雲林科技大學營建系 副教授
執行幹事：林克強　國家地震工程研究中心　研究員

2015/2/4

Taiwan Concrete Institute

- 97年，台大引進日本東京鐵鋼高強度鋼筋樣品
  少量鋼筋母材試驗，SA級續接器試驗，錨定頭試驗，梁柱接頭構件試驗
- 97-101年，TCI兩年期計畫與經濟部一年期科專計畫
  早先參考日本鋼筋規格及規範，建議前述三項材料規格及規範草案
  98年7月15日、10月30日召開二次專家座談會蒐集意見修訂部分內容
  98-99年間台灣鋼廠小爐試軋生產鋼筋SD685/785鋼筋
  NCREE 執行鋼筋母材試驗，續接器試驗，錨定頭試驗，構件試驗.....
- 100-101年，台灣鋼廠改造生產線到大量生產並不順利
  缺少量產品測試經驗,部分項目如鋼筋之韌性、續接器性能等仍有疑慮
  前述三項材料規格及規範草案 暫停保留2年 未提交委員會審查
- 102-103年執行國科會整合型計畫及日本東京鐵鋼委託國震中心執行之量產品及構件測試
  累積足夠之實驗數據，釐清諸多疑慮，更新前述三項材料規格及規範草案
- 103年提交高強度鋼筋及其附件之規範草案，交由本學會之技術委員會與規範委員會審議通過，公開參考使用。

審議委員會之組成

由本學會之技術委員會與規範委員會共同審議

職稱	姓名	職稱	姓名
技術委員會主任委員	李 釗	規範委員會主任委員	黃世建
技術委員會委員	侯琮欽	規範委員會委員	趙文成
技術委員會委員	干裕成	委 員	蔡益超
委 員	陳正誠	委 員	宋裕祺
委 員	陳國隆	委 員	陳奕信
委 員	王炤烈	委 員	夏沛禹
委 員	吳子良	委 員	何長慶

審查會議與書面審查

- 第一次審查會議：於6/10，假國家地震工程研究中心舉行
- 第二次審查會議：於6/30，假國家地震工程研究中心舉行
- 第三次審查會議：於7/22，假國家地震工程研究中心舉行
- 第四次審查會議：於8/13，假國家地震工程研究中心舉行
- 第一次書面審查：審查期間為8/18至9/1
- 第二次書面審查：審查期間為9/9至9/23

高強度鋼筋及其配件規範之出版

- 鋼筋混凝土用鋼筋 SD550W、SD685、SD785
- 高強度鋼筋續接器續接性能規範
- 鋼筋混凝土用鋼筋錨定頭規範


社團法人 台灣混凝土學會
Taiwan Concrete Institute

鋼筋混凝土用鋼筋
Steel bars for Concrete Reinforcement
－SD550W、SD685、SD785


社團法人 台灣混凝土學會
Taiwan Concrete Institute

高強度鋼筋續接器續接性能規範


社團法人 台灣混凝土學會
Taiwan Concrete Institute

鋼筋混凝土用鋼筋錨定頭規範

審查已於9月底完成，並已在學會網站公告，相關規範已可供外界下載

簡報結束
感謝聆聽

陳智軒

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

高強度鋼筋混凝土結構設計研討會 2014.12.05

高強度鋼筋混凝土構架接頭之耐震設計建議

李宏仁

Associate Professor at YunTech, Taiwan
Member of Joint ASCE-ACI Committee 352, Joints and Connections in Monolithic Concrete Structures & Task Group for High-strength materials in beam-column connections

Acknowledgements

國科會補助
台灣新型高強度鋼筋混凝土結構研發
子計畫：梁柱接頭耐震性能研究及資料庫建構

YUNTECH

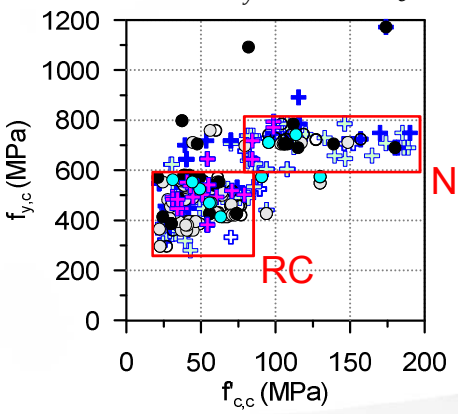
NCREE

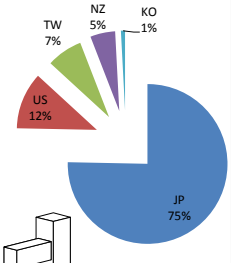
YunTech

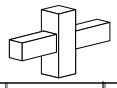
Database construction

- 357 test data published in Japan, US, NZ, and Taiwan
- Unified database for normal-strength and high-strength RC
- Beam-column joints without trans. beams/slabs, eccentricity

Column bar f_y versus f_c'



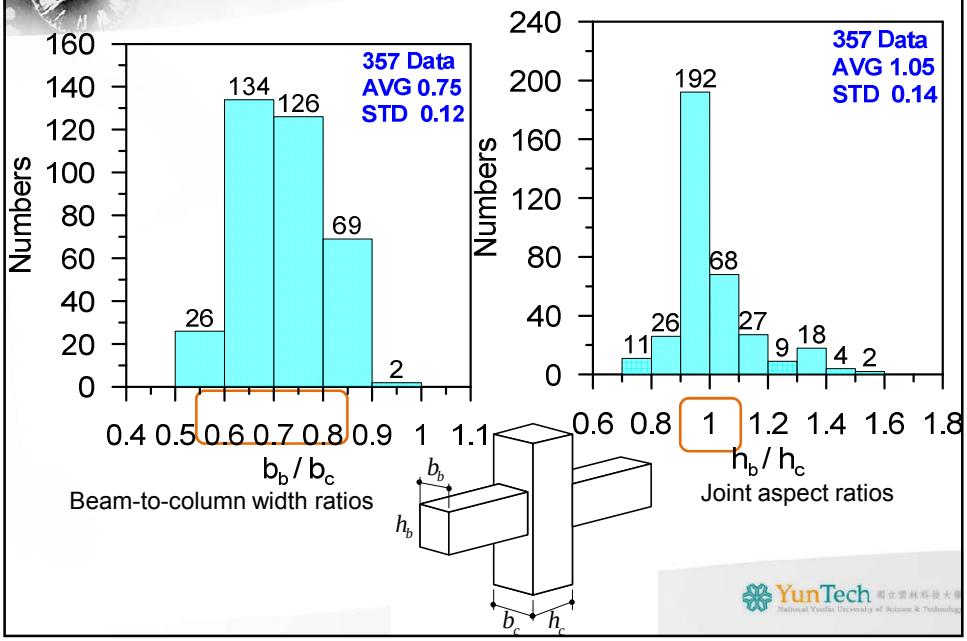




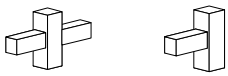
Type	Int.(202)	Ext.(155)
B	✚	○
BJ	✚	○
J	✚	●
BJ_a	✚	○

YunTech

Geometry of test data

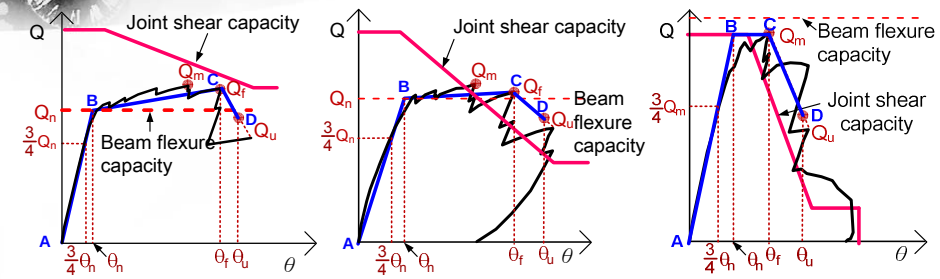


Test Data classified by failure modes



Failure mode	Interior	Exterior
Beam flexure failure (B-failure)	32	38
Joint shear failure occur with beam yielding (BJ-failure)	107	75
Joint shear failure occur without beam yielding (J-failure)	37	32
Anchorage failure of beam bar in the joint (BJa-failure)	26	10
Total	202	155

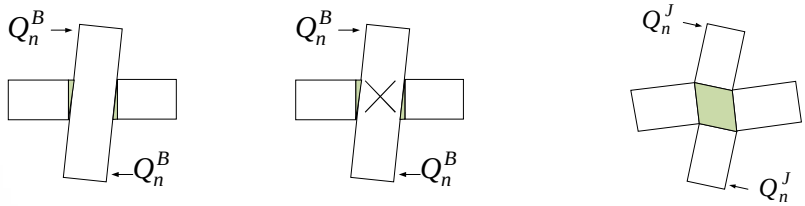
Failure modes and ductility



B-failure

BJ-failure

J-failure



Definitely well

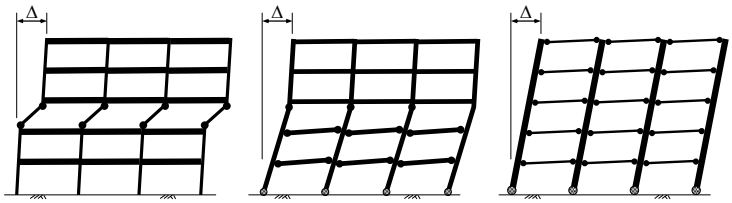
Okay with ductility

Not acceptable

Outline

- **Frame Mechanism – Avoid Story Mechanism**
- Hinge Development
  - Capacity design
  - Joint shear demand
- Joint Shear Design
  - Joint shear strength
  - Joint transverse reinforcement
- Development Length Requirements for Joints
- Concluding remarks

Frame Mechanisms
"strong column – weak beam"



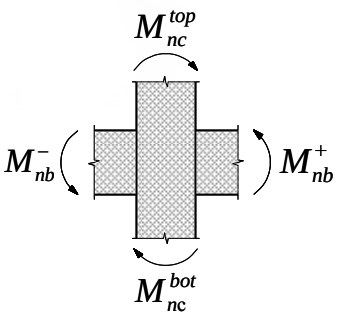
(a) Story Mechanism (b) Intermediate Mechanism (c) Beam Mechanism

- Avoid story mechanism
- Avoid joint failure

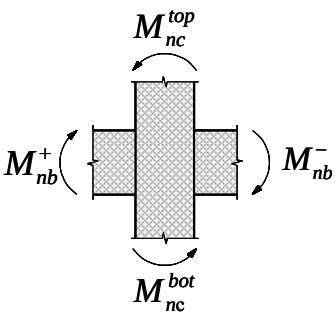
Inadequate confinement

Required Column Moment Strength

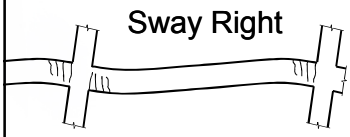
$$M_{nc}^{top} + M_{nc}^{bot} \geq \frac{6}{5}\left(M_{nb}^{+} + M_{nb}^{-}\right) \qquad M_R \geq 1.2$$

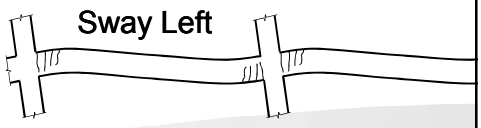


Sway Right



Sway Left

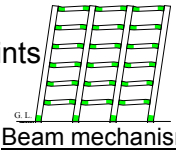




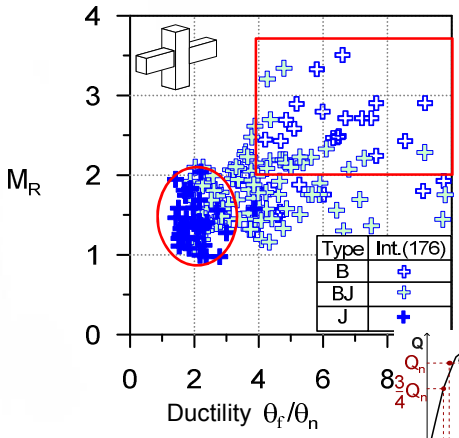
Figures courtesy of Jack P. Moehle

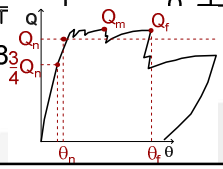
Moment Strength Ratio

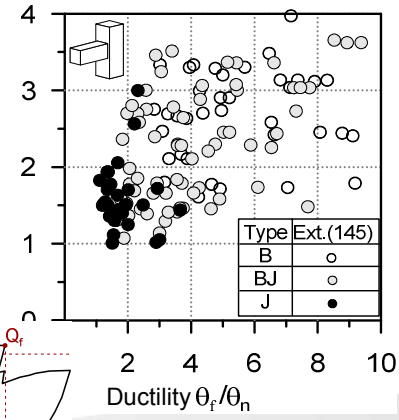
- Int. Joints, for M_R >2, B-failures for most of interior joints
- Ext. Joints, there is no conclusive result for M_R



Beam mechanism







Outline

- Frame Mechanism – Avoid Story Mechanism
- Hinge Development
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  - Joint shear demand
- Joint Shear Design
  - Joint shear strength
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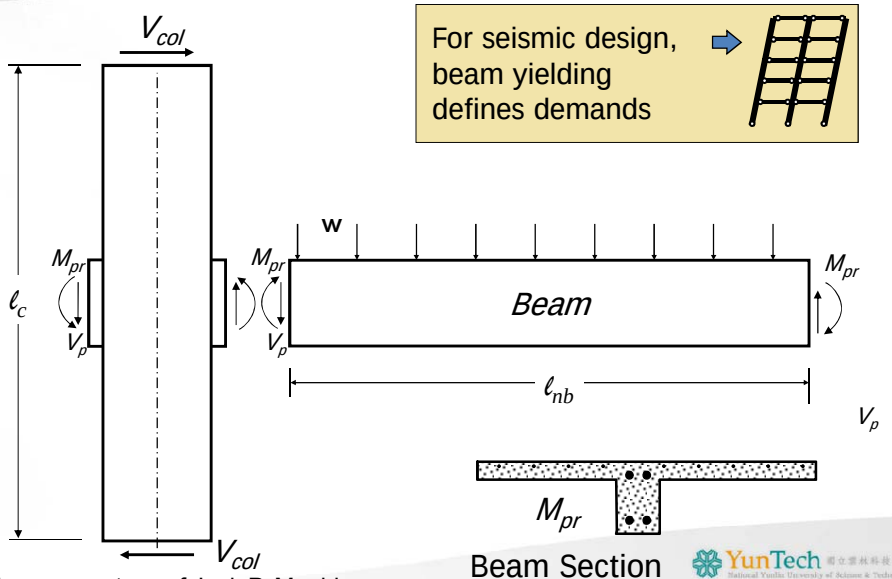
- Development Length Requirements for Joints
- Concluding remarks

Special Moment-Resisting Frames
- Probable Moment Capacity -

For seismic design, beam yielding defines demands

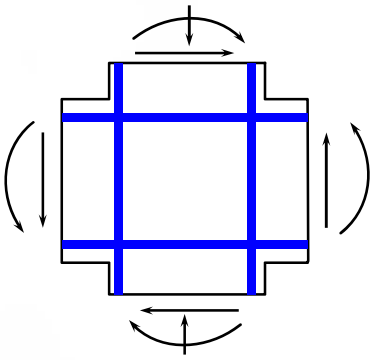


Figures courtesy of Jack P. Moehle

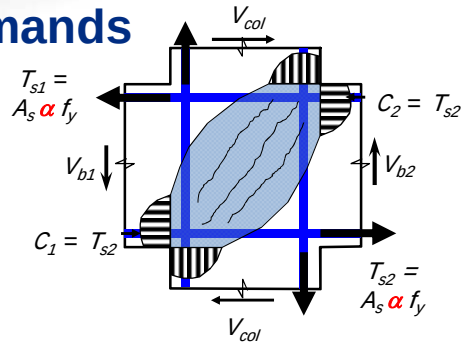
Joint shear demands

Typical α = 1.25 for SD420

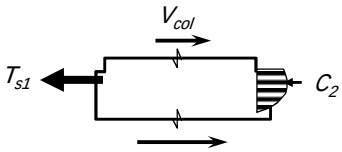
For SD685, α = ?



(a) moments, shears, axial loads acting on joint



(b) internal stress resultants acting on joint

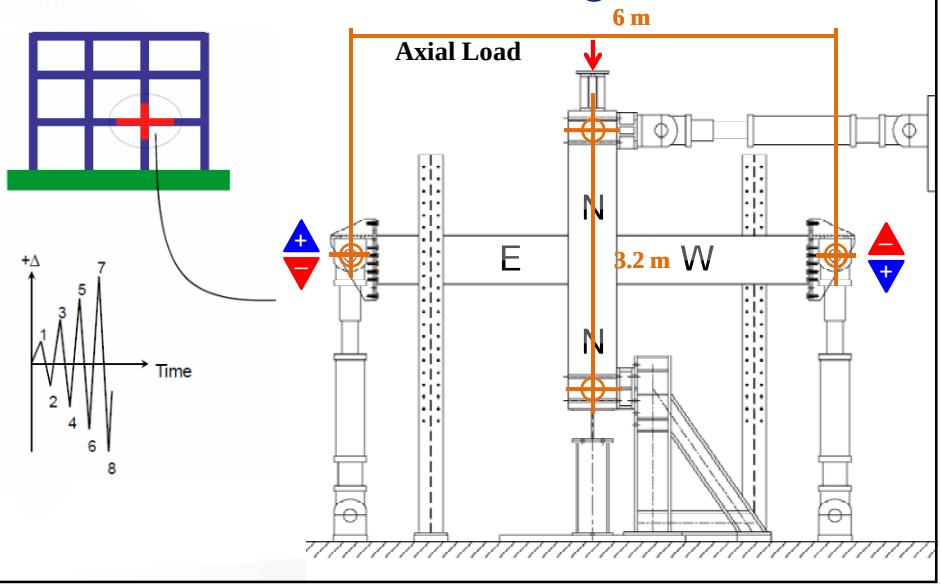


$V_u = V_j = Ts1 + C_1 - V_{col}$

(c) joint shear

Figures courtesy of Jack P. Moehle

Laboratory Specimen under Cyclic Deformation Loading



Laboratory testing

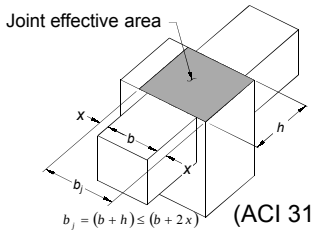
Outline

- Frame Mechanism – Avoid Story Mechanism
- Hinge Development
  - Capacity design
  - Joint shear demand
- **Joint Shear Design**
  - **Joint shear strength**
  - **Joint transverse reinforcement**
- Development Length Requirements for Joints
- Concluding remarks

Shear strength for confined joints

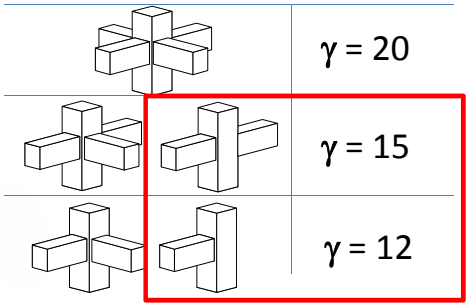
$$V_n = \gamma\sqrt{f_c'}b_j h_c \quad (f_c' \text{ in psi}) \text{ or}$$

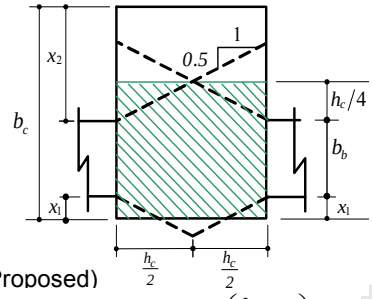
$$V_n = \frac{\gamma}{12}\sqrt{f_c'}b_j h_c \quad (f_c' \text{ in MPa})$$



(ACI 318)

γ-values for special frame joints



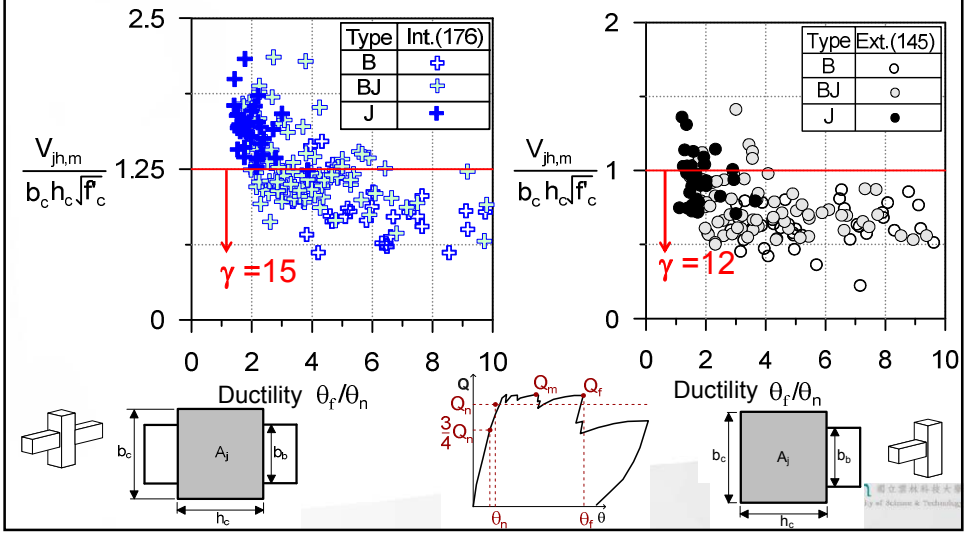


(Proposed)

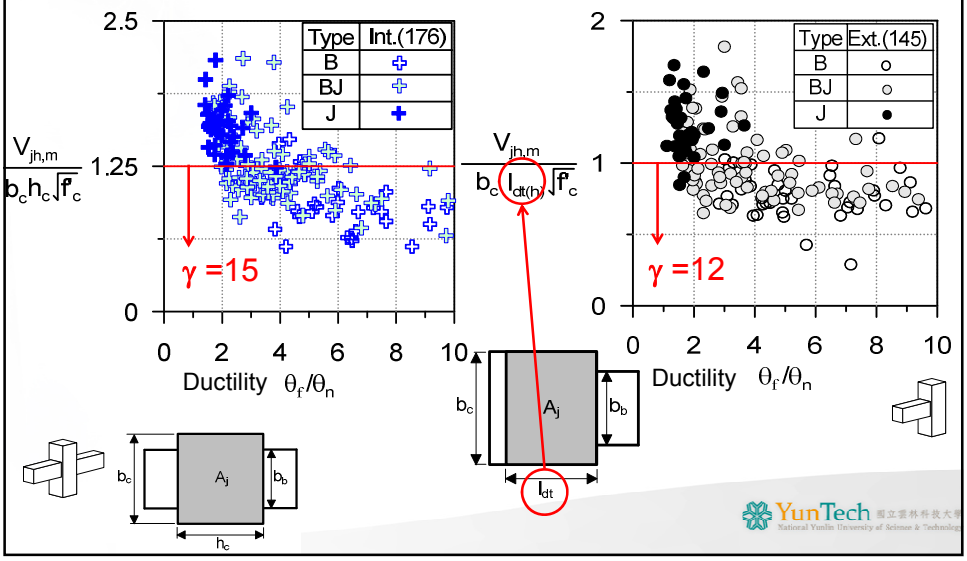
$$b_{j,pro} = b_b + \sum \min\left(\frac{h_c}{4}, x_i\right) \le b_c$$

Joint Shear Strength-Ductility Trends

- Experimental joint shear force, normalized to effective joint area and $\sqrt{f_c'}$ in MPa
- There is no J failures below the γ-value of 15 for interior joints, but quite a few below the γ-value of 12 for exterior joints. (To be evaluated later)

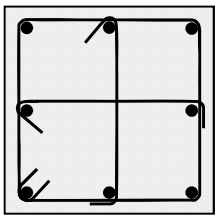


Change of Effective Joint Depth



Confinement of trans. reinforcement

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



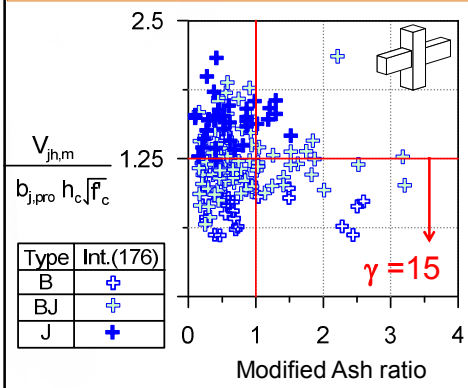
$$A_{sh,ratio} = \frac{A_{sh,provided}}{A_{sh,required}}$$

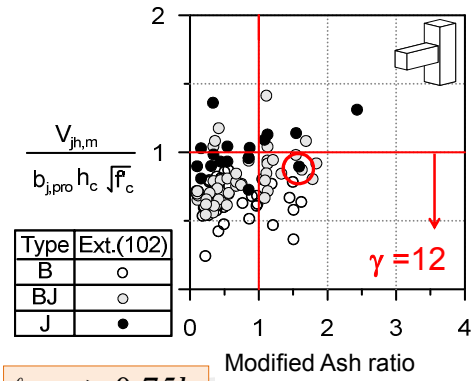
- "Ash ratio" is a key influence parameter (Kim and LaFave)
- Because the confining stress developed by transverse reinforcement depends on its stress-strain relations and configurations, a "modified Ash ratio" is used in this paper by taken a maximum yield strength of 800 MPa for transverse reinforcement.

$$\overline{f_{yt}} = \min(f_{yt}, 800) MPa$$

Confinement Requirements

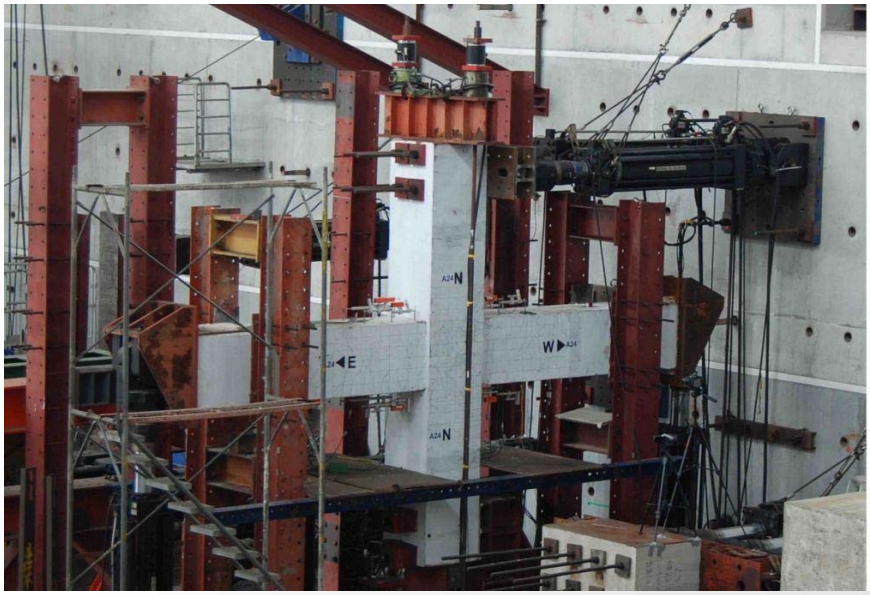
- Confinement reinforcement in the direction of loading may be reduced by 50% for the joints confined two opposite faces.
- Ash is adequate for exterior joints without transverse beams.



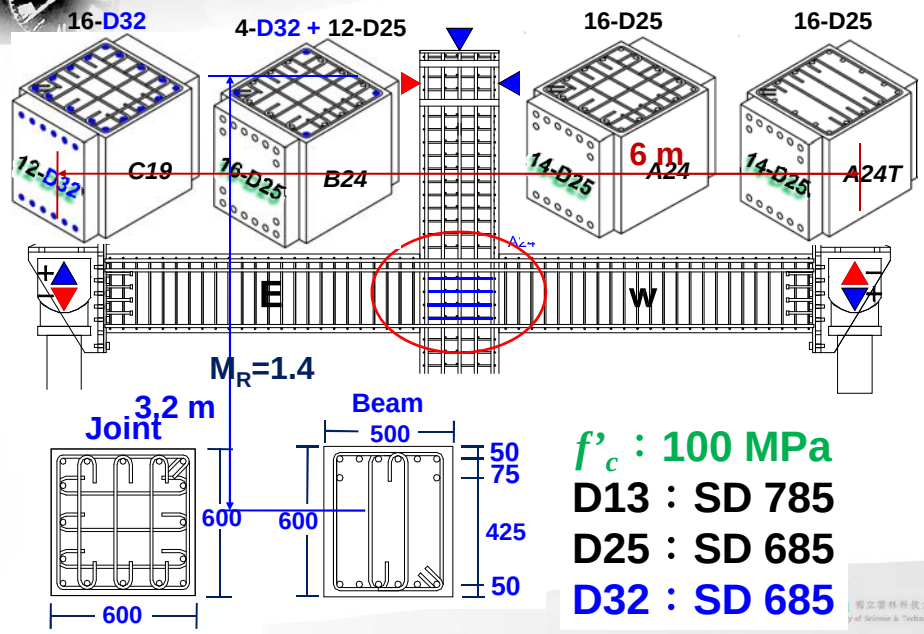


$$\ell_{dt(h)} \geq 0.75h_c$$

Seismic Testing for Interior Beam-Column Joints



Int. Joints tested in 2012-2013



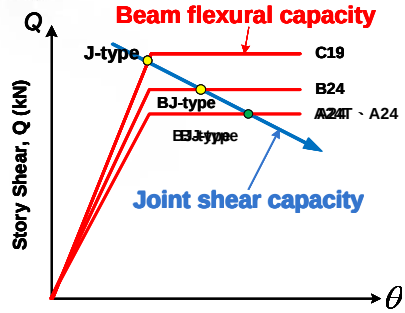
f'_c : 100 MPa

D13 : SD 785

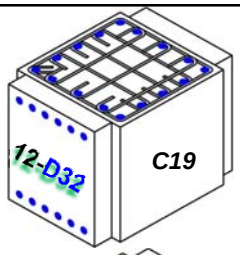
D25 : SD 685

D32 : SD 685

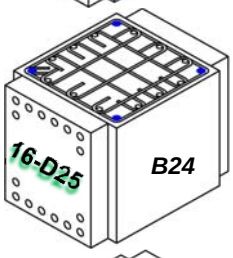
Test parameters

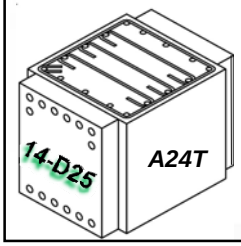


γ =21
h_c / d_b =19
$A_{sh,ratio}$ =0.9



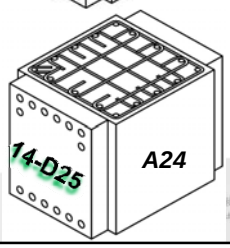
γ =18
h_c / d_b =24
$A_{sh,ratio}$ =0.9

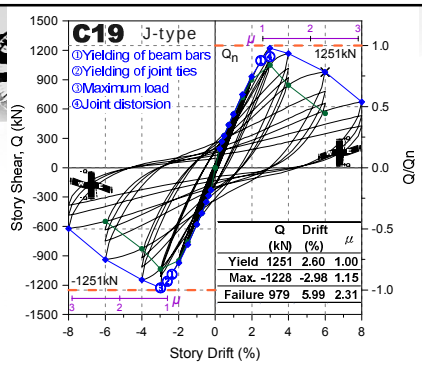


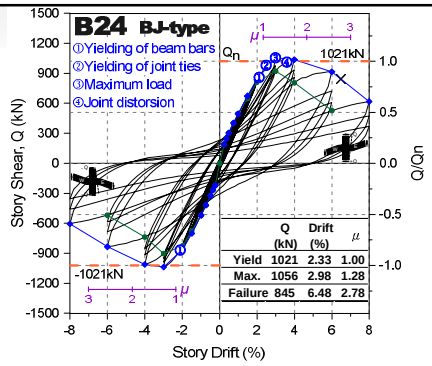


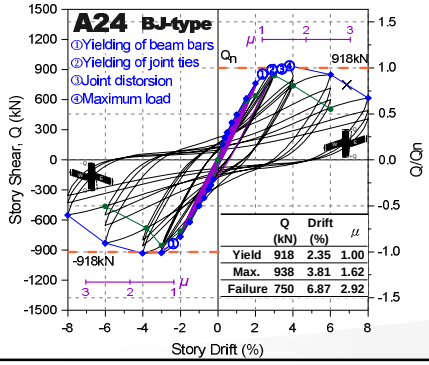
γ =15
h_c / d_b =24
$A_{sh,ratio}$ =0.4

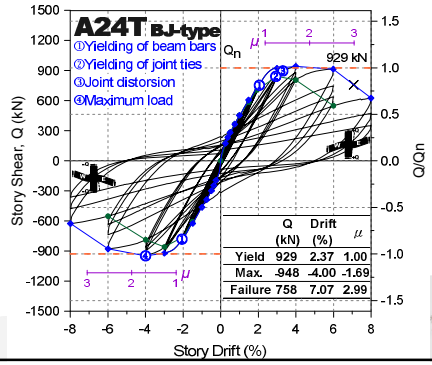
γ =15
h_c / d_b =24
$A_{sh,ratio}$ =0.9



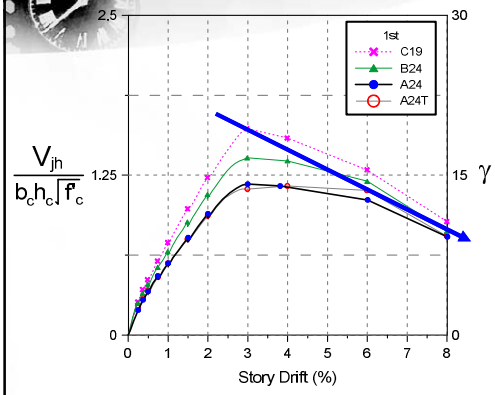


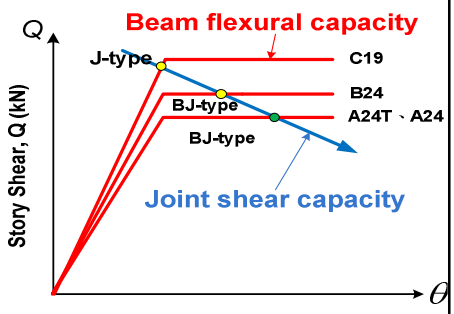






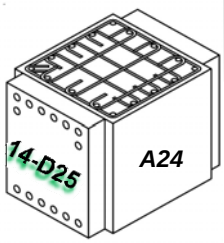
結論與建議

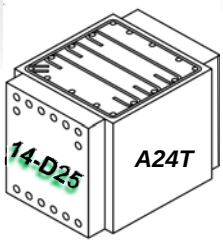


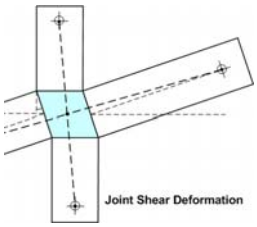


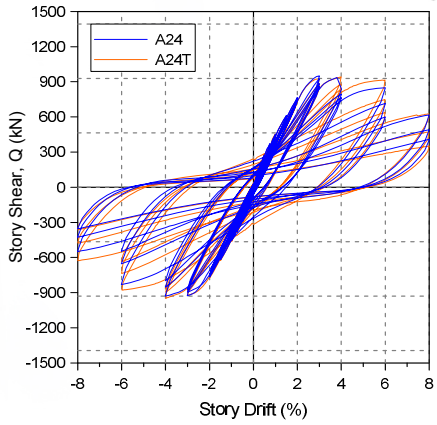
➢ 接頭剪力強度之衰減曲線是可以預測的

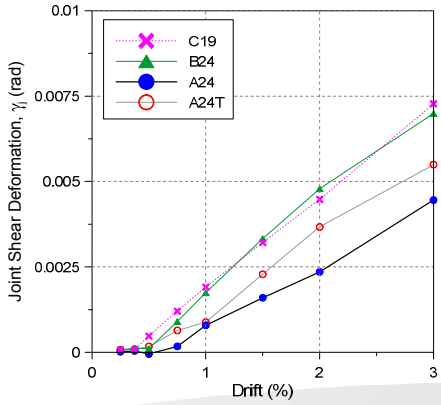
測試結果顯示現行ACI規範之構架接頭剪力計算強度公式沿用至高強度鋼筋混凝土構架接頭仍可以控制梁在2%層間變位產生塑鉸而接頭破壞在4%-6%層間變位



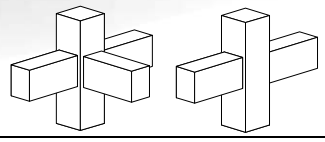
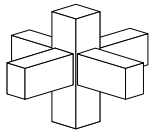




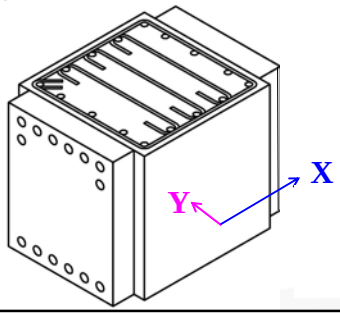




ACI 規範

	
雙向 **1.0 A_{sh}**	雙向 **0.5 A_{sh}**

本研究建議

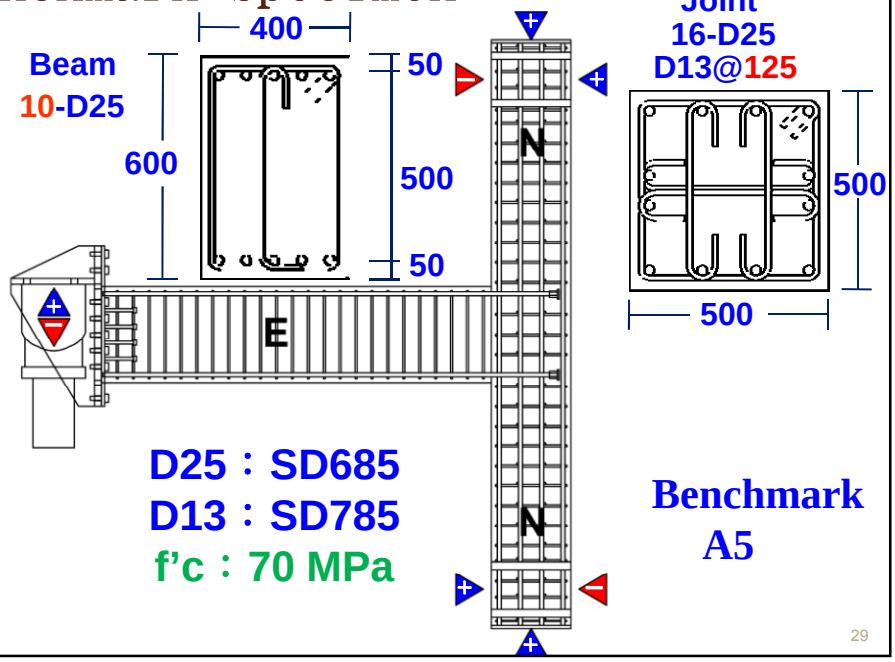


接頭兩對面接梁，接頭有效寬度內之繫筋量可以減少，但接頭箍筋維持與柱端相同之箍筋間距不放寬!

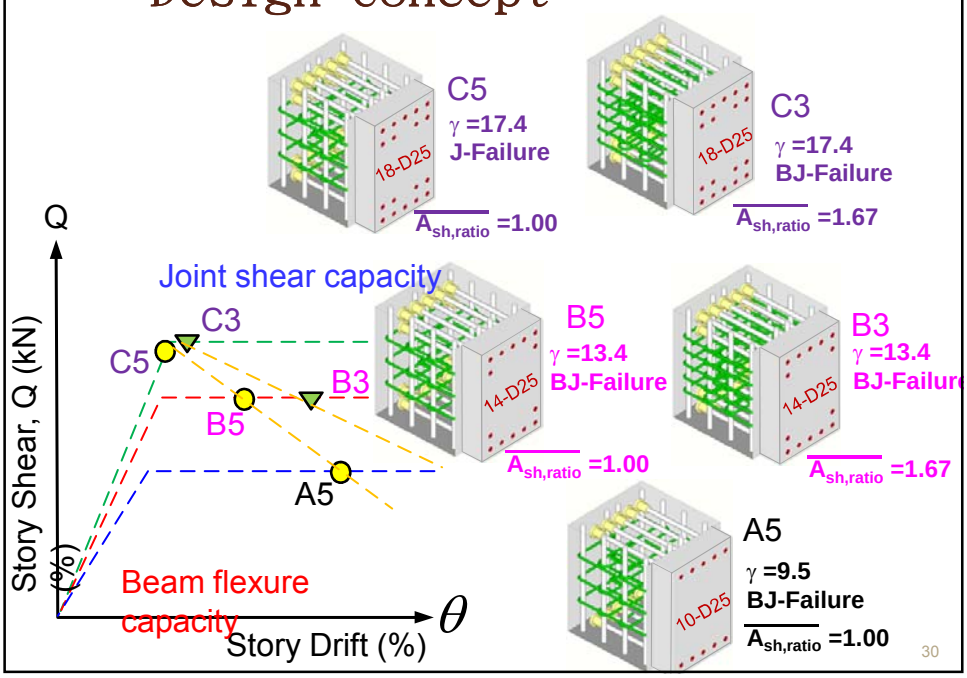
X、Y向 要求之 Ash 不同
垂直未圍束面才要求 1.0 Ash

Seismic Testing for Exterior Beam-Column Joints

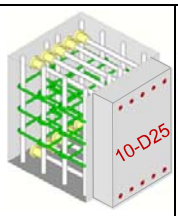
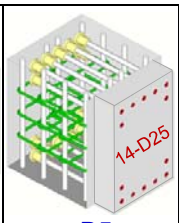
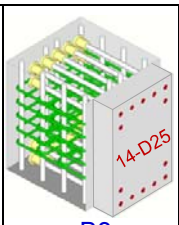
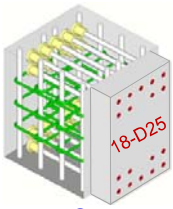
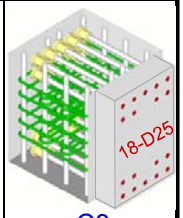
Benchmark specimen



Design Concept

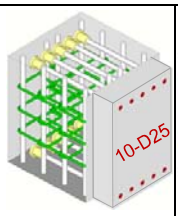


Connection Parameters

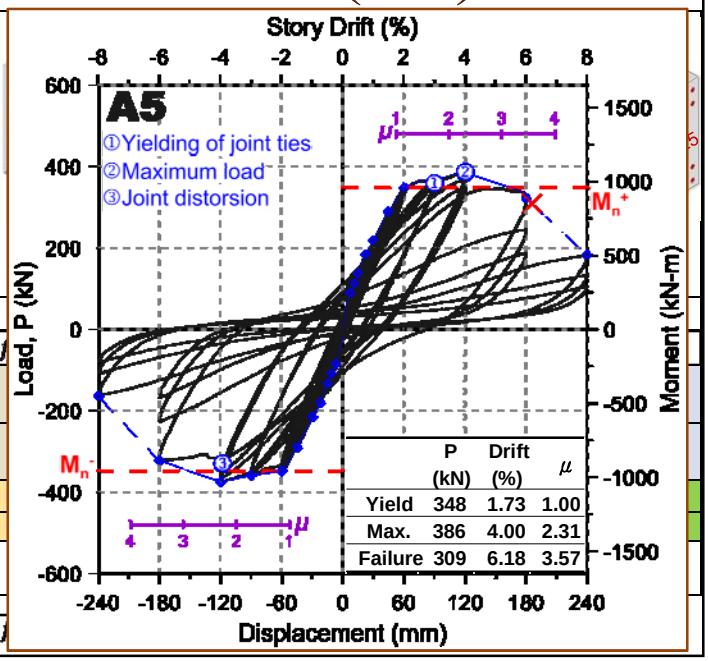
	A5	B5	B3	C5	C3
					
	D13@125mm	D13@125mm	D13@75mm	D13@125mm	D13@75mm
f'_c	70 (70) MPa	70 (64) MPa	70 (69) MPa	70 (70) MPa	70 (71) MPa
f_y = 685(735) MPa ; f_{yt} = 785(841) MPa					
M_R	2.13 (2.05)	1.61 (1.53)	1.61 (1.55)	1.31 (1.27)	1.31 (1.27)
γ	9.45 (10.21)	13.39 (14.38)	13.39 (14.00)	17.38 (15.50)	17.38 (16.65)
$\overline{A_{sh,ratio}}$	1.00	1.00	1.67	1.00	1.67
$A_{sh,ratio}$	1.20	1.32	2.05	1.20	1.99
	BJ-Failure	BJ-Failure	BJ-Failure	J-Failure	BJ-Failure

Remark : $\overline{A_{sh,ratio}}$ use $\overline{f_{yt}}$=700MPa

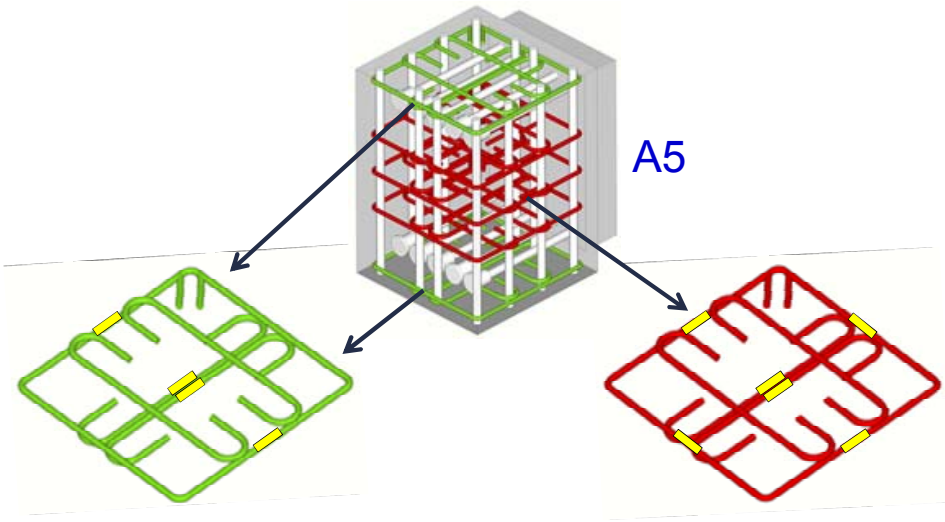
Test results (1/5)

	A5
	
	D13@125mm
f'_c	70 (70) MPa
M_R	2.13 (2.05)
γ	9.45 (10.21)
$\overline{A_{sh,ratio}}$	1.00
$A_{sh,ratio}$	1.20
	BJ-Failure

Remark : $\overline{A_{sh,ratio}}$ use

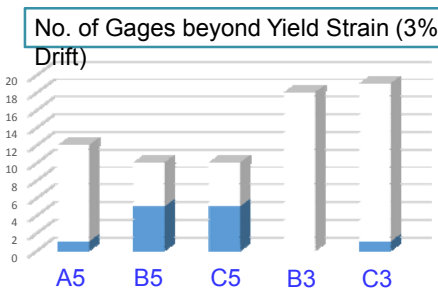


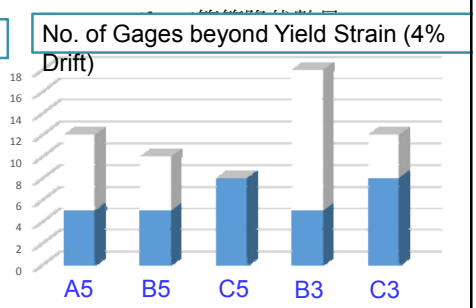
Strain Gages



實際降伏強度 843 MPa

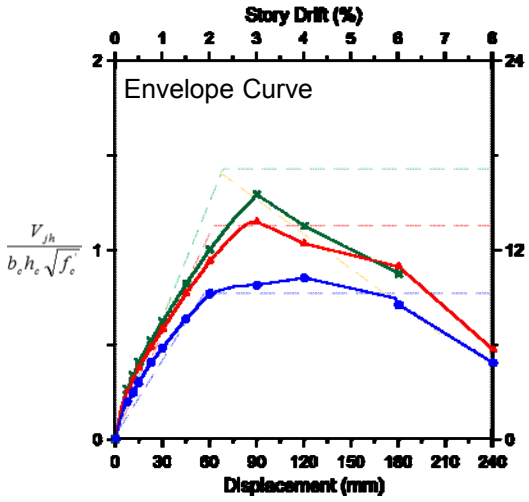
Maximum value of f_{yt} = 800 MPa permitted for computing the amount of A_{sh}





	A5	B5	C5	B3	C3	Average stress (MPa)
3% Drift	590	711	706	610	564	636
4% Drift	677	789	843	707	759	755

試體γ值討論



γ =17.38

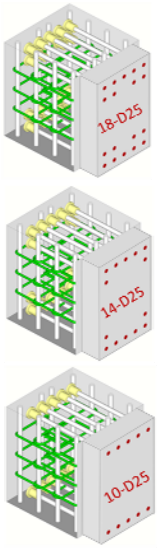
$\overline{A_{sh,ratio}} = 1.00$

γ =13.39

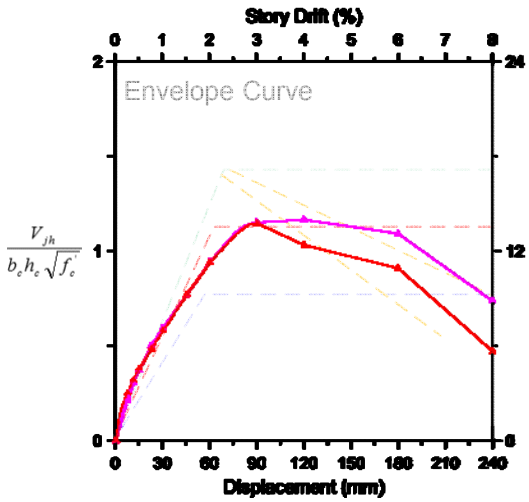
$\overline{A_{sh,ratio}} = 1.00$

γ =9.45

$\overline{A_{sh,ratio}} = 1.00$



試體γ值討論

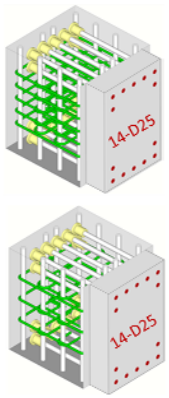


γ =13.39

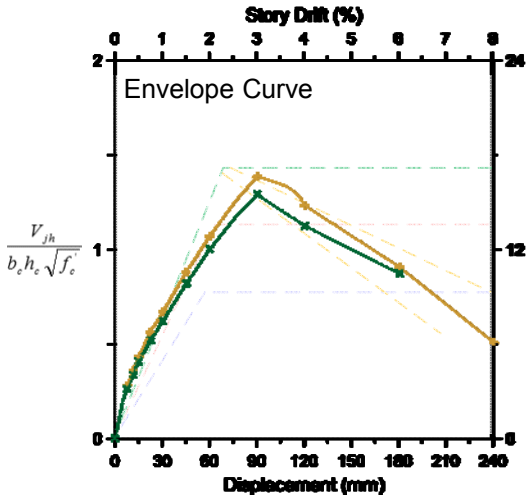
$\overline{A_{sh,ratio}} = 1.67$

γ =13.39

$\overline{A_{sh,ratio}} = 1.00$

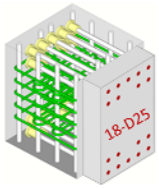


試體γ值討論



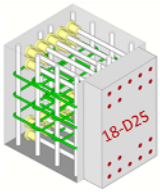
γ =17.38

$A_{sh,ratio}$ = 1.67



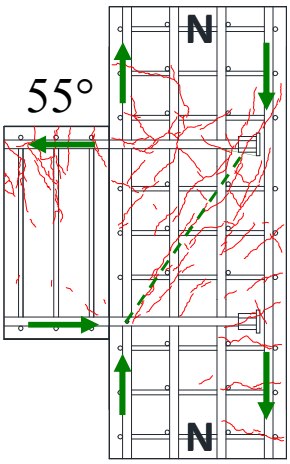
γ =17.38

$A_{sh,ratio}$ = 1.00

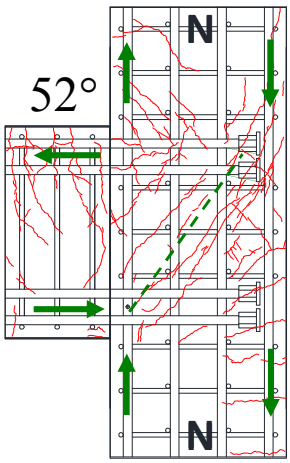


接頭剪力裂縫(1/2)

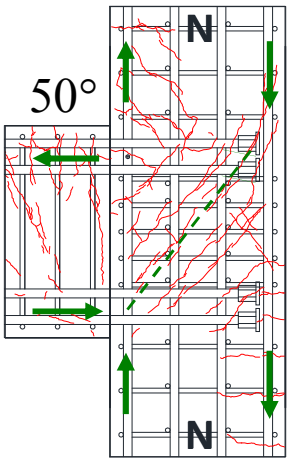
A5
Drift -4%



B5
Drift -4%

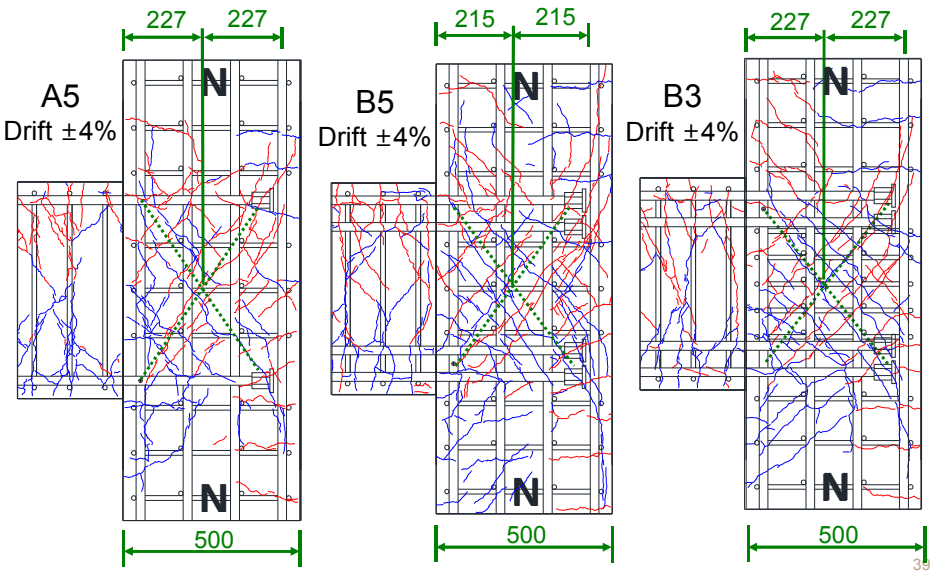


B3
Drift -4%



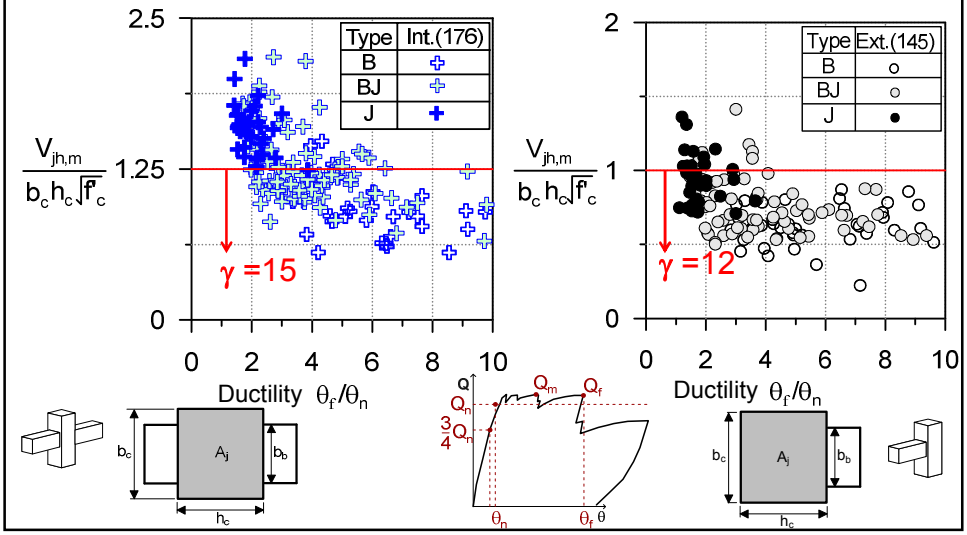
接頭剪力裂縫(2/2)

L_{dt} =16.3d_b =415 mm



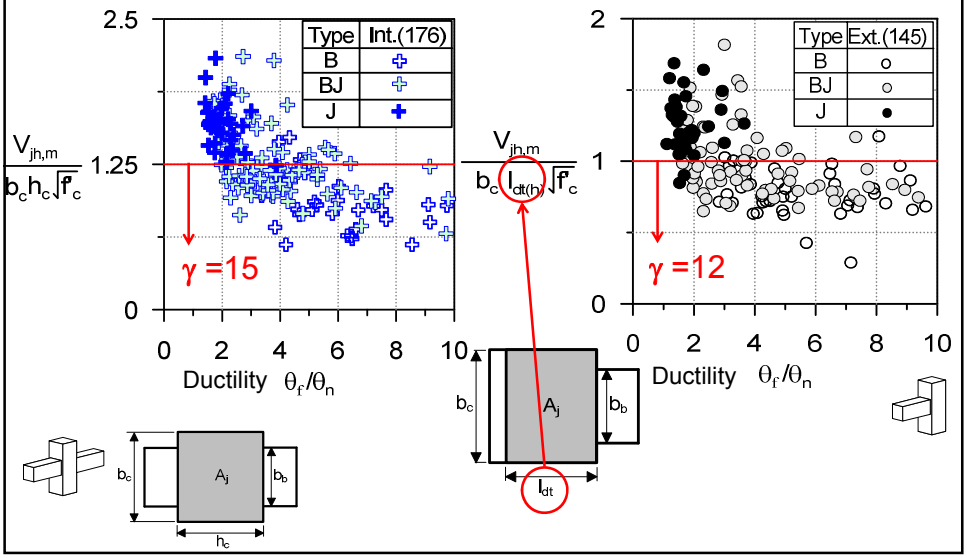
Joint Shear Strength-Ductility Trends

- Experimental joint shear force, normalized to effective joint area and $\sqrt{f'_c}$ in MPa
- There is no J failures below the γ-value of 15 for interior joints, but quite a few below the γ-value of 12 for exterior joints. (To be evaluated later)



Change of Effective Joint Depth

- Obviously improved, but…



Outline

- Frame Mechanism – Avoid Story Mechanism
- Hinge Development
  - Capacity design
  - Joint shear demand
- Joint Shear Design
  - Joint shear strength
  - Joint transverse reinforcement
- **Development Length Requirements for Joints**
- Concluding remarks

Hooked and Headed Bars Anchorage

Development length for a bar with a standard 90-degree hook

(ACI 318-14, 18.8.5.1)

$$\ell_{dh} = \frac{f_y d_b}{65\sqrt{f_c'}} \ (f_c' \text{ in psi}) \quad \text{or} \quad 0.185\frac{f_y d_b}{\sqrt{f_c'}} \ (f_c' \text{ in MPa})$$

Development length for headed deformed bars (ACI 318-14, 25.4.4)

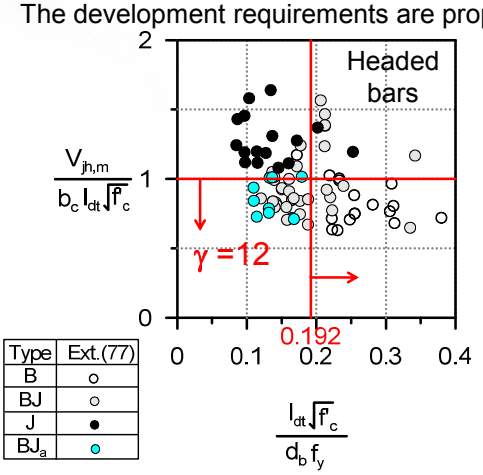
$$\ell_{dt} = 0.016\frac{f_y d_b}{\sqrt{f_c'}} \ (f_c' \text{ in psi}) \quad \text{or} \quad 0.192\frac{f_y d_b}{\sqrt{f_c'}} \ (f_c' \text{ in MPa})$$

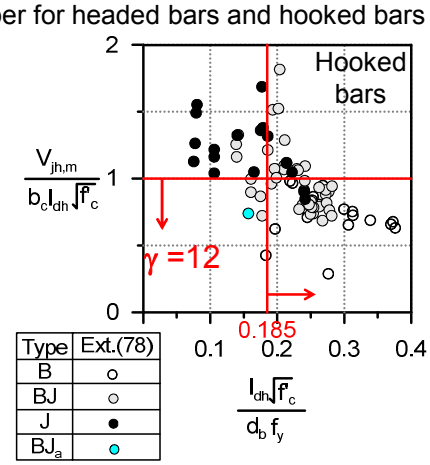
and value of f_c' shall not exceed 6000 psi → 10,000 psi ?

- ☐ Bar f_y shall not exceed 60,000 psi → 100,000 psi ?
- ✓ Net bearing area of head A_{brg} shall be at least $4A_b$
- ✓ Clear cover for bar shall be at least $2d_b$
- ☐ Clear spacing between bars shall be at least $4d_b$ → $2d_b$

Evaluation of Anchorage Requirements

- Anchorage failures (BJa) fall in 3rd quadrant
- The development requirements are proper for headed bars and hooked bars

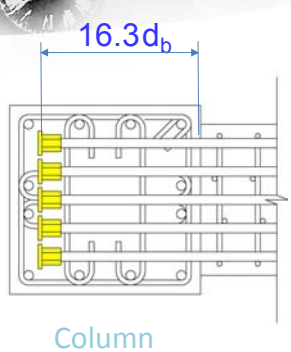




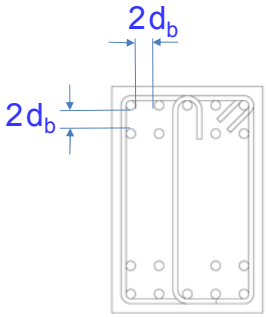
Required $\ell_{dt} \geq 0.192\frac{f_y d_b}{\sqrt{f_c'}}$

Required $\ell_{dh} \geq 0.185\frac{f_y d_b}{\sqrt{f_c'}}$

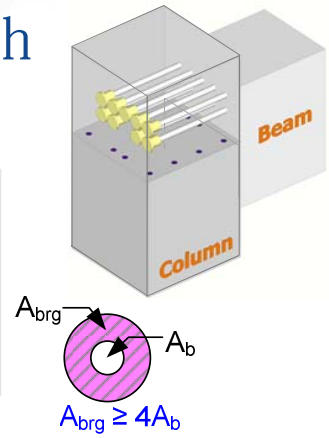
Anchorage Length



Column



Beam



$A_{brg}=5.5A_b$

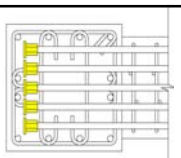
$f_y=685\ MPa$

$f_c'=70\ MPa$

$$\ell_{dt,required} = \frac{f_y d_b}{5.2\sqrt{f_c'}} = \frac{685 d_b}{5.2\sqrt{70}} = 15.8 d_b$$

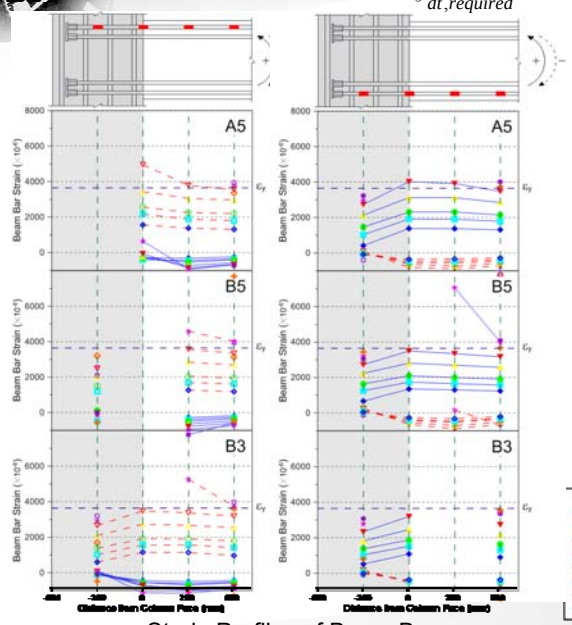
(ACI 318-14, 25.4.4.2)

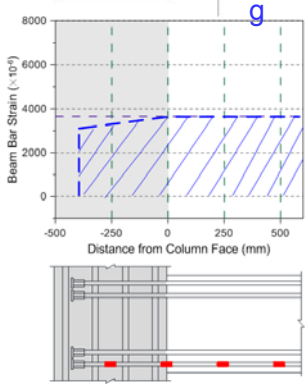
$$\frac{\ell_{dt,provided}}{\ell_{dt,required}} = 1.04$$

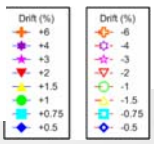


2 d$_b$

Clear spacing







Strain Profiles of Beam Bars

Outline

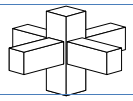
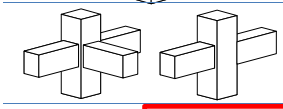
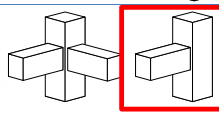
- Frame Mechanism – Avoid Story Mechanism
- Hinge Development
  - Capacity design
  - Joint shear demand
- Joint Shear Design
  - Joint shear strength
  - Joint transverse reinforcement
- Development Length Requirements for Joints
- Concluding remarks

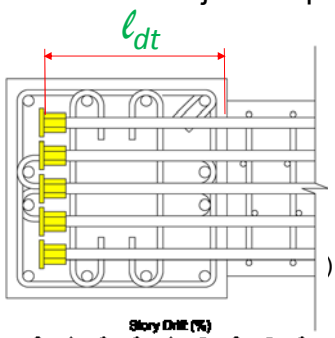
Concluding remarks - effective joint depth

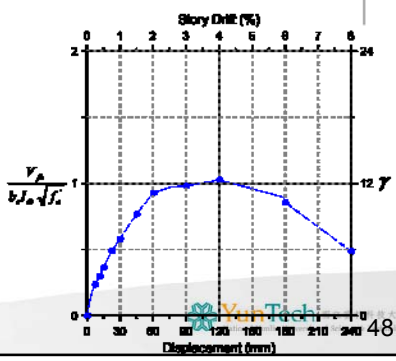
$$V_n = \gamma\sqrt{f_c'}b_j h_j \quad (f_c' \text{ in psi}) \text{ or}$$

$$V_n = \frac{\gamma}{12}\sqrt{f_c'}b_j h_j \quad (f_c' \text{ in MPa})$$

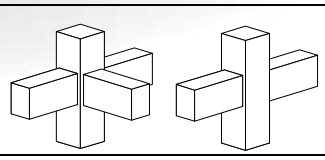
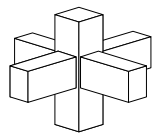
γ-values for special frame joints

	γ = 20
	γ = 15
	γ = 12

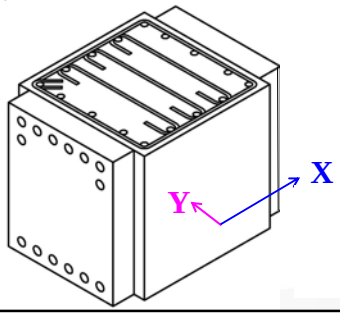




ACI 規範

	
雙向 **1.0 A_{sh}**	雙向 **0.5 A_{sh}**

本研究建議



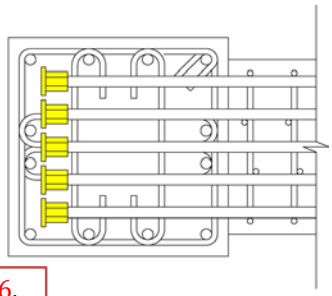
接頭兩對面接梁，接頭有效寬度內之繫筋量可以減少，但接頭箍筋維持與柱端相同之箍筋間距不放寬!

X、Y向 要求之 Ash 不同
垂直未圍束面才要求 1.0 Ash

Concluding remarks - Headed bars

$A_{brg}=5.5A_b$ $f_y=685\ MPa$

$2d_b$ bar spacing $f_c'=70\ MPa$



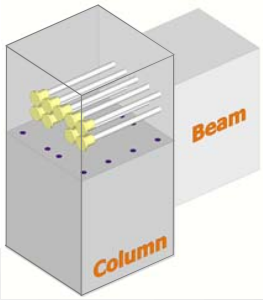
ACI 318-14

18.8.5.2 For headed deformed bars satisfying 20.2.1.6, development in tension shall be in accordance with 25.4.4, except clear spacing between bars shall be permitted to be at least $3d_b$ or greater.

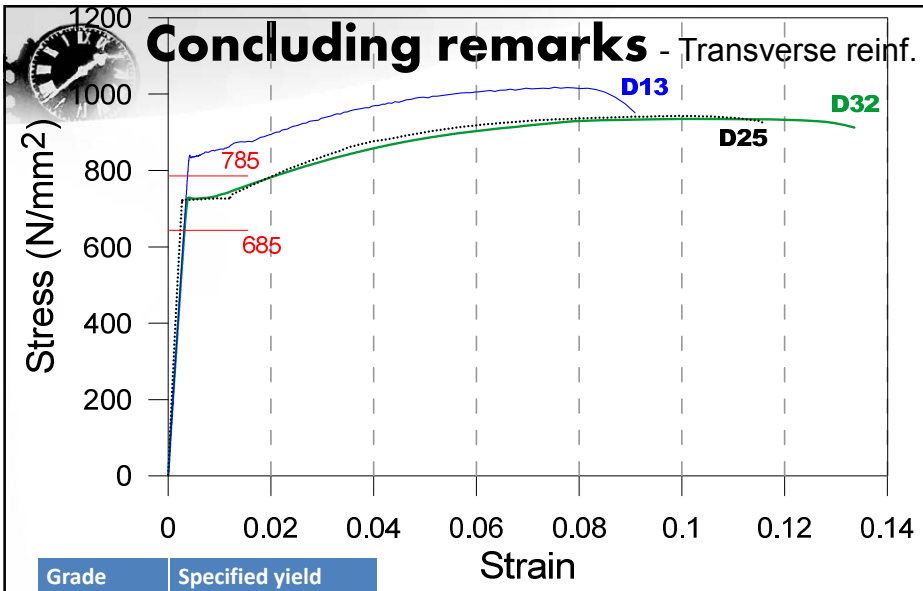
$$\ell_{dt} = 0.016\frac{f_y d_b}{\sqrt{f_c'}}\ (f_c' \text{ in psi}) \quad \text{or} \quad 0.192\frac{f_y d_b}{\sqrt{f_c'}}\ (f_c' \text{ in MPa})$$

and value of f_c' shall not exceed 14,000 psi [100 MPa]

- Bar f_y shall not exceed 100,000 psi [690 MPa]
- Clear spacing between bars shall be at least $2d_b$



Concluding remarks - Transverse reinf.



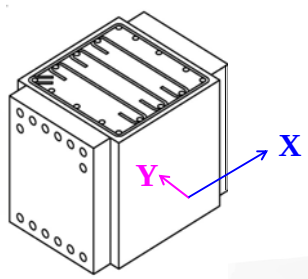
Grade	Specified yield strength
SD 785	785 MPa (114 ksi)
SD 685	685 MPa (100 ksi)

Key Design Parameters

- Moment strength ratio $M_R \geq 1.2$
- Joint shear strength $V_n = \gamma\sqrt{f_c'}\,b_j h_c$
- Confinement $A_{sh,provided} \geq A_{sh,required}$
- Anchorage of beam reinforcement

e f_c'

ℓ_{dt} for headed bars



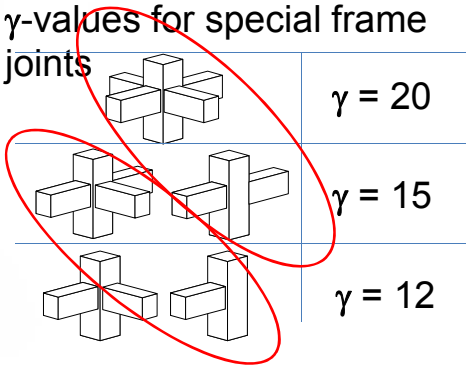
Maximum value permitted for design calculations	
Bar f_y	100,000 psi [690 MPa]
Bar f_{yt}	116,000 psi [800 MPa]
Concrete f_c'	14,000 psi [100 MPa]

Effective confining ratio-TBD

$$V_n = \gamma\sqrt{f_c'}b_j h_j \quad (f_c' \text{ in psi}) \text{ or}$$

$$V_n = \frac{\gamma}{12}\sqrt{f_c'}b_j h_j \quad (f_c' \text{ in MPa})$$

γ-values for special frame joints

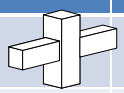
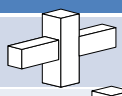
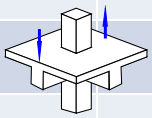
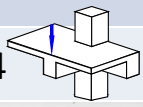


$$\frac{20}{15} = 1.33$$

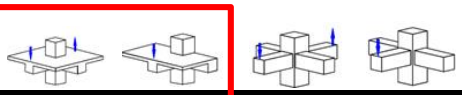
$$\frac{15}{12} = 1.25$$

Further study

- To develop a deformation-based model of the degradation of the joint shear strength.
- In 2014-2015, we plan to test 8 beam-column-slab connections made with HS reinforcement and concrete.

2012	2013	2014	2015
4 	4 		
	5 		
		4 	
			4 

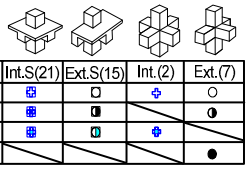
試體統計



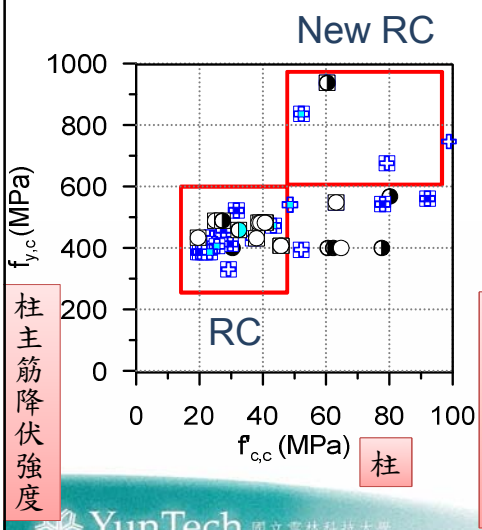
Failure mode	Int.S	Ext.S	Int.	Ext.	SUM
B 破壞	11	10	1	2	24
BJ 破壞	7	4	0	4	15
BJa破壞	3	1	1	0	5
J 破壞	0	0	0	1	1
CY破壞	0	2	0	0	2
SUM	21	17	2	7	47

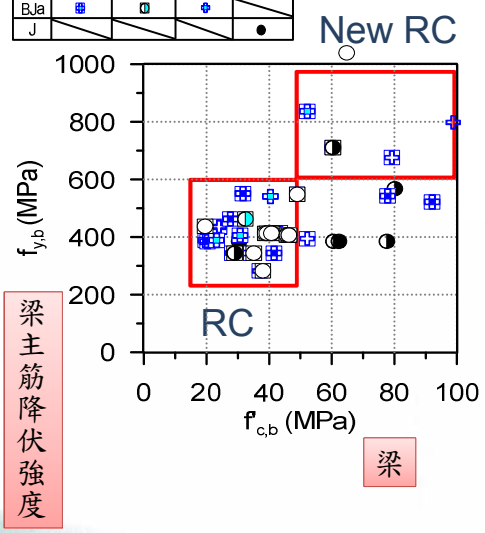
CY破壞：柱產生塑鉸破壞，非本研究主要破壞模式

資料庫數據範圍

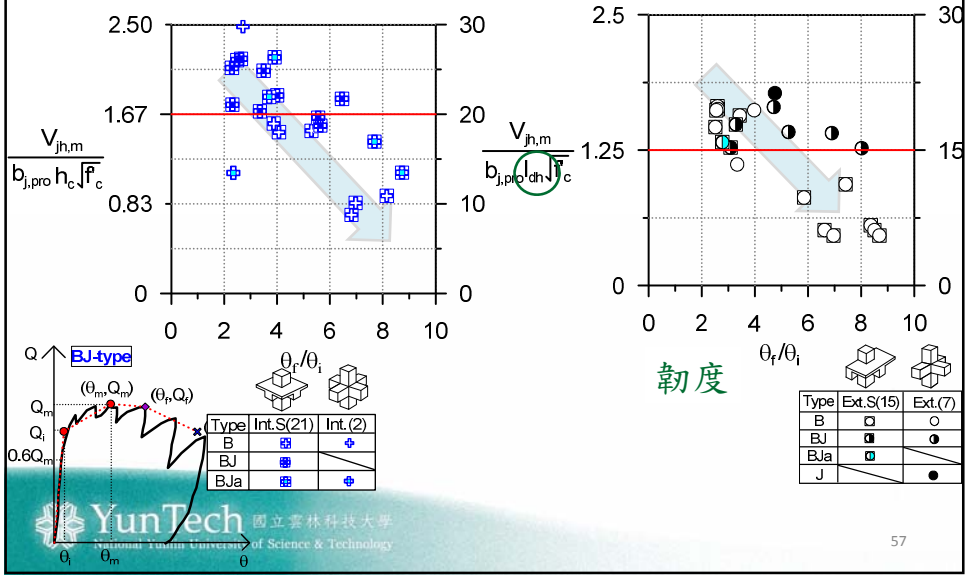


Type	Int.S(21)	Ext.S(15)	Int.(2)	Ext.(7)
B	✚	○	✚	○
BJ	▦	◑		◑
BJa	▦	◑	✚	
J				●

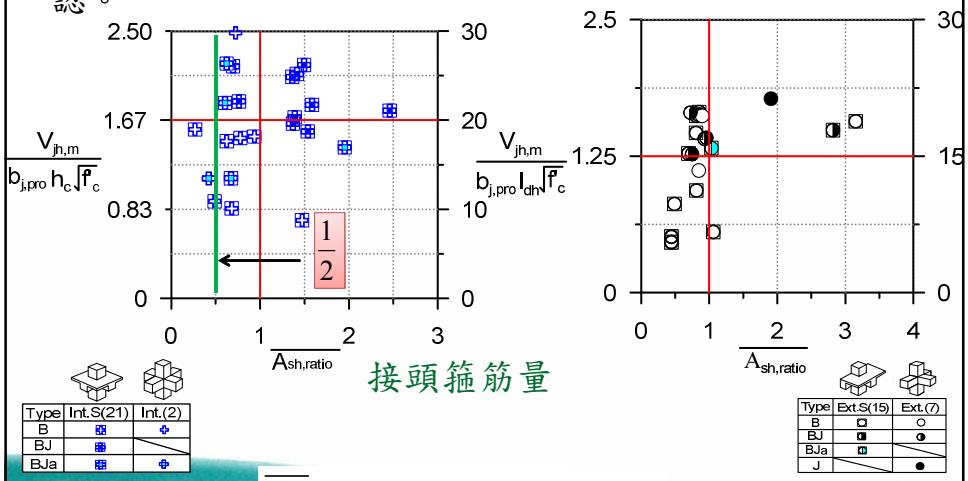




接頭剪應力 γ_m v.s. 韌度



■內部接頭接頭箍筋量可以減半，外部接頭則待確認。



$$\overline{f_{yt}} = \min(f_{yt}, 800)\ MPa$$

Thanks for your attentions

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

NARLabs 國家實驗研究院
國家地震工程研究中心

高強度鋼筋混凝土柱之耐震圍束設計

黃世建

國家地震中心副主任

國立臺灣大學土木工程系教授

Dec. 5, 2014

承諾・熱情・創新

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簡報大綱

- 前言
- 設計公式
- 實驗規劃
- 測試結果
- 預鑄應用
- 結論

前言

臺灣高樓層建築發展

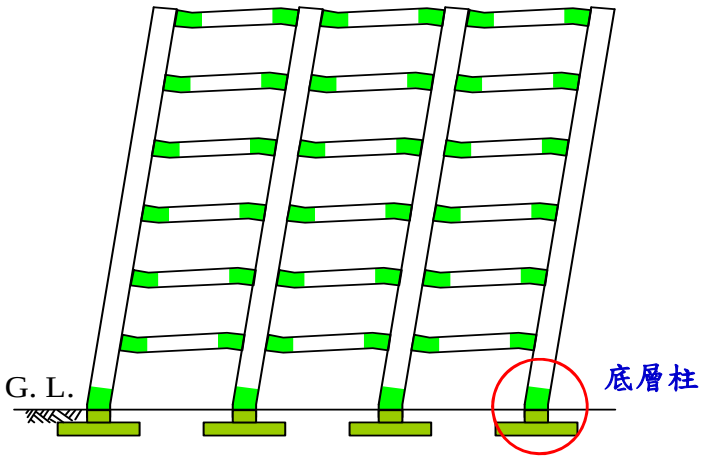
2.2 < 高深比 < 3

柱尺寸過大

使用高強度材料

縮減尺寸

高樓層建築之底層柱



重要

- 承受極大之軸力
- 變形能力降低

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%

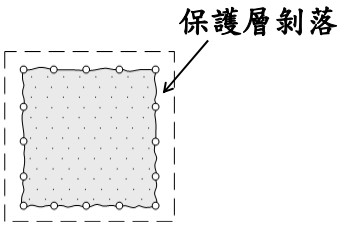
取3%為合格標準

設計公式

現行設計公式

- 橫箍柱 (ACI 318-11、土木 401-100)

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



其中 $f_{yt} \leq 700MPa$

修正設計公式

- 橫箍柱 (ACI 318-14)

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

其中 $f_{yt} \leq 700MPa$

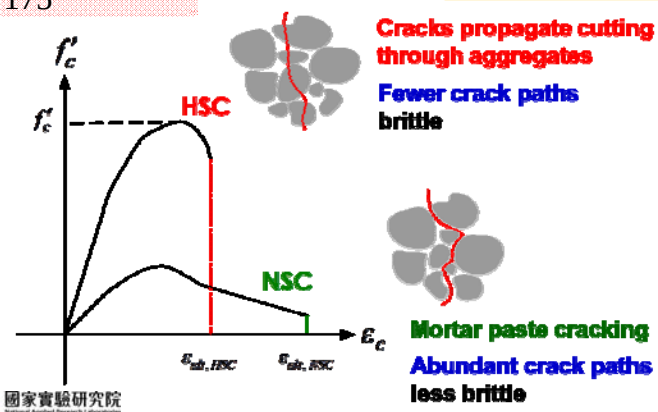
混凝土強度參數 k_f

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

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

混凝土強度 ↑

圍束箍筋量 ↑



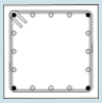
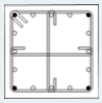
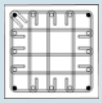
彎鉤有效性參數 k_n

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

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

n_l 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

n_l	k_n
 4	2.0
 8	1.33
 16	1.14

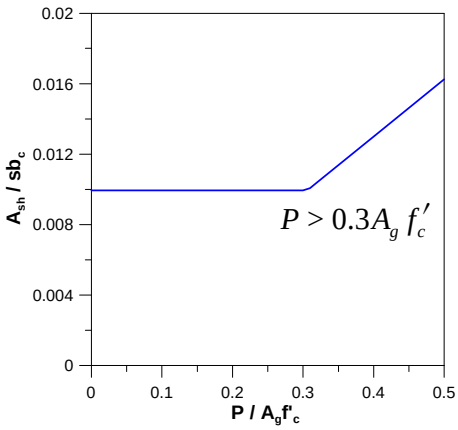
軸力參數 k_p

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

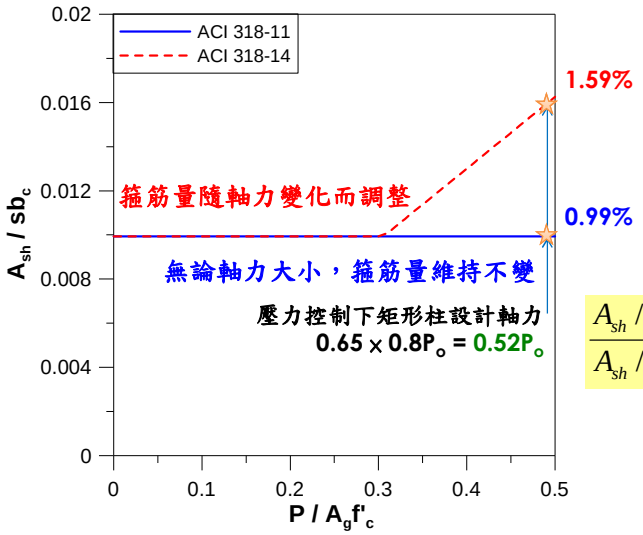
$$k_p = \frac{P}{A_g f_c'}$$

承受軸力 ↑

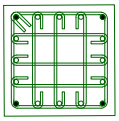
圍束箍筋量 ↑



圍束箍筋量之差異



Study Case:



$f_c' = 70\ MPa$

$f_y = 685\ MPa$

$f_{yt} = 785\ MPa$

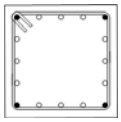
$$\frac{A_{sh}/sb_{c,\,ACI\ 318\text{-}14}}{A_{sh}/sb_{c,\,ACI\ 318\text{-}11}} = 1.61$$

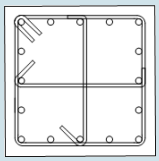
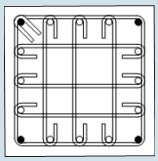
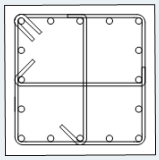
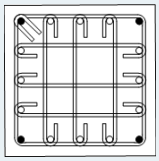
圍束箍筋配置之差異

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

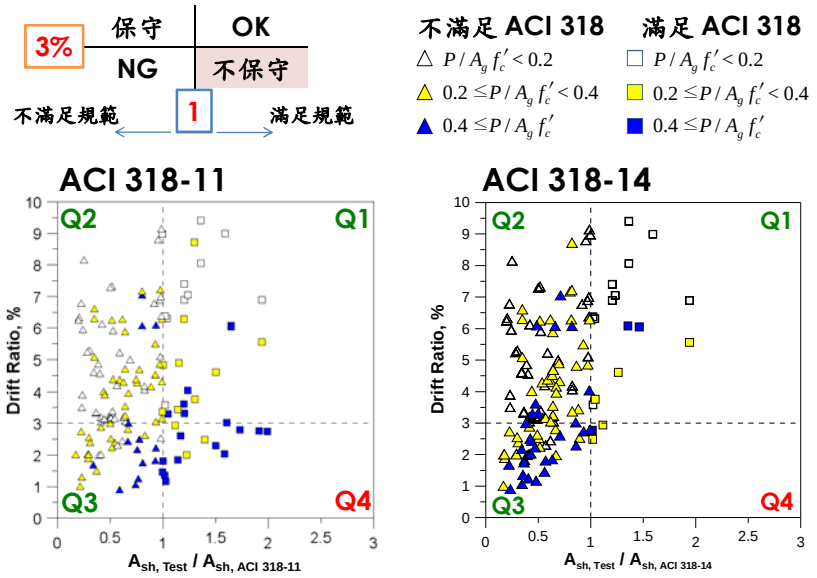
Study Case:

(600x600mm)



	ACI 318-11	ACI 318-14	
普通強度 $f_c' = 30\ MPa$ $f_{yt} = 420\ MPa$	 $\frac{A_{sh}}{sb_c} = 0.73\%$ D13@100mm	 $\frac{A_{sh}}{sb_c} = 1.02\%$ D13@120mm	+40%
高強度 $f_c' = 70\ MPa$ $f_{yt} = 785\ MPa$	 $\frac{A_{sh}}{sb_c} = 1.14\%$ D16@100mm	 $\frac{A_{sh}}{sb_c} = 1.59\%$ D16@120mm	+40%

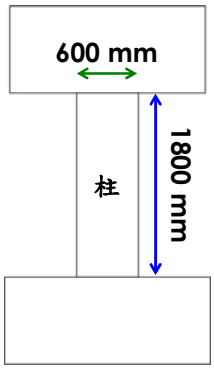
PEER 資料庫 (橫箍柱)



實驗規劃

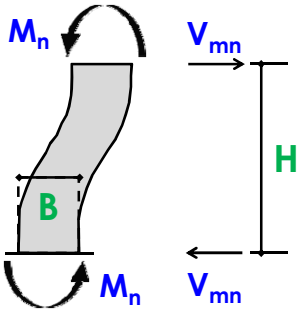
柱試體規劃

❶ 符合柱定義 (H/B ≥ 3)

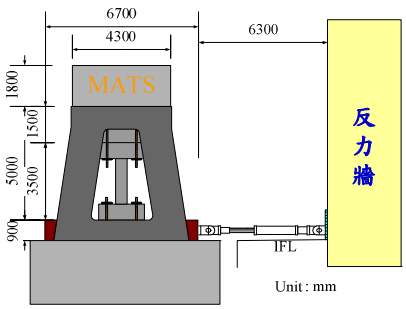


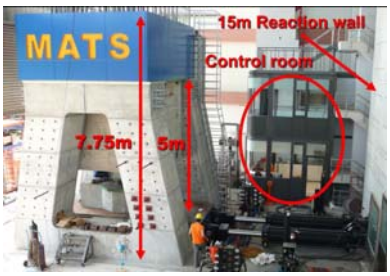
❷ 作用剪力高

$$V_{mn} = \frac{M_n}{H/2}$$

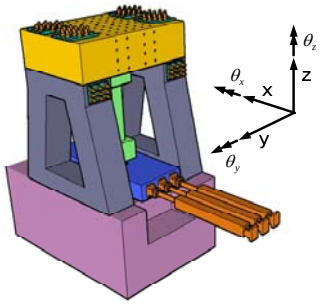


NCREE實驗設備



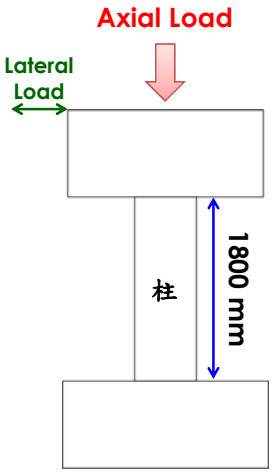


Multi-Axial Testing System

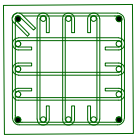


	x	y	z
Load (tf)	400	200	4,000
Disp. (m)	±1.2	±0.1	0.15

實驗規劃

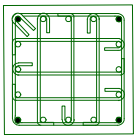


T70-1



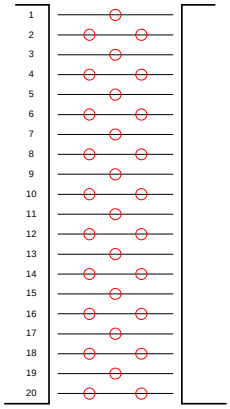
180˚-180˚彎鉤

T70-2



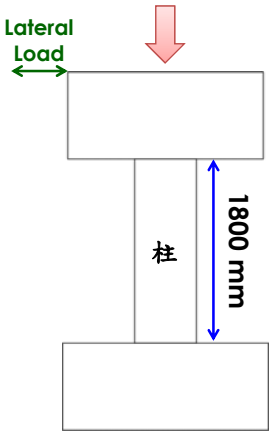
90˚-180˚彎鉤

90˚-180˚彎鉤
交錯配置

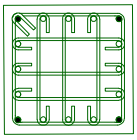


○ 90˚彎鉤端

實驗規劃

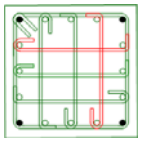


T70-3



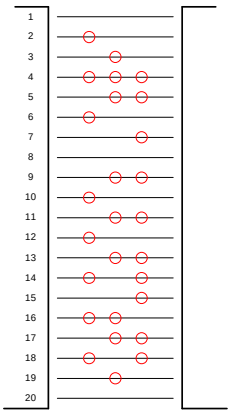
180˚-180˚彎鉤

T70-4



90˚-180˚彎鉤

90˚-180˚彎鉤
非交錯配置



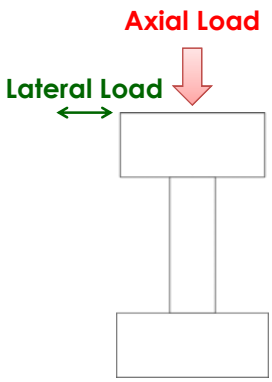
○ 90˚彎鉤端

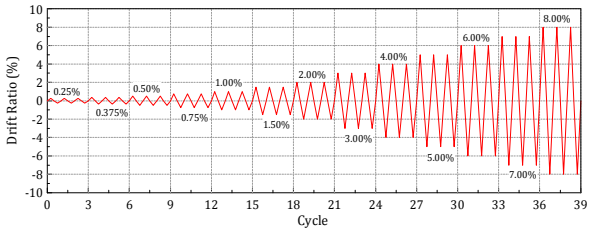
試體參數

	T70-1	T70-2	T70-3	T70-4
斷面尺寸 (mm)	600×600			
彎鉤型式				
f'_c (MPa)	70			
f_y (MPa)	685			
f_{yt} (MPa)	785			
垂直鋼筋	4-D32, 8-D25			
主筋比	2.60%			
橫向鋼筋	D13@90mm			
A_{sh}/sb_c	1.35%			

側力加載歷時

$$P = 12749\ kN$$
$$0.455 A_g f'_c$$



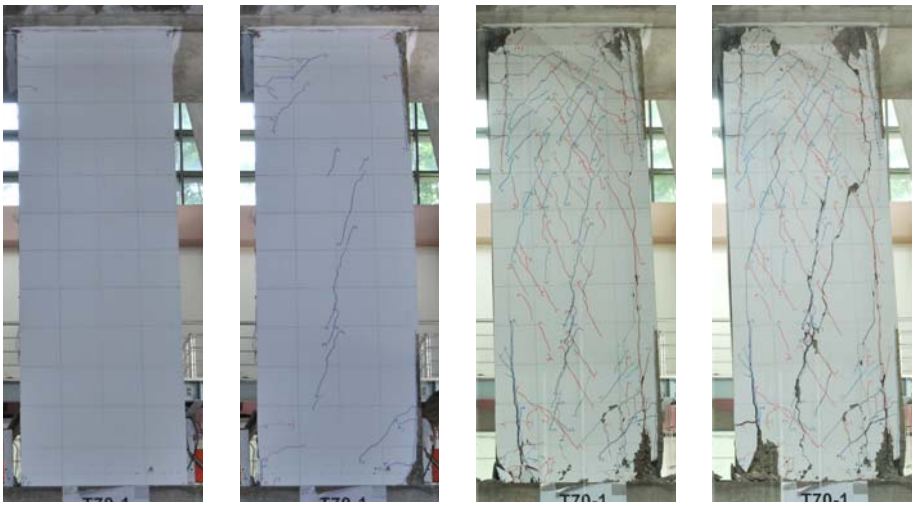


測試結果

測試結果

T70-1

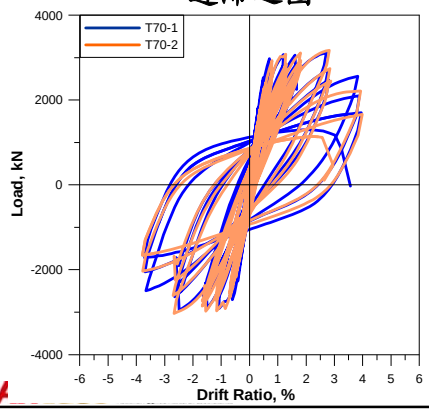
0.5% 1.0% 2.0% 3.0%



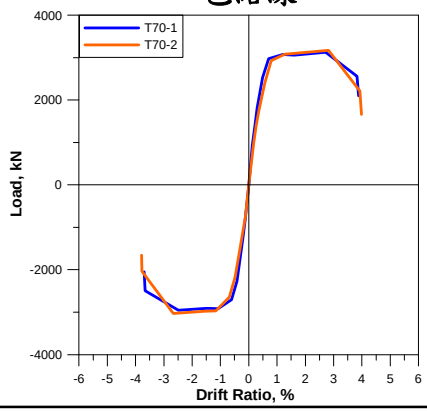
遲滯迴圈與包絡線

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



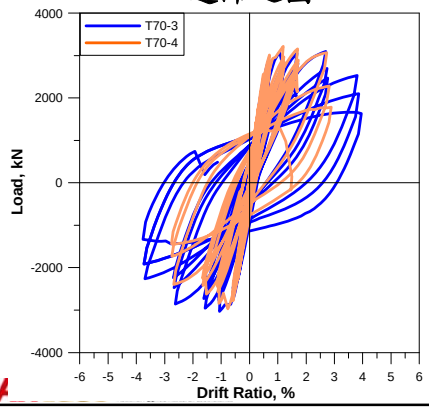
包絡線



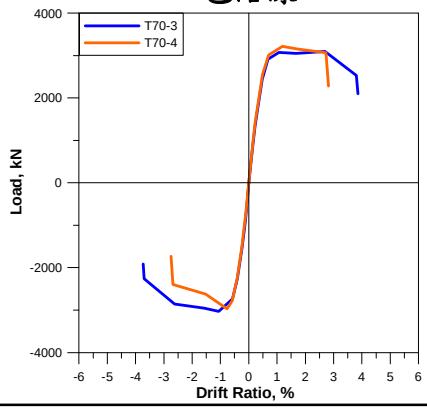
遲滯迴圈與包絡線

Specimen ID	f'c (MPa)	V_{max} (kN)	Drift@V_{max} (%)	0.8V_{max}	Drift@0.8V_{max} (%)
T70-3	77.91	3092	2.69	2473	3.81
T70-4	77.91	3216	1.19	2573	2.78

遲滯迴圈



包絡線



討論

紐西蘭設計公式

- 橫箍柱 (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 $f_{yt} \leq 800MPa$

NCREE 歷年矩形柱試體

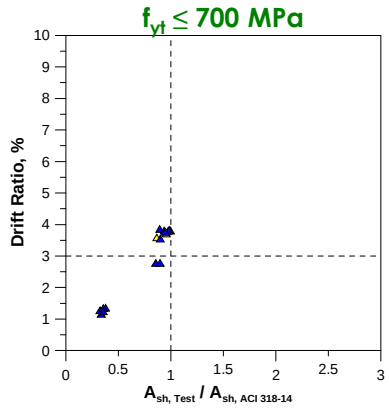
年度	2010	2011	2013	2014
試體數量	5	4	3	4
f'_c (MPa)	70、100	70、100	100	70
f_y (MPa)	685			
f_{yt} (MPa)	785			
主筋比	2.25%	2.25%	2.94%	2.60%
A_{sh}/sb_c	0.87% ~ 1.22%	1.22% ~ 1.90%	2.11%	1.35%
$P/A_g f'_c$	0.52 ~ 0.57	0.29 ~ 0.46	0.36	0.42 ~ 0.46

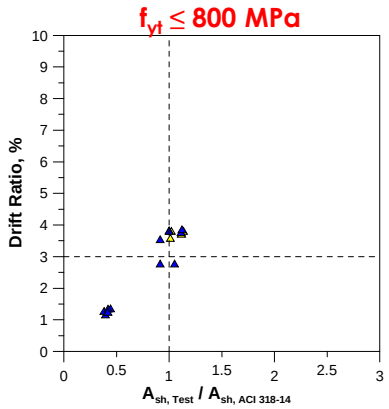
橫向鋼筋 f_{yt} 放寬至 800 MPa

△ $P/A_g f'_c < 0.2$

▲ $0.2 \le P/A_g f'_c < 0.4$

▲ $0.4 \le P/A_g f'_c$

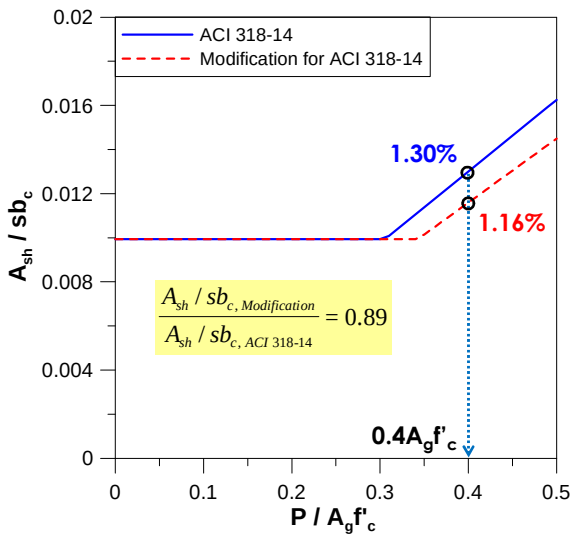




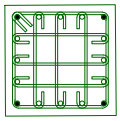
設計公式之修訂

ACI 318-14	修訂建議
$\frac{A_{sh}}{sb_c} \geq \max\begin{cases} 0.09\frac{f'_c}{f_{yt}} \\ 0.3\frac{f'_c}{f_{yt}}\left(\frac{A_g}{A_{ch}}-1\right) \\ 0.2k_f k_n k_p \frac{f'_c}{f_{yt}}\frac{A_g}{A_{ch}} \end{cases}$	$\frac{A_{sh}}{sb_c} \geq \max\begin{cases} 0.09\frac{f'_c}{f_{yt}} \\ 0.3\frac{f'_c}{f_{yt}}\left(\frac{A_g}{A_{ch}}-1\right) \\ 0.2k_f k_n k_p \frac{f'_c}{f_{yt}}\frac{A_g}{A_{ch}} \end{cases}$
其中 $f_{yt} \leq 700MPa$	其中 $f_{yt} \leq \cancel{700MPa}$ $800MPa$

降低圍束箍筋量



Study Case:



$f'_c = 70\ MPa$

$f_y = 685\ MPa$

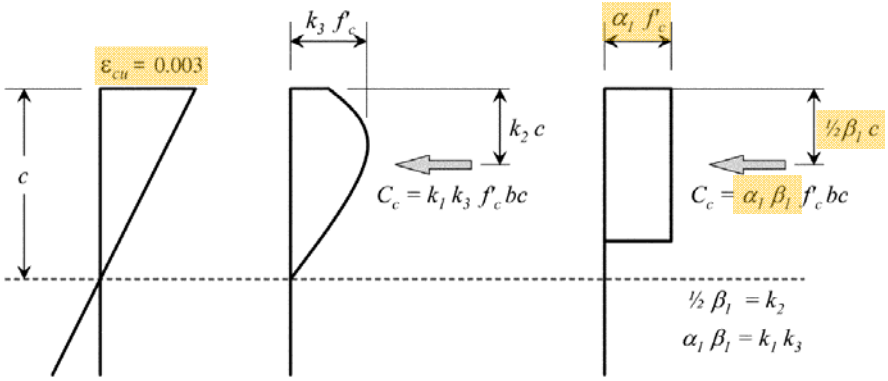
$f_{yt} = 785\ MPa$

700 MPa:
D16@140mm, A_{sh}/sb_c = 1.36%

800 MPa:
D13@100mm, A_{sh}/sb_c = 1.22%

計算撓曲強度

- 等值矩形應力塊



a) Strain Distribution

b) 2nd order Parabolic Stress Distribution and Equivalent Force

c) ACI Rectangular Stress Block and Equivalent Force

計算撓曲強度

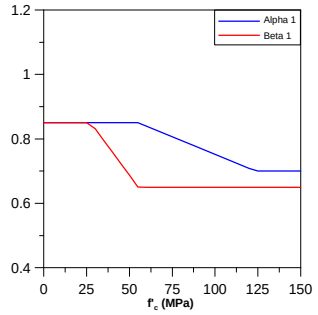
- **ITG-4.3R-07**

等值矩形應力塊參數

$$0.70 \le \alpha_1 = 0.85 - 0.0022(f'_c - 55) \le 0.85$$

$$0.65 \le \beta_1 = 0.85 - 0.0073(f'_c - 28) \le 0.85$$

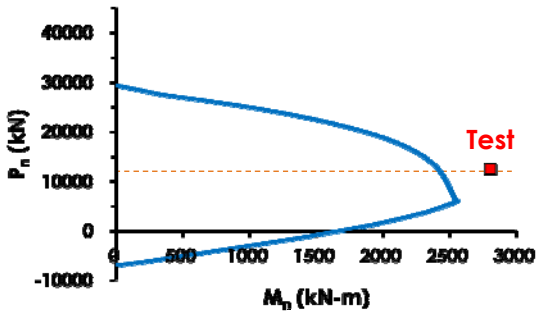
(f'$_c$ in MPa)



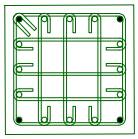
主筋壓力降伏強度

$$f_y \le 550\ MPa$$

撓曲強度



T70-1



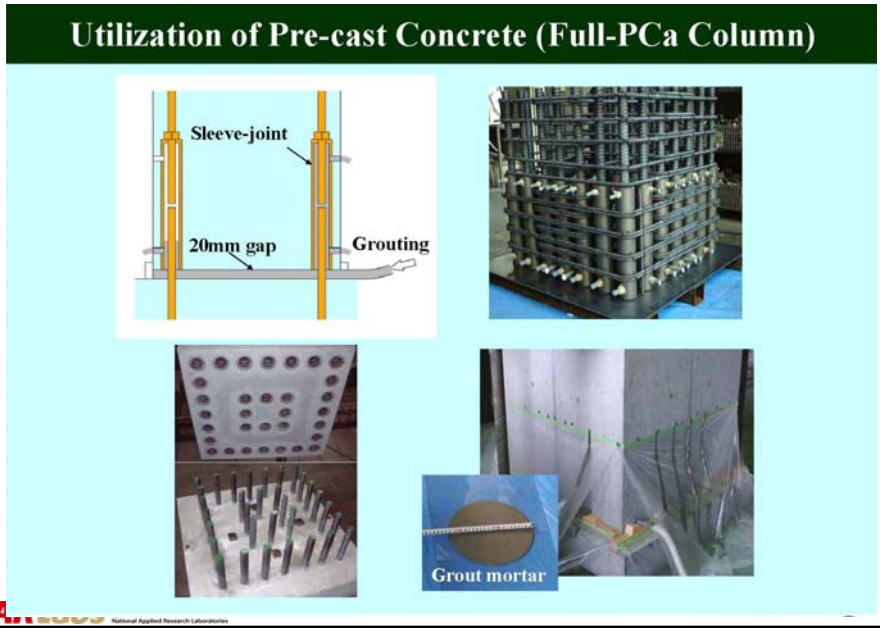
$f_c' = 81.94\ MPa$

$P = 12454\ kN$

	f'_c (MPa)	V_{test} (kN)	V_{mn} (kN)	V_{test}/V_{mn}
T70-1	81.94	3122	2688	1.16
T70-2	81.94	3174	2688	1.18
T70-3	77.91	3092	2592	1.19
T70-4	77.91	3216	2592	1.24

預鑄應用

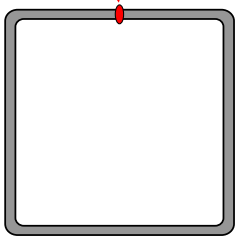
高強度鋼筋預鑄用套管式續接器



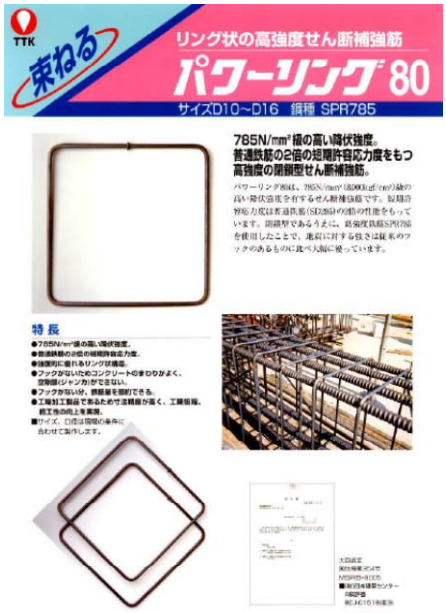
銲接閉合型高強度箍筋

Power Ring

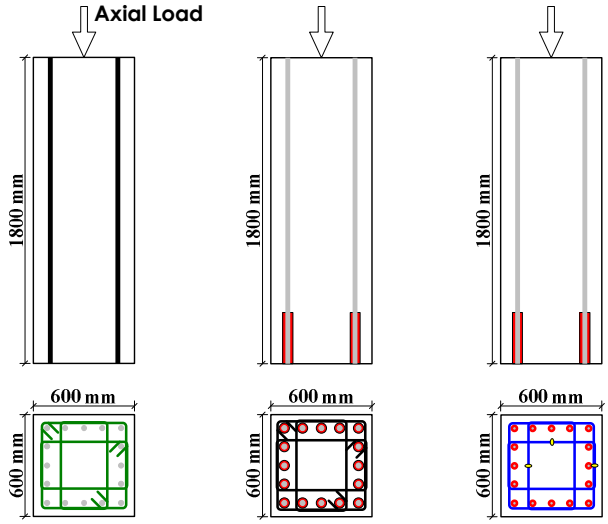
熔接點



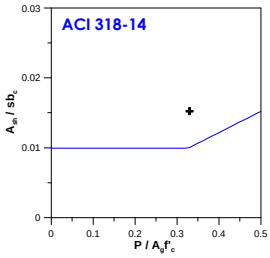
f_{yt} = 785 MPa



柱試體測試



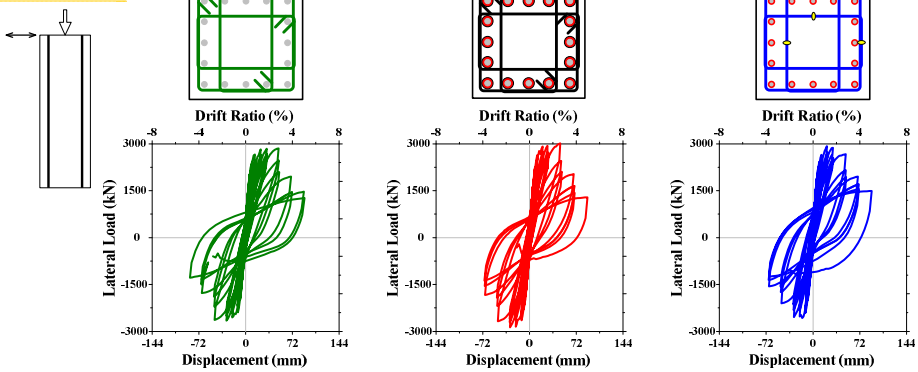
標準配筋　　套管式續接器　　套管式續接器 + 銲接閉合型箍筋



$$\frac{A_{sh}/sb_{c,test}}{A_{sh}/sb_{c,ACI\ 318\text{-}14}} = 1.39$$

測試結果

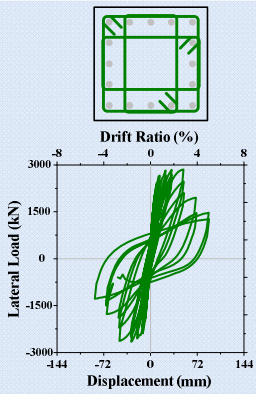
$0.33A_g f'_c$



TC2	TC4	TC6
V_{max}= 2853 kN	V_{max}= 3014 kN	V_{max}= 2925 kN
Drift@0.8V_{max} = 3.6%	Drift@0.8V_{max} = 3.5%	Drift@0.8V_{max} = 3.5%

預鑄 vs. 場鑄

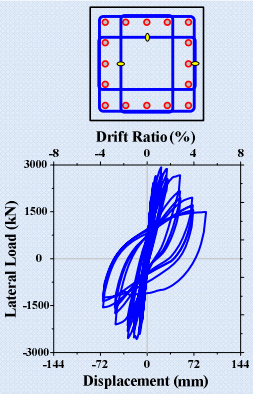
預鑄



TC2

V_{max}= 2853 kN

Drift@0.8V_{max} = 3.6%

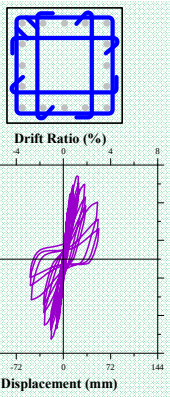


TC6

V_{max}= 2925 kN

Drift@0.8V_{max} = 3.5%

場鑄



TC10

V_{max}= 2654 kN

Drift@0.8V_{max} = 2.3%

結論

結論

- 承受高軸力作用下之高強度鋼筋混凝土柱，其變形能力符合需求
- 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
- 橫向鋼筋 f_{yt} 放寬至 800 MPa，可降低圍束箍筋量

設計公式修訂建議

- 橫箍柱 (ACI 318-14)

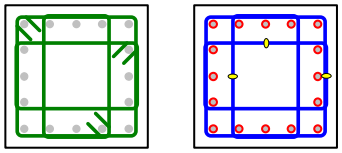
$$\frac{A_{sh}}{sb_c} \geq \max \begin{cases} 0.09\dfrac{f_c'}{f_{yt}} \\ 0.3\dfrac{f_c'}{f_{yt}}\left(\dfrac{A_g}{A_{ch}}-1\right) \\ 0.2k_f k_n k_p \dfrac{f_c'}{f_{yt}}\dfrac{A_g}{A_{ch}} \end{cases} \quad \text{當} \quad P > 0.3A_g f_c' \quad \text{或} \quad f_c' > 70MPa$$

其中 $f_{yt} \leq 800MPa$

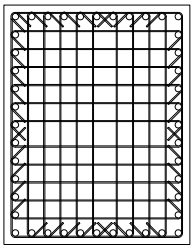
結論

- 以預鑄方式製作之柱試體，高軸力作用下之變形能力亦符合需求，可改善傳統耐震彎鉤施工不易之問題

閉合型箍筋



耐震彎鉤



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國家地震工程研究中心

簡報結束，感謝聆聽

SHEAR DESIGN OF NEW RC COLUMNS

Yu-Chen Ou
Associate Professor

TAIWAN TECH
National Taiwan University of Science and Technology

Outline

- Current ACI shear strength equations and limitations
- NCREE shear tests on New RC columns
- New RC column shear strength database
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ACI 318-11 shear strength provisions

- Nominal shear strength

$$V_n = V_c + V_s$$

$V_c =$ Nominal shear strength of concrete

$V_s =$ Nominal shear strength of shear reinforcement

ACI 318-11 shear strength provisions

- Shear strength of concrete

Provision	Note
Concrete compressive strength	ACI 318-11 Chapter 11.1.2 $\sqrt{f_c^{'}} \le 8.3\,MPa$ Or $f_c^{'} \le 70\,MPa$
Simplified shear strength of concrete	ACI 318-11 Chapter 11.2.1.2 $V_c = 0.17\left(1+\frac{P_u}{14A_g}\right)\sqrt{f_c^{'}}bd$ Eq (11-4)

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ACI 318-11 shear strength provisions

- Shear strength of concrete

Provision	Note
Detailed shear strength of concrete	ACI 318-11 Chapter 11.2.2.1 $V_c = \left(0.16\sqrt{f_c^{'}} + 17\rho_w \frac{V_u d}{M_m}\right)bd$ Eq (11-5) $M_m = M_u - P_u \frac{(4h-d)}{8} \ge 0$ Eq (11-6)
Upper limit shear strength of concrete	ACI 318-11 Chapter 11.2.2.2 $V_c = 0.29\sqrt{f_c^{'}}bd\sqrt{\left(1+\frac{0.29P_u}{A_g}\right)}$ Eq (11-7)

ACI 318-11 shear strength provisions

- Shear strength of reinforcement

Provision	Note
Yield strength of shear reinforcement	ACI 318-11 Chapter 11.4.2 $f_{yt} \le 420\ MPa$ For deformed bars $f_{yt} \le 550\ MPa$ For welded deformed wires
Minimum shear reinforcement	ACI 318-11 Chapter 11.4.6.3 $A_{v,\min} = 0.062\sqrt{f_c^{'}}\dfrac{bs}{f_{yt}} \ge \dfrac{0.35bs}{f_{yt}}$ Eq (11-13)

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ACI 318-11 shear strength provisions

- Shear strength of reinforcement

Provision	Note
Shear strength of reinforcement	ACI 318-11 Chapter 11.4.7.2 $V_s = \dfrac{A_v f_y d}{s}$ Eq (11-15)
Maximum shear strength of reinforcement	ACI 318-11 Chapter 11.4.7.9 $V_{s,\max} \le 0.66\sqrt{f_c^{'}}bd$

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ACI 318-11 shear strength provisions

Summary

- ACI 318-11 does not allow the use of high-strength concrete with f_c' more than 70 MPa. But ACI also states for beams, if minimum shear reinforcement is provided, f_c' can be higher than 70 MPa.
- ACI 318-11 does not allow the use of shear reinforcement (deformed bars) with f_y more than 420 MPa.

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Specimen Design

Columns	$\frac{P}{A_g f_c'}$	f_{cs}'	f_c'	Longitudinal Reinforcement			Transverse Reinforcement		
				f_{yls}	f_{yl}	ρ_l	f_{yts}	f_{yt}	ρ_t
A-1	10%	70	92.5	685 (MPa)	735 (MPa)	3.62% (16 D32)	785 (MPa)	862 (MPa)	0.15% (3D13 - 450)
A-2		100	103.2						
A-3		70	96.9						0.26% (3D13 - 260)
A-4		100	107.1						
E-1		70	92.2			3.38% (24 D25)			
E-2		70	90.8						0.59% (4D13 – 150)
E-5		70	97.4						0.98% (4D13 – 90)
B-1	15%	70	108.3	685 (MPa)	735 (MPa)	3.62% (16 D32)	785 (MPa)	862 (MPa)	0.15% (3D13 - 450)
B-2	18%	100	125.0						
B-3	20%	70	112.8						0.26% (3D13 - 260)
B-4		100	121.0						
E-3		70	79.0			3.38% (24 D25)			
E-4		70	101.1						0.59% (4D13 – 150
C-1	30%	70	104.1	685 (MPa)	735 (MPa)	3.62% (16 D32)	785 (Mpa)	862 (MPa)	0.15% (3D13 - 450
C-2		100	138.8						
C-3		70	104.6						0.26% (3D13 - 260)
C-4		100	130.0						
D-1	40%	70	101.0	685 (MPa)	735 (MPa)	3.62% (16 D32)	785 (MPa	862 (MPa)	0.15% (3D13 - 450
D-2		100	125.5						
D-3		70	106.4						0.26% (3D13 - 260)
D-4		100	127.8						

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Specimen Design

Construction of Specimen

Testing Setup

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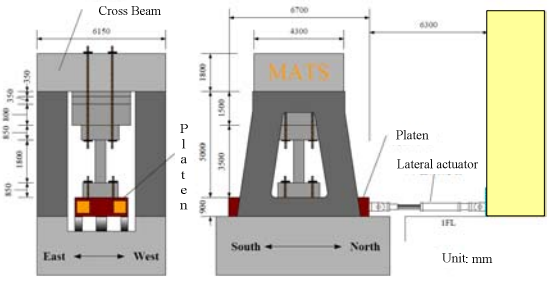
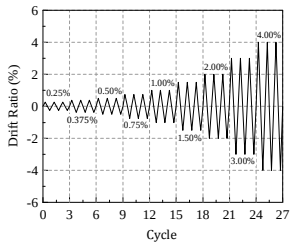
Testing Procedure



Testing Setup



Loading Protocol

Experimental Result

- Column with 0.1 axial load ratio

Columns	$\frac{P}{A_g f_c'}$	f_c'	Diagonal Cracking					Ultimate Condition				
			Drift (%)	σ_{st} (MPa)	V_{test} (kN)	$V_{s\,test}$ (kN)	$V_{c\,test}$ (kN)	Drift (%)	σ_{st} (MPa)	V_{test} (kN)	$V_{s\,test}$ (kN)	$V_{c\,test}$ (kN)
A-1	10%	92.5	0.35	19	1264	9	1255	0.57	243	1578	150	1428
A-2		103.2	0.33	6	1286	3	1283	0.53	235	1638	150	1488
A-3		96.9	0.32	16	1279	13	1266	0.75	359	1772	413	1359
A-4		107.1	0.33	14	1298	10	1288	0.79	418	1781	447	1334
E-1		92.2	0.37	20	1348	15	1333	0.84	397	1817	574	1244
E-2		90.8	0.35	30	1336	55	1281	1.82	862	2480	2327	153

✓ Significant increase in Vtest from diagonal cracking to ultimate condition.

✓ Transverse reinforcement not yielding for most of the columns (could be due to strain gauges not at critical locations)

Experimental Result

- Column with 0.2 axial load ratio

Columns	$\frac{P}{A_g f_c'}$	f_c'	Diagonal Cracking					Ultimate Condition				
			Drift (%)	σ_{st} (MPa)	V_{test} (kN)	$V_{s\,test}$ (kN)	$V_{c\,test}$ (kN)	Drift (%)	σ_{st} (MPa)	V_{test} (kN)	$V_{s\,test}$ (kN)	$V_{c\,test}$ (kN)
B-1	15%	108.3	0.45	18	1862	10	1852	0.59	223	2078	165	1913
B-2	18%	125.0	0.41	20	2007	11	1996	0.50	183	2298	195	2103
B-3	20%	112.8	0.40	16	2098	17	2081	0.54	214	2418	411	2007
B-4		121.0	0.42	18	2103	14	2089	0.64	380	2528	522	2006
E-3		79.0	0.52	47	1999	70	1929	0.52	47	1999	70	1929
E-4		101.1	0.50	22	2167	70	2097	1.19	777	2856	2597	259

- ✓ The difference between Vtest at diagonal cracking and that at ultimate condition reduces as the increase of axial load.
- ✓ Transverse reinforcement not yielding for most of the columns (could be due to strain gauges not at critical locations)

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Experimental Result

- Column with 0.3 axial load ratio

Columns	$\frac{P}{A_g f_c'}$	f_c'	Diagonal Cracking					Ultimate Condition				
			Drift (%)	σ_{st} (MPa)	V_{test} (kN)	$V_{s\,test}$ (kN)	$V_{c\,test}$ (kN)	Drift (%)	σ_{st} (MPa)	V_{test} (kN)	$V_{s\,test}$ (kN)	$V_{c\,test}$ (kN)
C-1	30%	104.1	0.42	28	2036	45	1991	0.42	28	2036	45	1991
C-2		138.8	0.60	28	2958	39	2919	0.60	28	2958	39	2919
C-3		104.6	0.38	28	2153	56	2097	0.70	602	2210	1140	1070
C-4		130.0	0.62	32	3018	68	2950	0.62	32	3018	68	2950

- ✓ Vtest at diagonal cracking is equal to that at ultimate condition for C1, C2, and C3 (failure right after diagonal cracking).
- ✓ Transverse reinforcement not yielding for the only column that does not show failure right after diagonal cracking (could be due to strain gauges not at critical locations)

Experimental Result

- Column with 0.4 axial load ratio

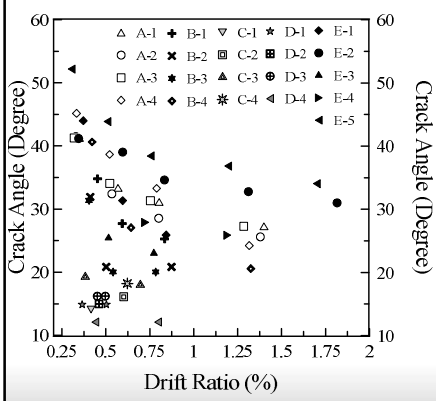
Columns	$\frac{P}{A_g f_c^{'}}$	$f_c^{'}$	Diagonal Cracking					Ultimate Condition				
			Drift (%)	σ_{st} (MPa)	V_{test} (kN)	$V_{s\,test}$ (kN)	$V_{c\,test}$ (kN)	Drift (%)	σ_{st} (MPa)	V_{test} (kN)	$V_{s\,test}$ (kN)	$V_{c\,test}$ (kN)
D-1	40%	101.0	0.37	30	2239	46	2193	0.37	30	2239	46	2193
D-2		125.5	0.46	24	2486	36	2450	0.46	24	2486	36	2450
D-3		106.4	0.45	32	2355	77	2278	0.45	32	2355	77	2278
D-4		127.8	0.44	28	2547	92	2455	0.44	28	2547	92	2455

✓ All the column showed failure right after diagonal cracking.

✓ Meaningless to discuss stress in transverse reinforcement at the ultimate condition.

Experimental Result

- Shear cracking angles



$\frac{P}{A_g f_c^{'}}$	Diagonal Crack	Ultimate Condition
0.1	44 (Range: 41-52)	33 (Range: 26-39)
0.2	32 (Range: 25-41)	24 (Range: 28-20)
0.3	17 (Range: 14-19)	17 (Range: 14-19)
0.4	15 (Range: 12-16)	15 (Range: 12-16)

Conclusions from NCREE tests

- A term to include axial load effect in the minimum shear reinforcement to prevent failure right diagonal cracking is needed.
- Design stress in transverse reinforcement for shear may have to be limited to be less than the yield stress.
- Cracking angles are less than 45 degrees.

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Database of New RC columns

78 shear-critical columns

Researcher	No Column	$(f_c^{'})$	(f_{yl})	(f_{yt})	$(P/A_g f_c^{'})$	a/d	(ρ_l) %	(ρ_t) %
Sakaguchi (1990)	6	77-93	957	1360-1400	0.24-0.48	1.25	2.14	0.15-0.80
Maruta (2008)	14	121-130	1030	1053-1450	0.15-0.60	1.25	3.80	0.30-1.80
Takami (1997)	6	99	999	757	0.13-0.51	1.25	3.80	0.51-1.02
Takaine (2010)	4	188-205	1106	1313-1334	0.30-0.60	1.25-1.56	5.70	0.45
Kuramoto (1992)	4	113.8	736	407-766	0.17-0.33	1.88	3.80	1.19
Aoyama (2001)	8	73.5	757	402-1091	0.17-0.33	1.88	3.80	0.53-1.19
Shinohara (2008)	5	98-122	1034	596-1425	0.10-0.28	1.50	3.17	0.38 -0.78
Akihiko (1990)	6	114	736	406-765	0.17-0.33	1.88	3.80	0.53 - 1.19
Sibata (1997)	5	118	685	870	0.30	1.25-1.88	3.52	0.20 - 1.14
Ou	20	79 - 130	735	862	0.10-0.40	1.88	3.38-3.52	0.15 - 0.98

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Database of New RC columns

Ranges of design parameters of the database (78 columns)

Parameter	Range
Concrete compressive strength $(f_c^{'})$	73-205 MPa
Yield strength of long. Reinforcement (f_{yl})	685-1106 MPa
Yield strength of long. Reinforcement (f_{yt})	402-1450 MPa
Axial load ratio $(P/A_g f_c^{'})$	0.1 – 0.6
Aspect ratio (a/d)	1.25 - 1.88
Ratio of long. Reinforcement (ρ_l)	2.14% - 5.70%
Ratio of trans. Reinforcement (ρ_t)	0.15%– 1.80%

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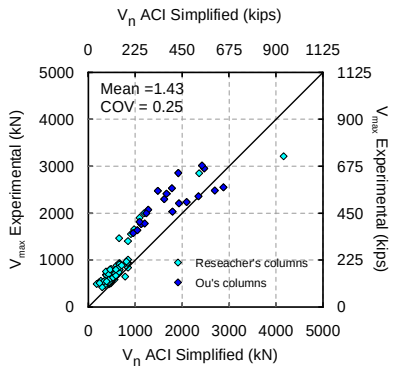
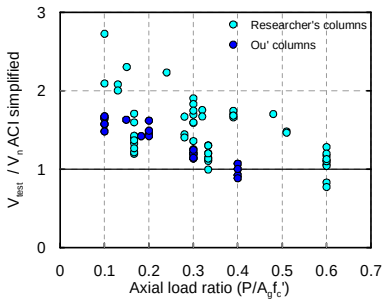
Evaluation of ACI shear strength

- Examination of ACI simplified shear strength with the database

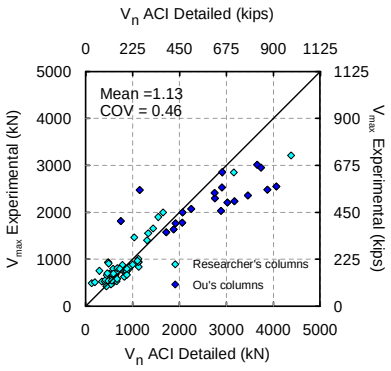


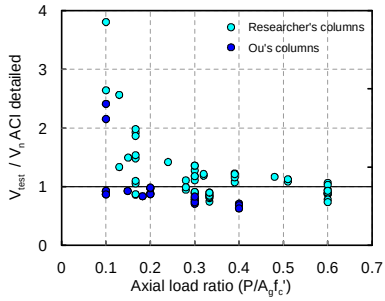


Conservative for most of the columns (73/78 conservative prediction).

Evaluation of ACI shear strength

- Examination of ACI detailed shear strength with the database





45/78 columns show unconservative prediction.

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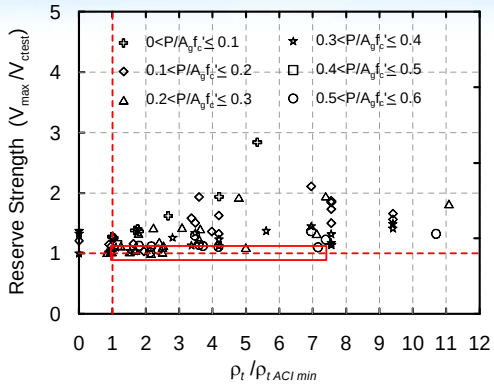
Evaluation of ACI minimum shear reinforcement



The ACI minimum shear reinforcement does not provide similar reserve strength (Vmax/Vctest) for different axial load ratio. As the axial load ratio increases, the reserve strength decreases.

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Conclusions for database evaluation

- ACI simplified shear strength is conservative for most of the columns.
- ACI detailed shear strength is not conservative for most of the columns.
- ACI minimum shear reinforcement can not reflect the effect of axial load on reserve strength

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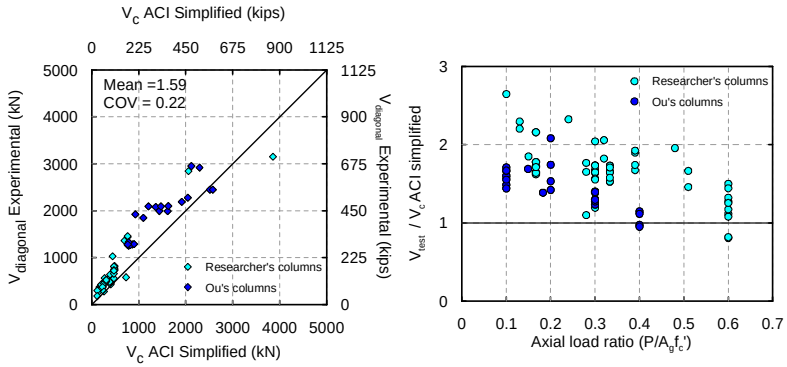
Shear strength of concrete Vc

ACI simplified concrete shear strength



Conservative for most of the columns (74/78 conservative prediction).

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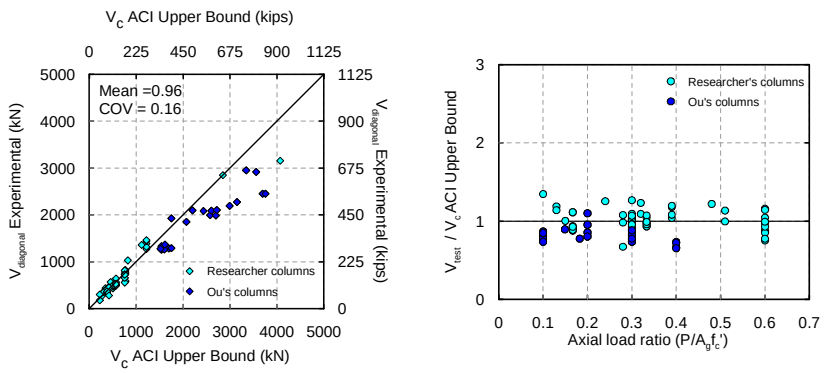
Shear strength of concrete

ACI detailed concrete shear strength (upper bound)



49/78 columns show unconservative prediction.

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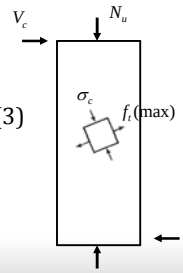
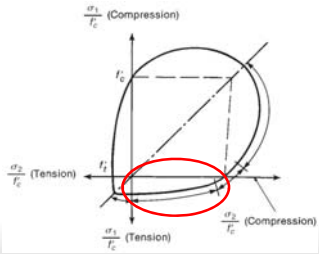
Proposed modification to ACI detailed upper bound concrete shear strength

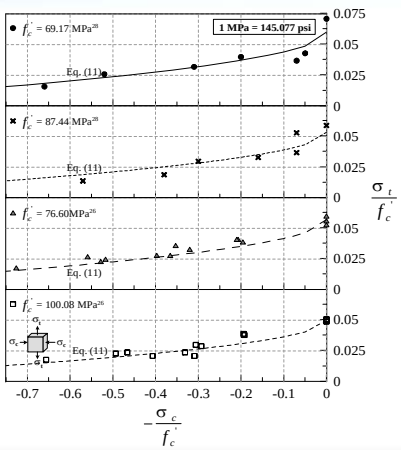
- The ACI detailed upper bound equation was derived based on principal tensile cracking strength. The equation was modified to consider the softening effect of axial compression on principal tensile strength

$$V_c = \frac{f_t(\max)}{F_2} b_w d \sqrt{1 + \frac{N_u}{f_t(\max) b_w d}} \quad (3)$$





Softening coefficient



The softening coefficient is based on biaxial test results of high-strength concrete

$$\alpha = \left(1 - 0.85\sqrt{\frac{\sigma_c}{f_c'}}\right) \quad \text{for} \quad 0 \le \frac{\sigma_c}{f_c'} \le 0.6$$

Since $\sigma_c \approx P/A_g$, the above equation can be simplified as

$$\alpha = \left(1 - 0.85\sqrt{\frac{P}{A_g f_c'}}\right) \quad \text{for} \quad 0 \le \frac{P}{A_g f_c'} \le 0.6$$

Proposed equation

- Proposed upper bound equation for detailed calculation of concrete shear strength

$$V_c = 0.29\alpha\sqrt{f_c^{'}}b_w d\sqrt{1+\frac{2P}{\alpha\sqrt{f_c^{'}}b_w d}}$$

$$\alpha = \left(1-0.85\sqrt{\frac{P}{A_g f_c^{'}}}\right) \quad \text{for} \quad 0 \le \frac{P}{A_g f_c^{'}} \le 0.6$$

Minimum shear reinforcement

- **Since minimum shear reinforcement is to ensure successful redistribution of internal forces after diagonal cracking, it should be related to diagonal cracking strength (V_c).**

$$V_{s,\min} \propto V_c$$

- **A coefficient 0.38 is used so that when axial load is zero, the equation reduces to the current ACI $V_{s\,\min}$.**

$$V_{s,\min} = \frac{A_{v,\min} f_{yt} d}{s} = 0.38V_c$$

Proposed concrete shear strength equations Vc

- Simplified shear strength equation

$$V_n = V_c + V_s = 0.17\left(1+\frac{P}{14A_g}\right)\sqrt{f_c^{'}}bd + \frac{A_v f_{yt} d}{s}$$

- Detailed shear strength equation

$$V_n = V_c + V_s = \left(0.16\sqrt{f_c^{'}} + 17\rho_w \frac{V_u d}{M_n}\right)bd + \frac{A_v f_{yt} d}{s} \quad \le 0.29\alpha\sqrt{f_c^{'}}\sqrt{1+\frac{2P}{\alpha\sqrt{f_c^{'}}bd}}bd + \frac{A_v f_{yt} d}{s}$$

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Proposed reinforcement shear strength equations Vs

- Minimum shear reinforcement

$$A_{v,\min} = 0.38\frac{V_c s}{f_{yt} d}$$

- Maximum shear strength of reinforcement

$$V_{s,\max} \le 0.66\sqrt{f_c^{'}}bd$$

- Maximum yield strength of shear reinforcement

$$f_{yt} \le 600\ MPa$$

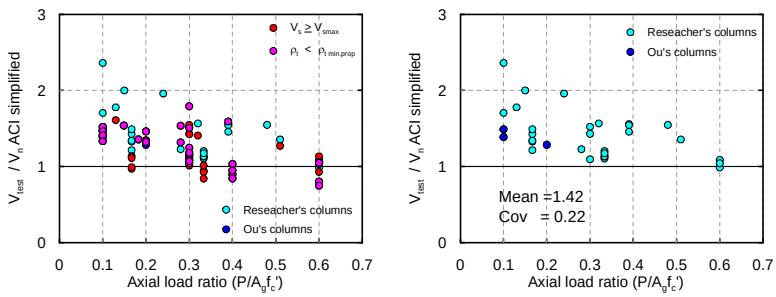
TAIWAN TECH 國立臺灣科技大學

Outline

- Current ACI shear strength equations and limitations
- NCREE shear tests on New RC columns
- New RC column shear strength database
- Evaluation of ACI shear strength equations using the database
- Proposed changes to ACI shear strength equations for New RC columns
- Evaluation of proposed changes using the database
- Proposed shear strength equations

Simplified shear strength equations

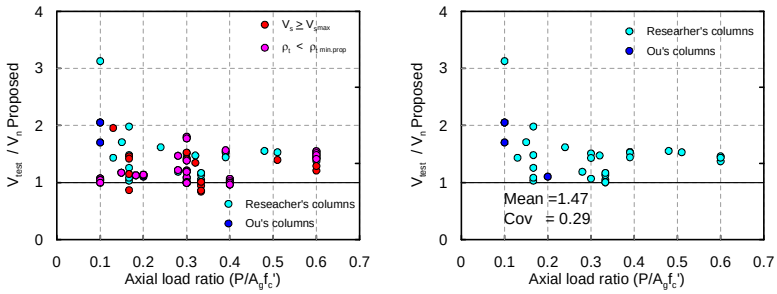
$$V_n = V_c + V_s = 0.17\left(1+\frac{P}{14A_g}\right)\sqrt{f_c'}bd + \frac{A_v f_{yt} d}{s}$$



After applying Vsmax and Vsmin, 47 columns were removed. For the 31 columns left, only 1 showed unconservative prediction with Vtest/Vn=0.99

Detailed shear strength equations

$$V_n = V_c + V_s = \left(0.16\sqrt{f_c^{'}} + 17\rho_w \frac{V_u d}{M_n}\right) bd + \frac{A_v f_{yt} d}{s} \le 0.29\alpha\sqrt{f_c^{'}}\sqrt{1+\frac{2P}{\alpha\sqrt{f_c^{'}}bd}}\,bd + \frac{A_v f_{yt} d}{s}$$



All 31 columns showed conservative prediction.

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Outline

- Current ACI shear strength equations and limitations
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Proposed shear strength equations for New RC Columns

- Shear strength of concrete

Provision	ACI 318-11	Proposed
Simplified shear strength of concrete	$V_c = 0.17\left(1+\frac{P}{14A_g}\right)\sqrt{f_c^{'}}bd$	
Detailed shear strength of concrete	$V_c = \left(0.16\sqrt{f_c^{'}} + 17\rho_w \frac{V_u d}{M_m}\right)bd$ $M_m = M_u - P\left(\frac{4h-d}{8}\right)$	
Upper Bound shear strength of concrete	$V_c = 0.29\sqrt{f_c^{'}}bd\sqrt{\left(1+\frac{0.29P_u}{A_g}\right)}$	$V_c = 0.29\alpha\sqrt{f_c^{'}}\sqrt{1+\frac{2P}{\alpha\sqrt{f_c^{'}}bd}}bd$ $\alpha = \left(1-0.85\sqrt{\frac{P}{A_g f_c^{'}}}\right)$ for $0 \le \frac{P}{f_c^{'}A_g} \le 0.6$

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Proposed shear strength equations for New RC Columns

- Shear strength of reinforcement

Provision	ACI 318-11	Proposed
Limit stress of shear reinforcement	$f_{yt} \le 420\ MPa$	$f_{yt} \le 600\ MPa$
Shear strength of reinforcement	$V_s = \frac{A_v f_y d}{s}$	
Minimum shear reinforcement	$A_{v,\min} = 0.062\sqrt{f_c^{'}}\frac{bs}{f_{yt}} \ge \frac{0.35bs}{f_{yt}}$	$A_{v,\min} = 0.38V_c\frac{s}{df_{yt}}$
Maximum shear strength of reinforcement	$V_{s,\max} \le 0.66\sqrt{f_c^{'}}bd$	

THANK YOU

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高拉力強度鋼纖維於高強度鋼筋混凝土柱之應用評估

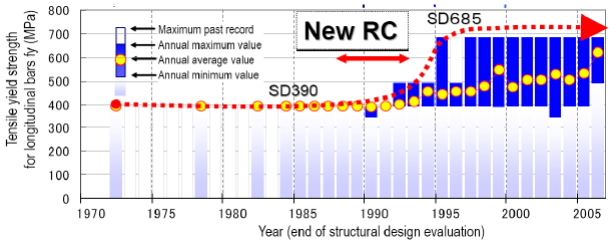
廖文正
國立台灣大學 土木工程學系

Outline

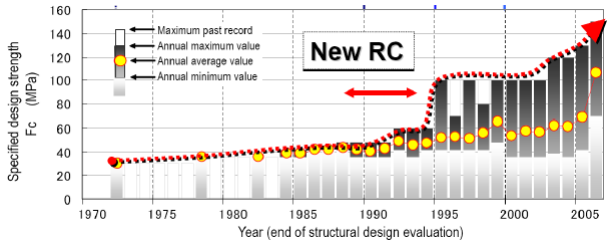
- Background and Objective
- Highly Flowable Strain Hardening Fiber Reinforced Concrete (HF-SHFRC)
- Proposed Toughness Ratio / Shear Strength Equation
- Experimental Programs
- Conclusions

Background and Objective

New RC Project



Change in Yield Strength of Longitudinal Bars



Change in Specified Concrete Strength

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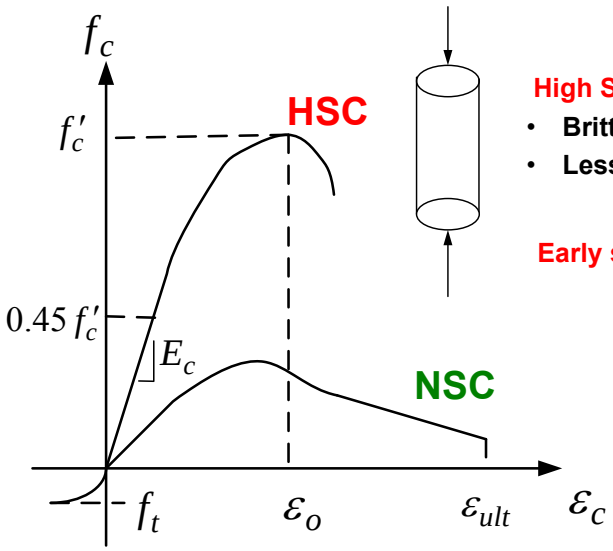
Background and Objective

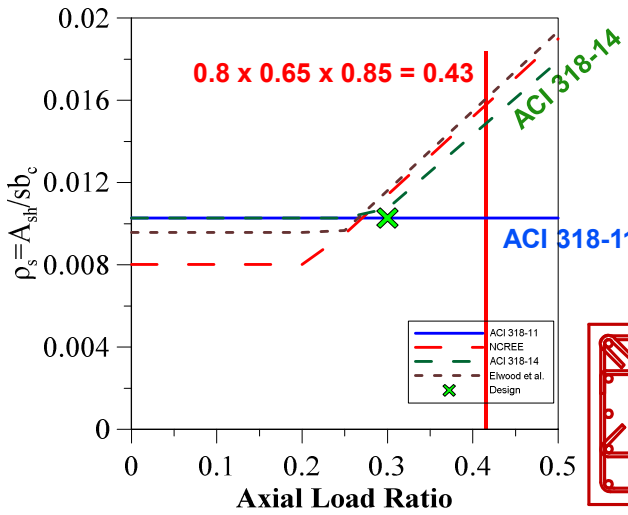


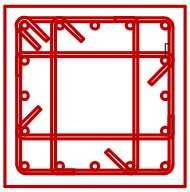
High Strength Concrete

- **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！！

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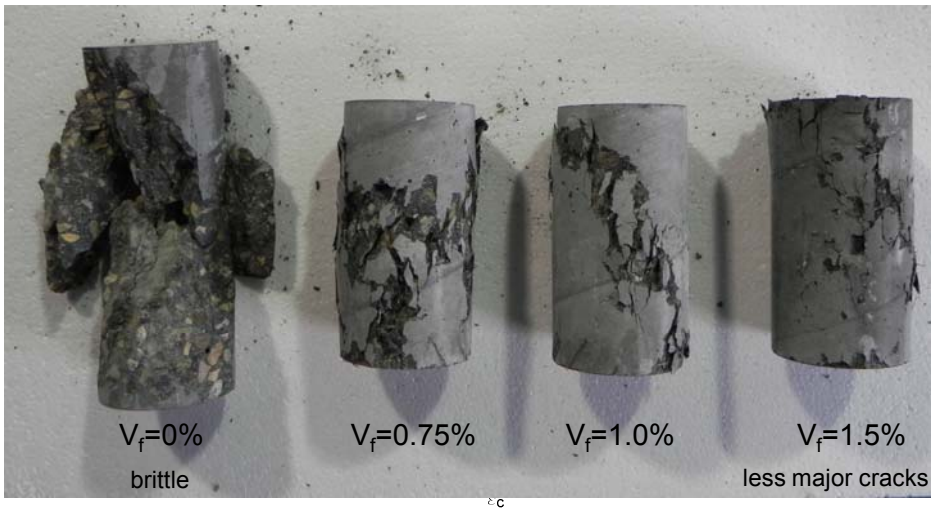
Background and Objective

Alternative solution: Fibers



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

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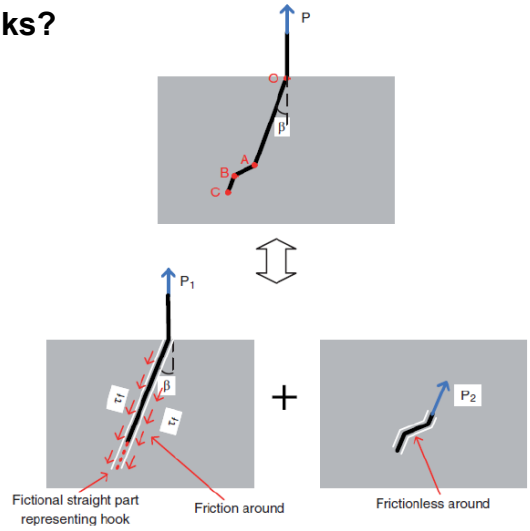
Key parameters of Fiber Reinforced Concrete

V_f Fiber volume fraction

α_f Fiber Aspect Ratio (L/d)

τ_b bond strength between fiber and matrix

Why τ_b talks?

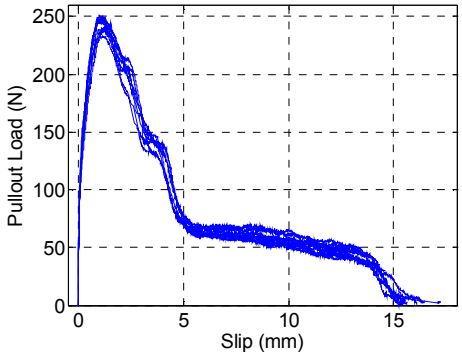


Pullout Energy = Frictional Energy + Plastic Deformation Energy
(Straight part) (Hooked-end part)

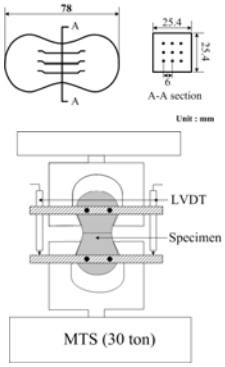
Selection of fibers

Fiber Type		Diameter (mm)	Length (mm)	Density (g/cc)	Aspect ratio	**Tensile Strength (MPa)**	Elastic Modulus (GPa)
High strength		0.38	30	7.85	79	**2300**	200
Normal Strength		0.55	30	7.85	55	**1100**	200

Pullout energy



Steel Fiber Pullout Curve of 28 Days



	Experiment		Prediction (Xu, 2011)		Error (%)
	Pullout Energy (N-mm)	τ_{eq} (MPa)	Pullout Energy (N-mm)	τ_{eq} (MPa)	
Average	1369	10.20	1294	9.64	5.5
Std. Deviation	53	0.40	-	-	-

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

- **SCC**

  Self-Consolidating Concrete;
  Self-Compacting Concrete

- **HPFRCC**

  High Performance Fiber-Reinforced Cementitious Composites

a **highly workable** concrete that can flow under its own weight without segregation and vibration.

strain-hardening response under tension accompanied by **multiple cracking**

HYBRID

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Highly Flowable Strain Hardening Fiber Reinforced Concrete

Highly Flowable Strain Hardening Fiber Reinforced Concrete

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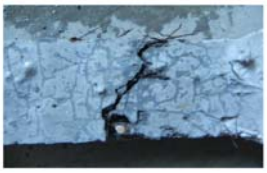
Highly Flowable Strain Hardening Fiber Reinforced Concrete



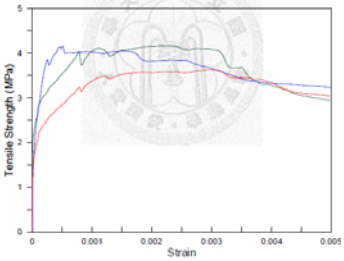


圖 5－12 C30 直接拉力應力應變圖（0.5 %）

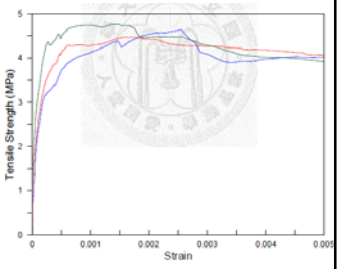


圖 5－14 C40 直接拉力應力應變圖（0.5 %）

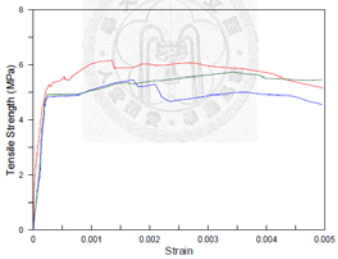


圖 5－16 C50 直接拉力應力應變圖（0.5 %）

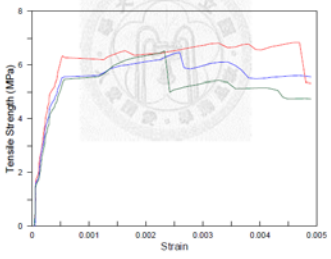
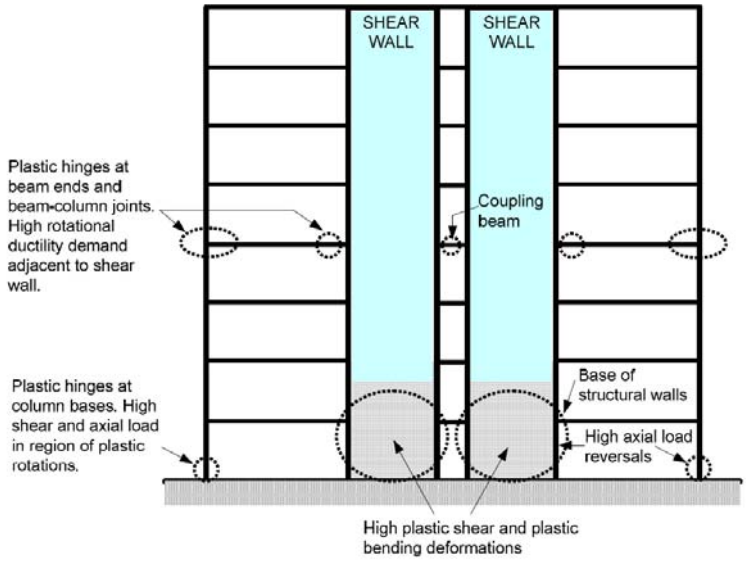


圖 5－18 C60 直接拉力應力應變圖（0.5 %）

Highly Flowable Strain Hardening Fiber Reinforced Concrete



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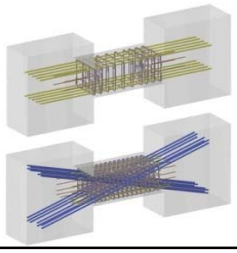
Highly Flowable Strain Hardening Fiber Reinforced Concrete



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

CONSTRUCTION INDUSTRY SPOTLIGHT

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.

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.

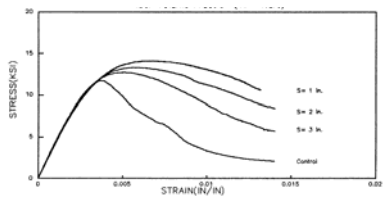
"There is a wide range of

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

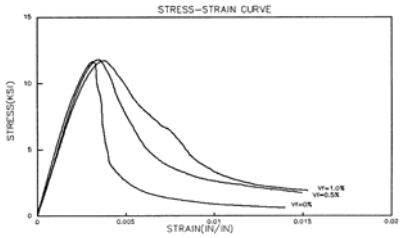
Fibers as an alternative for transverse reinforcement

1. Confinement Efficiency



Lateral reinforcement ratio ↑

Ductility ↑



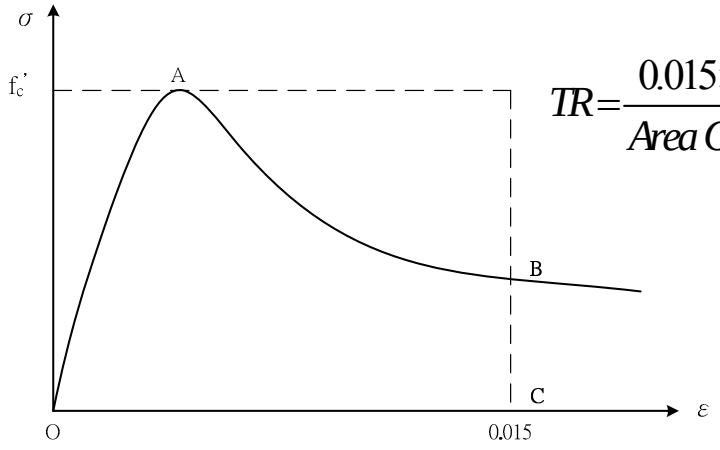
Fiber Volume, aspect ratio, τ_b ↑

Ductility ↑

2. Shear Capacity

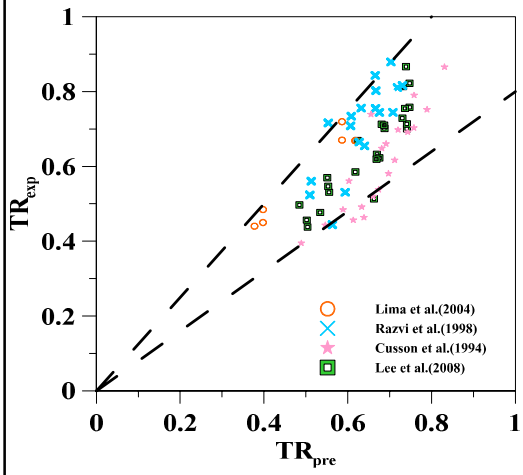
Confinement Efficiency

Toughness Ratio, TR



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

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

Shear Capacity

- **ACI 318-11**

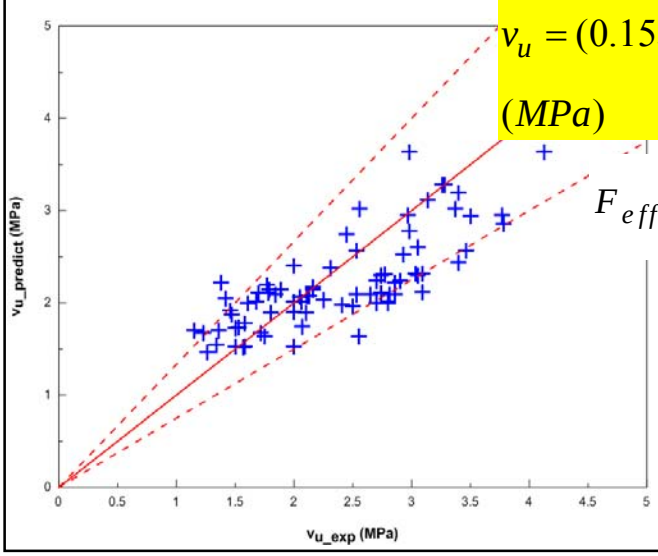
11.4.6 — **Minimum shear reinforcement**

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 $3.5\sqrt{f_c'}\,b_w d$.

Proposed Shear Strength Equation



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

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

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

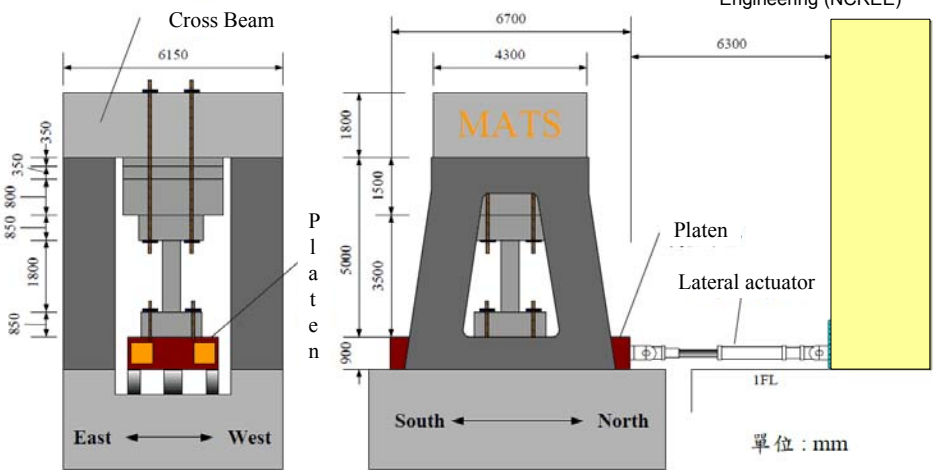
Experimental Program

Experimental Program

Multi-Axial Resting System (MATS)

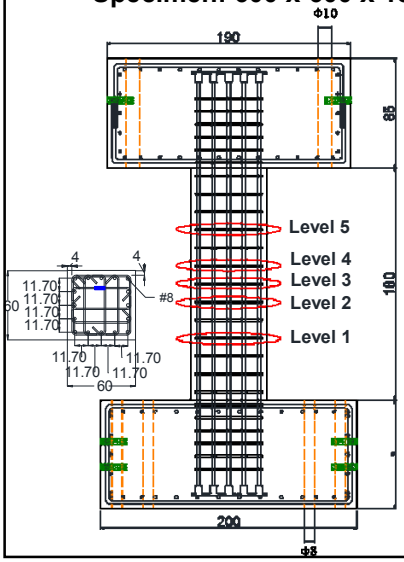
Axial loading capacity: up to 60 MN

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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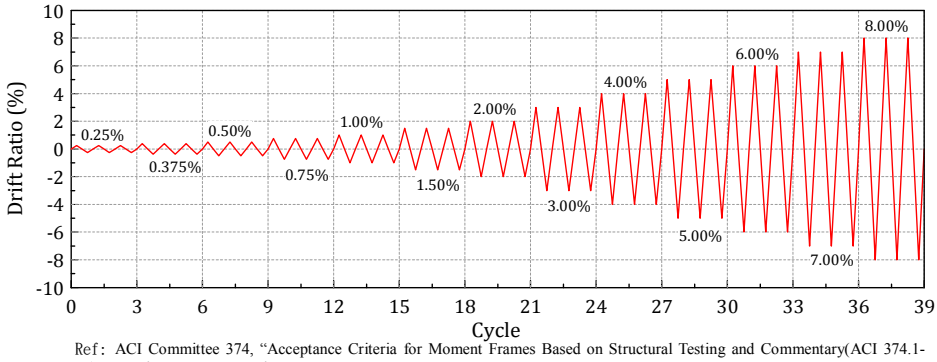
Experimental Program

Loading Protocol

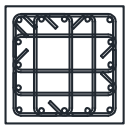
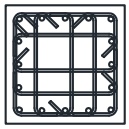
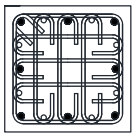


Ref: ACI Committee 374, "Acceptance Criteria for Moment Frames Based on Structural Testing and Commentary(ACI 374.1-05)," American Concrete Institute, 2006.

Criteria:

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

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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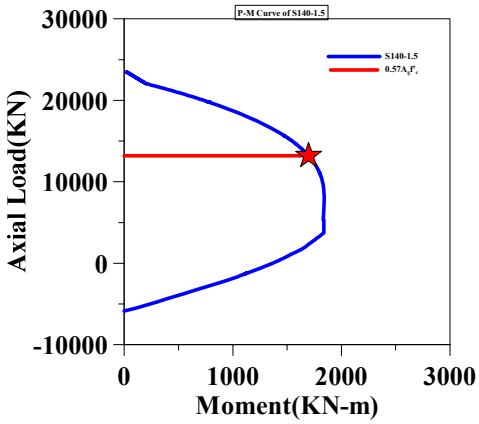
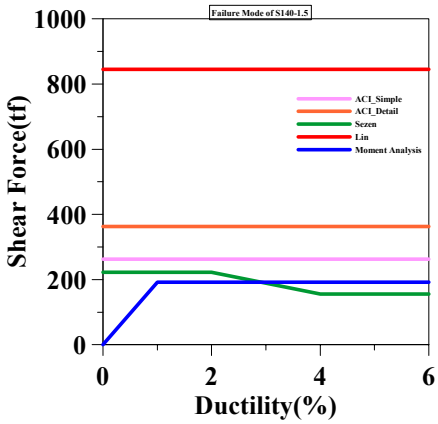
Experimental Program

Failure modes prediction of S140-0 and S140-1.5





Test Results
S140-0 VS. S140-1.5

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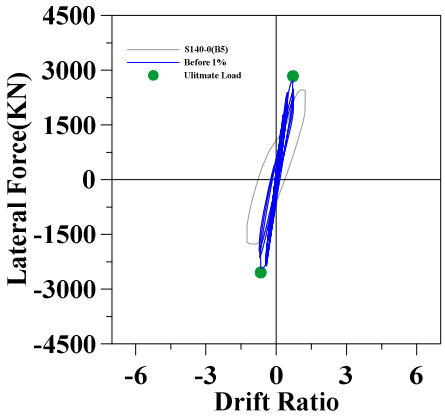
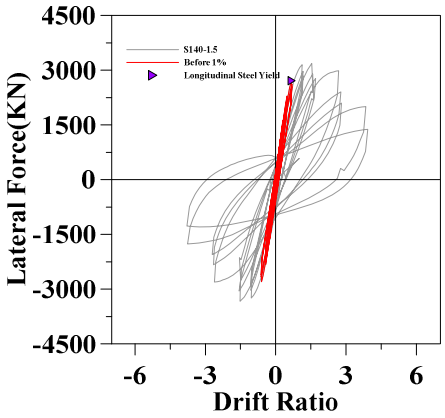
Test Results

S140-0

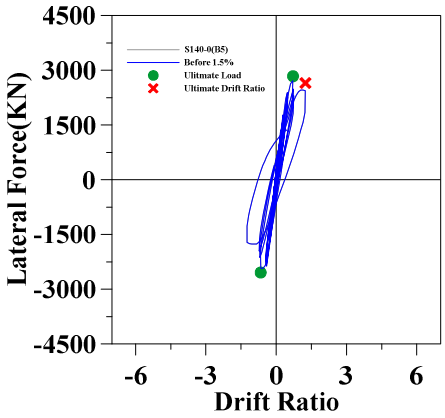


S140-1.5

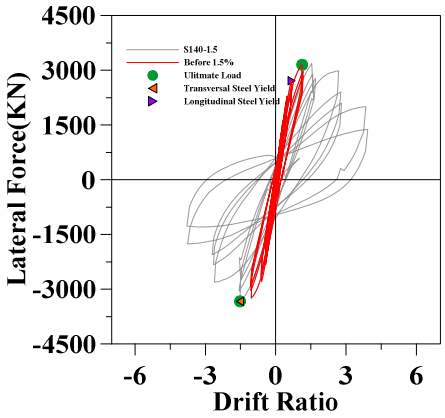


Test Results

S140-0



S140-1.5



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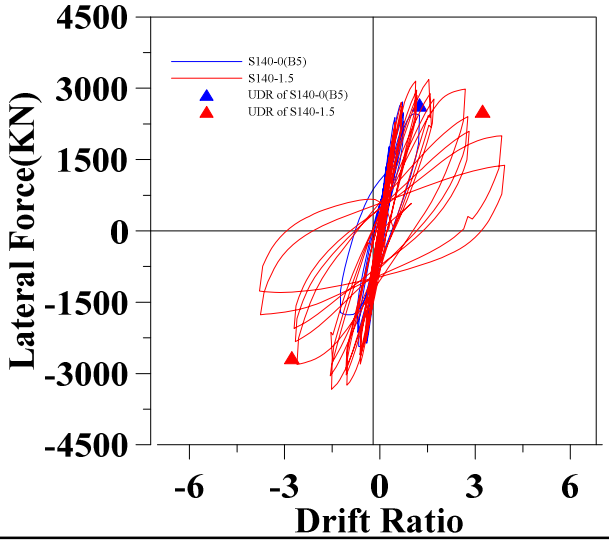
Test Results

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



Test Results

Drift ratio: 1.0%

S140-0 (Max Lateral Capacity)

S140-1.5

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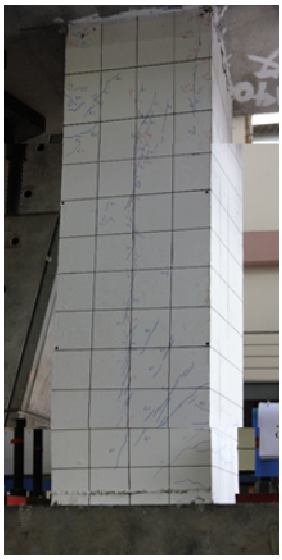
Test Results

Drift ratio: 1.5%

S140-0

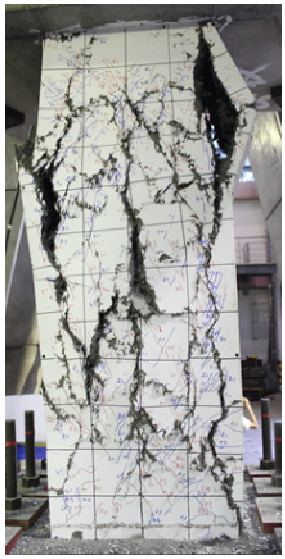


S140-1.5 (Max Lateral Capacity)

Test Results

Failure

S140-0 (2nd cycle of 1.5%)



S140-1.5 (4.0%)

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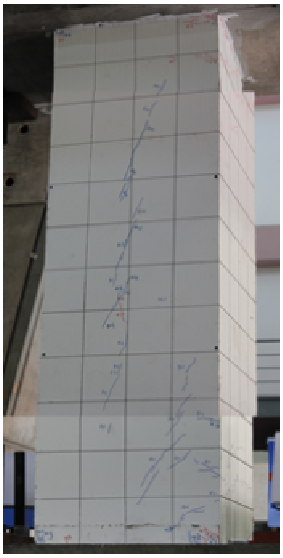
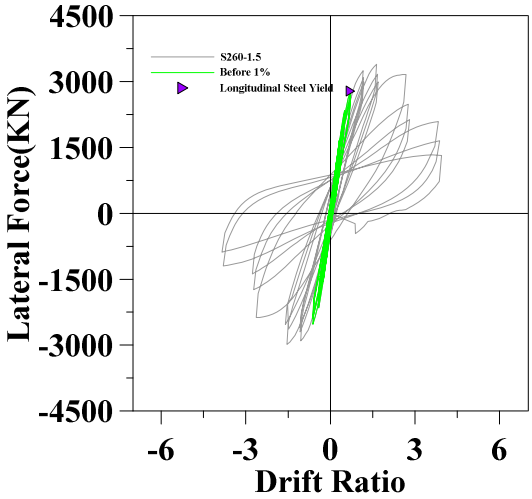
Test Results
S260-1.5 (spacing D/2)

Test Results



Test Results (S260 – 1.5) 1.0%



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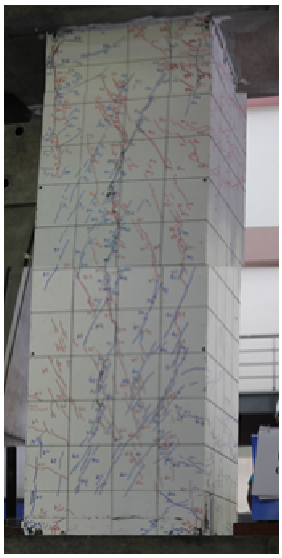
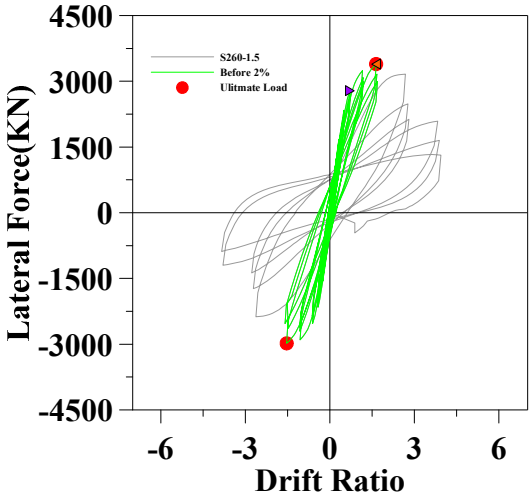
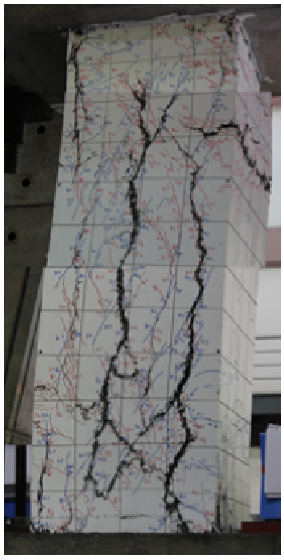
Test Results



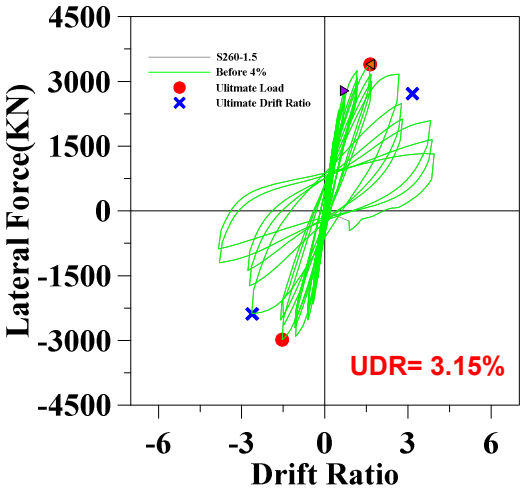
Test Results (S260 – 1.5) 2.0%



Test Results



Test Results (S140 – 1.5) failure



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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**.

Thank you ☺

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PRELIMINARY DESIGN RECOMMENDATION FOR FLEXURAL MEMBER WITH HIGH STRENGTH MATERIALS

Min-Yuan Cheng
National Taiwan University of Science and Technology

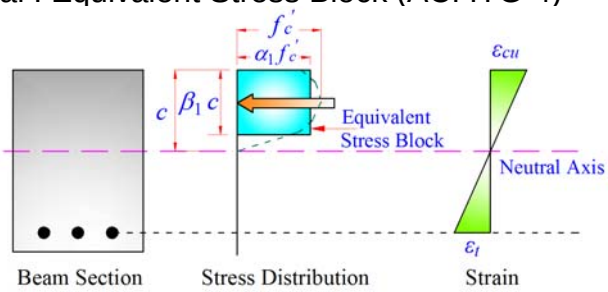
高強度鋼筋混凝土結構桿件設計研討會 December 5th, 2014

Outline

- Design Equation
- Experimental Verification
- Issues
- Conclusions

Design Equation (Flexure)

Flexural : Equivalent Stress Block (ACI ITG-4)



f_c'	α_1
$\leq$ 55 MPa	0.85
$>$ 55 MPa	$0.85-0.0022\,(f_c'-55) \geq 0.70$

$$\beta_1 = 0.85-0.05\,\frac{(f_c'-28)}{7} \geq 0.65$$

$$\varepsilon_{cu} = 0.003$$

remains the same

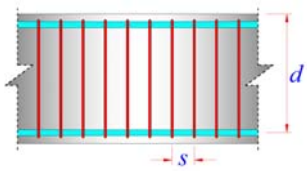
Design Equation (Shear)

$$V_n = V_c + V_s$$

$$V_c = 0 \;;$$

$$V_s = \frac{A_v . f_{yt} . d}{s}$$

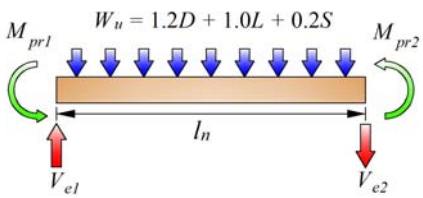
$f_{yt} \leq 600$ MPa



V_u : Evaluated from M_{pr} using $f_y = 1.25 f_y$

For SD685 longitudinal bars use 1.25 x 685 MPa

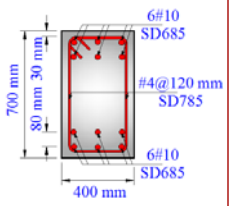
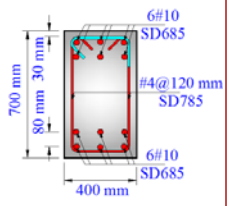
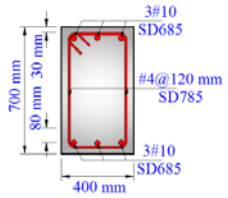
In general



$$V_u = \frac{M_{pr1} + M_{pr2}}{l_n} + \frac{W_u . l_n}{2}$$

V_u = larger (V_{e1}, V_{e2})

Experimental Verification

	SP1	SP2	SP3
Beam Design			
M_{pr} (kNm)	2223.55	2223.55	1252.76
$V_u = \frac{M_{pr}}{a}$ (kN)	1058.64	1058.64	596.44
$V_n = \frac{A_{vt} \cdot f_{yt} \cdot d}{s}$ (kN)	771.32	771.32	822.64
V_n / V_u	0.73	0.73	1.38
s (d_b)	3.75	3.75	3.75
a (mm)	2100	2100	2100
a/d	3.5	3.5	3.3

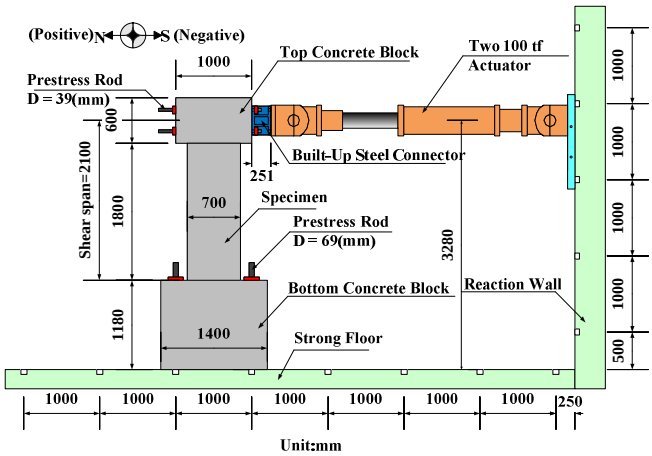
Specified Material Properties:

$f_c' = 68.95$ MPa and $f_y = 1.25 \times 685$ MPa

$f_{yt} = 600$ MPa

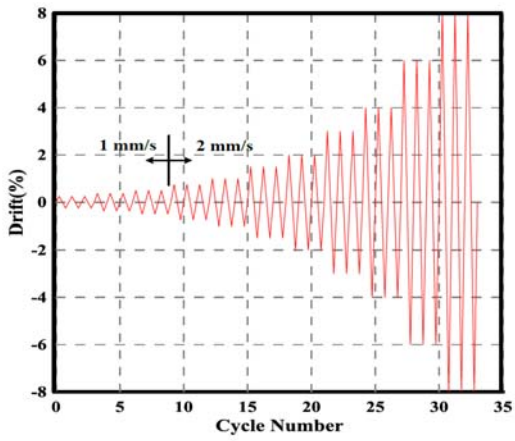
Experimental Verification

Experimental Setup



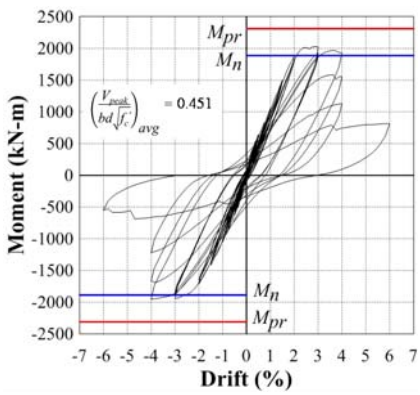
Experimental Verification

Loading History



Experimental Verification

Test Result SP1



flexure-shear

3 %
(West)

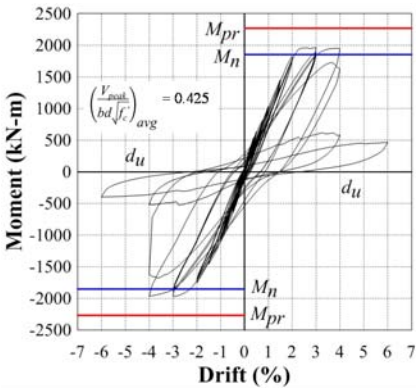
$(d_u)_{avg}$ = 4.56 %
(West)

4 %
(West)

$(d_u)_{avg}$ = 4.56 %
(North)

Experimental Verification

Test Result SP2



flexure-shear

3 %
(West)

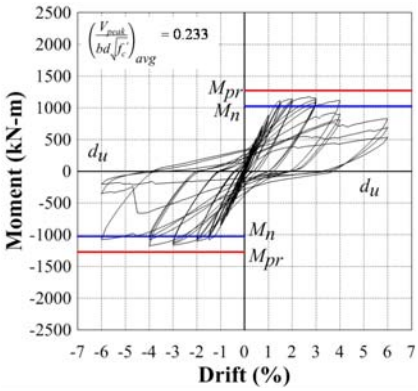
$(d_u)_{avg}$ = 4.50 %
(West)

4 %
(West)

$(d_u)_{avg}$ = 4.50 %
(North)

Experimental Verification

Test Result SP3



flexure

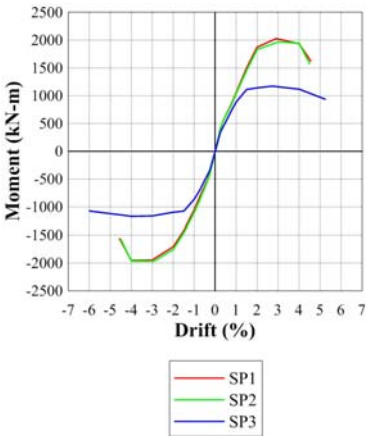
3 %
(West)

$(d_u)_{avg}$ = 5.61 %
(West)

4 %
(West)

$(d_u)_{avg}$ = 5.61 %
(North)

Experimental Verification

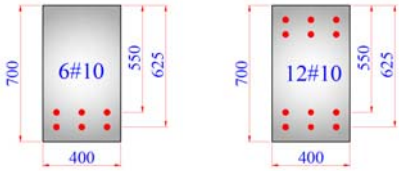


Envelope from first cycle hysteresis loop

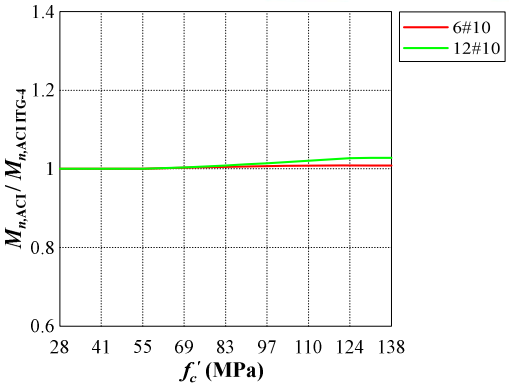
	Strength		Deformation
	$\left(\frac{M_n}{M_{exp}}\right)_{avg}$	$\left(\frac{M_{pr}}{M_{exp}}\right)_{avg}$	$(d_u)_{avg}$ (%)
SP1	0.948	1.160	4.56
SP2	0.942	1.153	4.50
SP3	0.875	1.085	5.61

SP1 SP2 SP3

Design Issues (Transition of Stress Block)

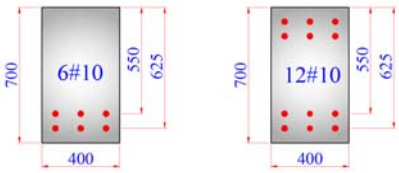


f_y = 414 MPa
unit : mm

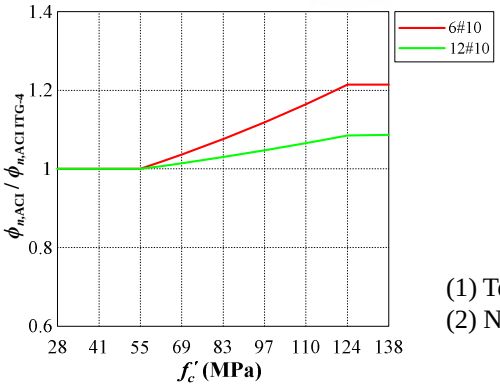


	ACI 318-11	ACI ITG-4
f_c' (MPa)	α_1	α_1
28 - 55	0.85	0.85
62 - 117		0.84 – 0.72
124 - 138		0.70

Design Issues



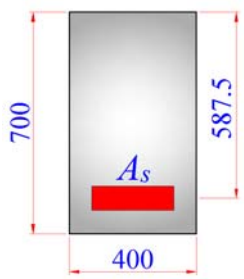
f_y = 414 MPa
unit : mm

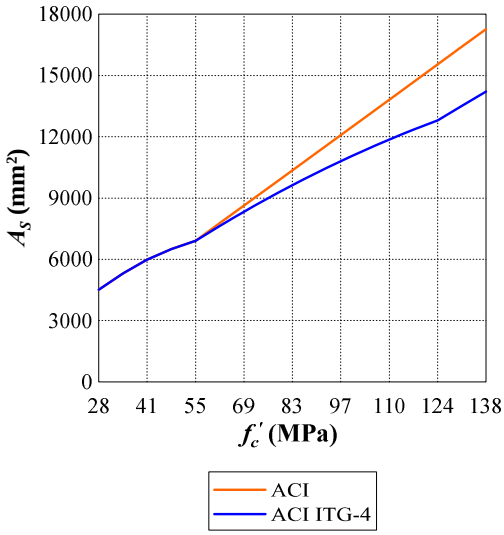


	ACI 318-11	ACI ITG-4
f_c' (MPa)	α_1	α_1
28 - 55	0.85	0.85
62 - 117		0.84 – 0.72
124 - 138		0.70

(1) Tension-control limit
(2) Need of Special Boundary Element

Design Issues





Conclusion

Preliminary of Design Equation are proposed for flexural members with High Strength Material. Through experimental verification, the following conclusion is drawn:

1. Nominal flexural strength of the beam specimens, M_n, can be conservatively evaluated using the modified equivalent concrete stress block provided by ACI ITG4 and elastic-perfectly-plastic steel properties for SD685 high-strength longitudinal steel.

2. Shear design with the provided shear capacity greater than the demand from nominal flexural strength, $V_n > V_u$, M_{pr}, seems satisfactory to prevent the shear failure mode. In which, shear capacity of the beam is attributed to transverse reinforcement only with $f_{yt} \leq 600$ Mpa.

3. Experimental results show that specimens with shear reinforcement in either one-piece hoop or two-piece hoop configuration exhibit equivalent cyclic behaviors in terms of strength and deformation capacities.

Conclusion

4. A minimum of 5% drift can be achieved for a high-strength flexural beam specimen that fails in flexural governed mode.

5. Future research is needed to verify steel strain limit for tension-controlled section. A practical spacing for transverse reinforcement to prevent buckling is needed.

Thank you
for your attention !

高強度鋼筋混凝土梁撓曲剪力行為研究與設計探討

王勇智
國立中央大學土木系教授

大　綱

ACI 318-11 RC梁撓曲剪力設計法
NZS 3101-2006 RC梁撓曲剪力設計法

RC梁撓曲剪力實驗 印證

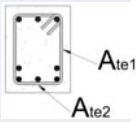
- 整體耐震行為觀察
- 裂縫角度觀察
- 箍筋應變量測
- 混凝土剪力衰減情形
- 梁拉力筋SD685超額強度係數α_0調查

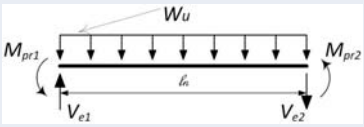
結論

ACI 318-11與NZS 3101-2006梁撓曲剪力設計

項目	ACI 318-11	NZS 3101-2006
計算 材料強度	$f_c' \le 70MPa$ f_y & $f_{yt} \le 420MPa$	$f_c' \le 60MPa$ f_y & $f_{yt} \le 500MPa$ $f_{yt} \le 800MPa$(圍束相關)
計算 剪力強度	$V_n = V_c + V_s$	$V_n = V_c + V_s$ $v_n = \frac{V_n}{b_w d}$ 非塑鉸區: $v_n \le \min[0.2f_c', 8\text{MPa}]$ 塑鉸區: $v_n \le \min[0.16f_c', 8\text{MPa}]$

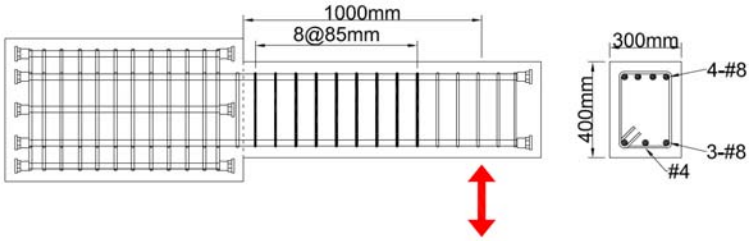
項目		ACI 318-11	NZS 3101-2006
V_c	非塑鉸區	精算式 $V_c = \left(0.16\sqrt{f_c'}17\rho_w \frac{V_u d}{M_u}\right) b_w d$ $\le 0.29\sqrt{f_c'} b_w d$ 簡算式 $V_c = 0.17\sqrt{f_c'} b_w d$	$V_c = (0.07 + 10\rho_w)\sqrt{f_c'} b_w d$ $0.08\sqrt{f_c'} b_w d \le V_c \le 0.2\sqrt{f_c'} b_w d$
	塑鉸區	假設 $V_c = 0$	一般塑鉸區 DPR (ductile plastic region, DPR) 假設$V_c = 0$ 限定塑鉸區LDPR (limited ductile plastic region, LDPR) $V_c \le 0.5V_c$
V_s		$V_s = \frac{A_v f_{yt} d}{S_v} \le 0.66\sqrt{f_c'} b_w d$ (假設斜裂縫角θ=45°) $A_{v,min} \ge$ $\max(0.062 \frac{\sqrt{f_c'} b_w s}{f_{yt}}, 0.35 \frac{b_w s}{f_{yt}})$	$V_s = \frac{A_v f_{yt} d}{S_v} \le 0.85\sqrt{f_c'} b_w d$ (假設斜裂縫角θ=45°) 非塑鉸區: $A_{vmin} \ge 0.062\sqrt{f_c'} \frac{b_w s}{f_{yt}}$ 塑鉸區: $A_{vmin} \ge 0.083\sqrt{f_c'} \frac{b_w s}{f_{yt}}$

項目		ACI 318-11	NZS 3101-2006
橫向箍筋間距限制	非塑鉸區	$S=min[S_v, d/2, 600mm]$ for $V_s \le 0.33\sqrt{f_c'}b_w d$ $S=min[S_v, d/4, 300mm]$ for $V_s > 0.33\sqrt{f_c'}b_w d$	$S=min[S_v, d/2, 600mm]$ for $V_s \le 0.33\sqrt{f_c'}b_w d$ $S=min[S_v, d/4, 300mm]$ for $V_s > 0.33\sqrt{f_c'}b_w d$ 縱向鋼筋反挫曲 $S=min[b_w, 16d_{bl}]$
	塑鉸區	$S=min[S_v, d/4, 6d_{bl}, 150mm]$ ※d_{bl}：主筋直徑	剪力箍筋間距 $S=min[S_v, d/4, 6d_{bl}, 300mm]$ 縱向鋼筋反挫曲 一般塑鉸區DPR $S=min[d/4, 6d_{bl}, 200mm]$ 限定塑鉸區LDPR $S=min[d/4, 10d_{bl}, 200mm]$ 檢核單股繫筋最小面積 $A_{te}=\frac{\sum A_{bl}f_y}{96f_{yt}}\frac{s}{d_{bl}}$ 

項目	ACI 318-11	NZS 3101-2006
地震引致之剪力 (V_e)	$V_e=\frac{M_{pr1}+M_{pr2}}{l_n}\pm\frac{w_u l_n}{2}$ M_{pr}: 由$\alpha_0 f_y$計算之超額彎矩強度 α_0為拉力筋超額強度係數 (steel overstrength factor) =1.25 	$V_e=\frac{M_{pr1}+M_{pr2}}{l_n}\pm\frac{w_u l_n}{2}$ M_{pr}: 由$\alpha_0 f_y$計算之超額彎矩強度， α_0為拉力筋超額強度係數 α_0 =1.25 SD300鋼筋 α_0 =1.40 SD500鋼筋

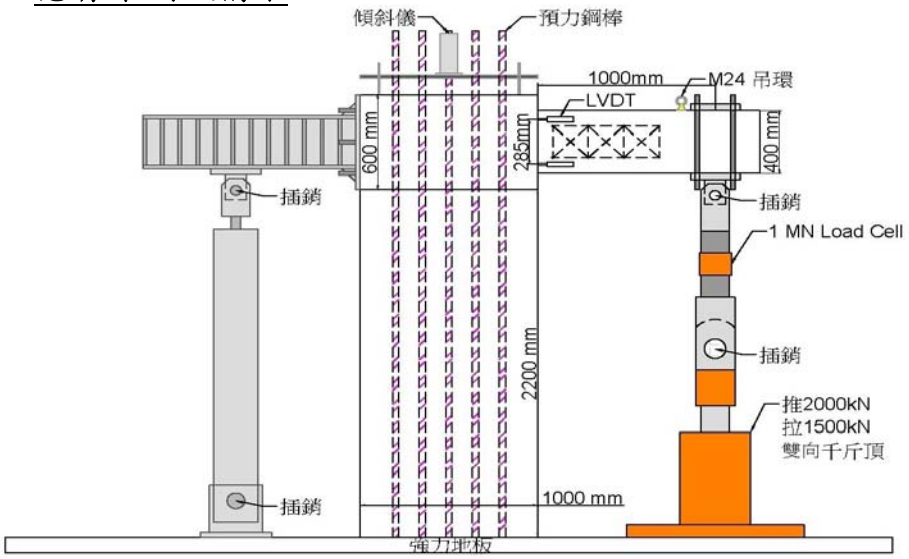
RC梁撓曲剪力實驗 印證

試體規劃



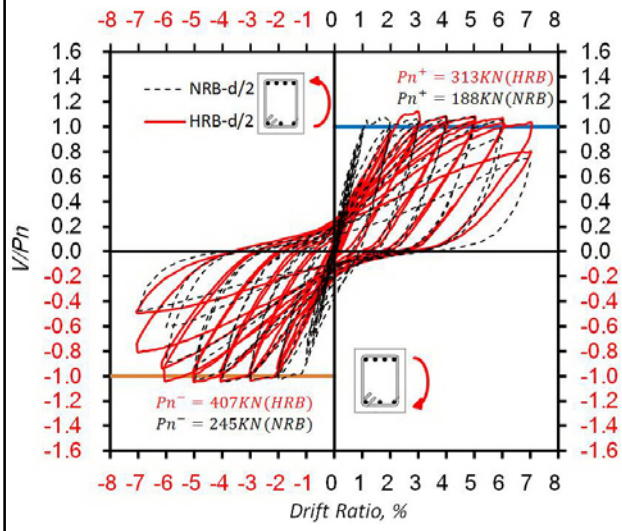
試體編號	f'_c MPa	$f_{y,\#8}$ MPa	$f_{yt,\#4}$ MPa	b,mm	d,mm	s,mm	a/d
NRB-d	30	420 (468)	420 (497)	300	335	335	3
NRB-d/2						165	
NRB-d/4						85	
HRB-d	63	685 (754)	785 (840)	300	335	335	3
HRB-d/2						165	
HRB-d/4						85	

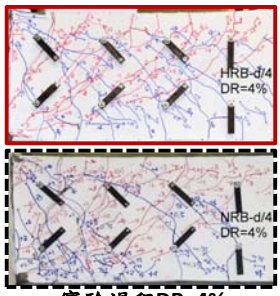
懸臂梁測試構架


傾斜儀
預力鋼棒
1000mm
M24 吊環
LVDT
600 mm
285mm
400 mm
插銷
插銷
1 MN Load Cell
插銷
2200 mm
推2000kN
拉1500kN
雙向千斤頂
插銷
1000 mm
強力地板

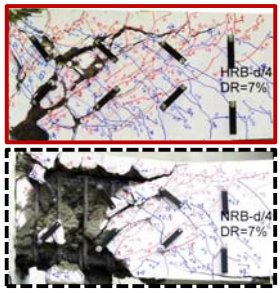
遲滯迴圈圖與裂縫圖

HRB(New RC)-d/4 & NRB(Normal RC)-d/4



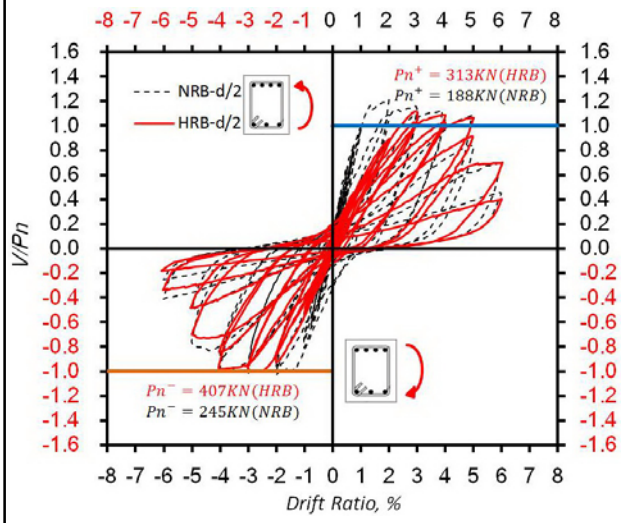


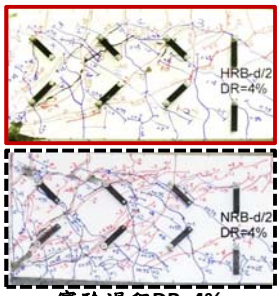
實驗過程DR=4%



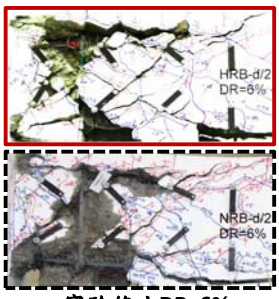
實驗終止DR=7%

HRB-d/2 & NRB-d/2



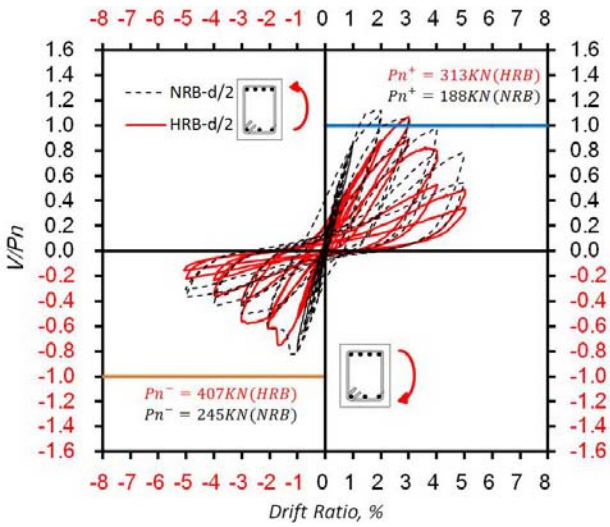


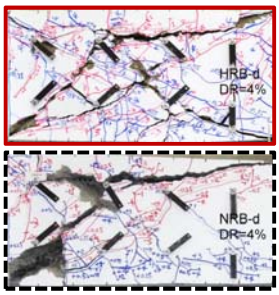
實驗過程DR=4%



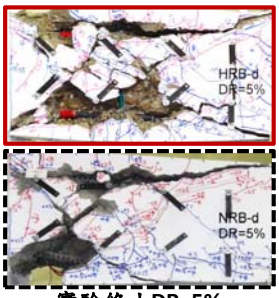
實驗終止DR=6%

HRB-d & NRB-d





實驗過程DR=4%

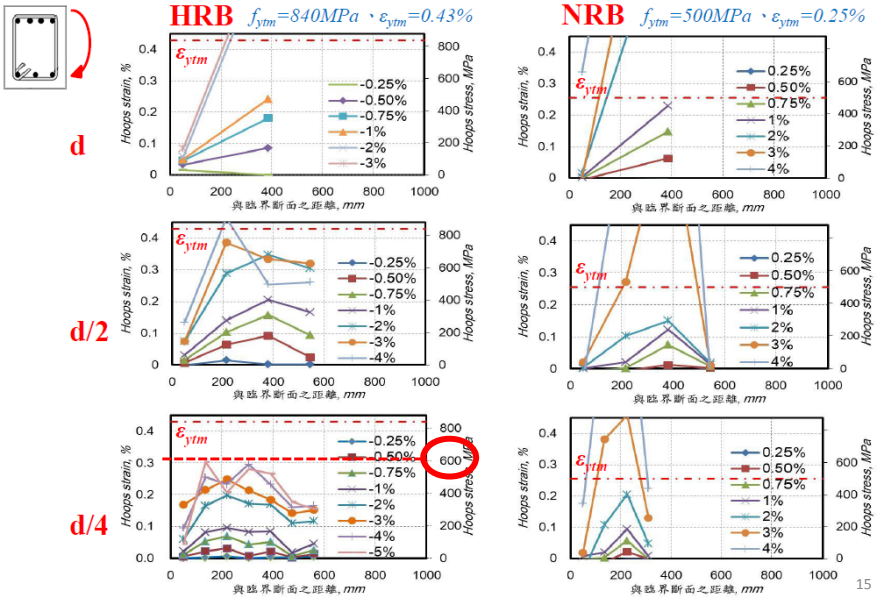


實驗終止DR=5%

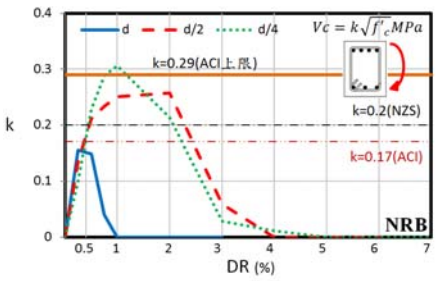
試驗結果統計

試體編號	加載方向	最大載重 P_{max} ,kN	降伏位移Δ_y mm	初始勁度K_i, kN/mm (取0.6P_{max}斜率)	斜裂縫角度, DR=4%	相對消能比β DR=4%
HRB-d	正	335	12.6	26.6	37°	23%
	負	-305	-16.6	18.4	42°	
HRB-d/2	正	350	12.7	27.5	35°	24%
	負	-409	-15.2	26.9	39°	
HRB-d/4	正	351	13.9	25.2	37°	34%
	負	-426	-17.0	25.1	40°	
NRB-d	正	211	7.54	28.0	38°	24%
	負	-203	-9.51	21.3	37°	
NRB-d/2	正	229	7.19	31.9	35°	24%
	負	-251	-7.52	33.4	39°	
NRB-d/4	正	204	6.86	29.8	37°	32%
	負	-256	-8.78	29.2	43°	

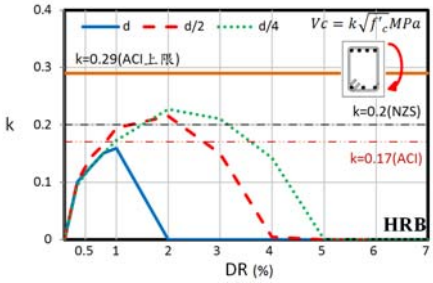
箍筋應變量測



混凝土剪力衰減情形



NRB混凝土剪力衰減曲線圖



HRB混凝土剪力衰減曲線圖

New RC梁拉力筋超額強度係數α_0

$$\alpha_0 = \frac{M_{max}}{M_n}$$

M_{max} = maximum moment occured during testing

M_n = nominal moment predicted using specified yield strength f_y and measured f_c'

New RC梁拉力筋超額強度係數α_0統計

試體編號	bw (mm)	h (mm)	a/d	ρs(%)	$\rho s'$(%)	meas. f_c' (MPa)	Spec. f_y (MPa)	Meas. f_{ym} (MPa)	f_{ym}/f_y	α_0
HRB-d/4 (+)	300	400	3.0	1.51	2.02	63	685	754	1.10	1.12
HRB-d/4 (-)	300	400	3.0	2.02	1.51	63	685	754	1.10	1.05
HRB-d/2 (+)	300	400	3.0	1.51	2.02	63	685	754	1.10	1.12
HRB-d/2 (-)	300	400	3.0	2.02	1.51	63	685	754	1.10	1.00
R685-NC (+)	250	350	6.2	2.03	2.03	56	685	754	1.10	1.16
R685-NC (-)	250	350	6.2	2.03	2.03	56	685	754	1.10	1.15
R685-C110C(+)	250	350	6.2	2.03	2.03	56	685	754	1.10	1.15
R685-C110C(-)	250	350	6.2	2.03	2.03	56	685	754	1.10	1.19
R685-C140C(+)	250	350	6.2	2.03	2.03	56	685	754	1.10	1.15
R685-C140C(-)	250	350	6.2	2.03	2.03	56	685	754	1.10	1.16
HR1a(+)	300	400	5.5	1.47	1.47	35	685	730	1.07	1.20
HR1a(-)	300	400	5.5	1.47	1.47	35	685	730	1.07	1.19
HR2c(+)	300	400	5.5	1.47	1.47	35	685	730	1.07	1.16
HR3c(+)	300	400	5.5	1.47	1.47	35	685	730	1.07	1.17
HT1b	1200	400	2.4	1.46	2.35	60	685	746.8	1.09	1.15
HT1a	1200	400	5.46	1.46	2.35	37	685	769	1.12	1.20
HT1c	1200	400	5.51	1.47	1.88	42	685	754	1.10	1.13
BS-T (+)	400	700	3.5	2.03	2.03	78	685	707	1.03	1.13

試體編號	bw (mm)	h (mm)	a/d	ρs(%)	$\rho s'$(%)	meas. f_c' (MPa)	Spec. f_y (MPa)	Meas. f_{ym} (MPa)	f_{ym}/f_y	α_0
BS-T (-)	400	700	3.5	2.03	2.03	78	685	707	1.03	1.08
BS-W (+)	400	700	3.5	2.03	2.03	77	685	707	1.03	1.10
BS-W (-)	400	700	3.5	2.03	2.03	77	685	707	1.03	1.13
BF-W (+)	400	700	3.4	1.23	1.23	85	685	716	1.05	1.16
BF-W (-)	400	700	3.4	1.23	1.23	85	685	716	1.05	1.18
BS-FT (+)	400	700	3.5	2.03	2.03	84	685	689	1.01	1.09
BS-FT (-)	400	700	3.5	2.03	2.03	84	685	689	1.01	1.09
BS-FTN (+)	400	700	3.5	2.03	2.03	75	685	689	1.01	1.12
BS-FTN (-)	400	700	3.5	2.03	2.03	75	685	689	1.01	1.10
BF-T (+)	400	700	3.3	0.95	0.95	87	685	707	1.03	1.15
BF-T (-)	400	700	3.3	0.95	0.95	87	685	707	1.03	1.15
									mean	1.14
									Upper bound	1.16
									Lower bound	1.12

目前結果: SD685 鋼筋 $\alpha_0 = 1.15$

結論

1. New RC梁整體耐震行為與Normal RC梁相似，故ACI剪力設計法可保守使用於New RC梁剪力設計。

2. New RC梁剪力衰減行為，比 Normal RC梁緩和。

3. New RC梁拉力筋SD685 之超額強度係數α_o (steel overstrength factor)， 初步調查結果為 α_o=1.15。

4. New RC梁混凝土抗剪強度設計建議

$V_n = V_c + V_s$

$V_c = 0.17\sqrt{f'_c}\, b_w d$ [非塑鉸區]

$V_c = 0$ [塑鉸區，離柱面2h範圍]

5. New RC梁SD785剪力筋設計建議

$V_s = \frac{A_v f_{yt} d}{s_v} \leq 0.66\sqrt{f'_c}\, b_w d$

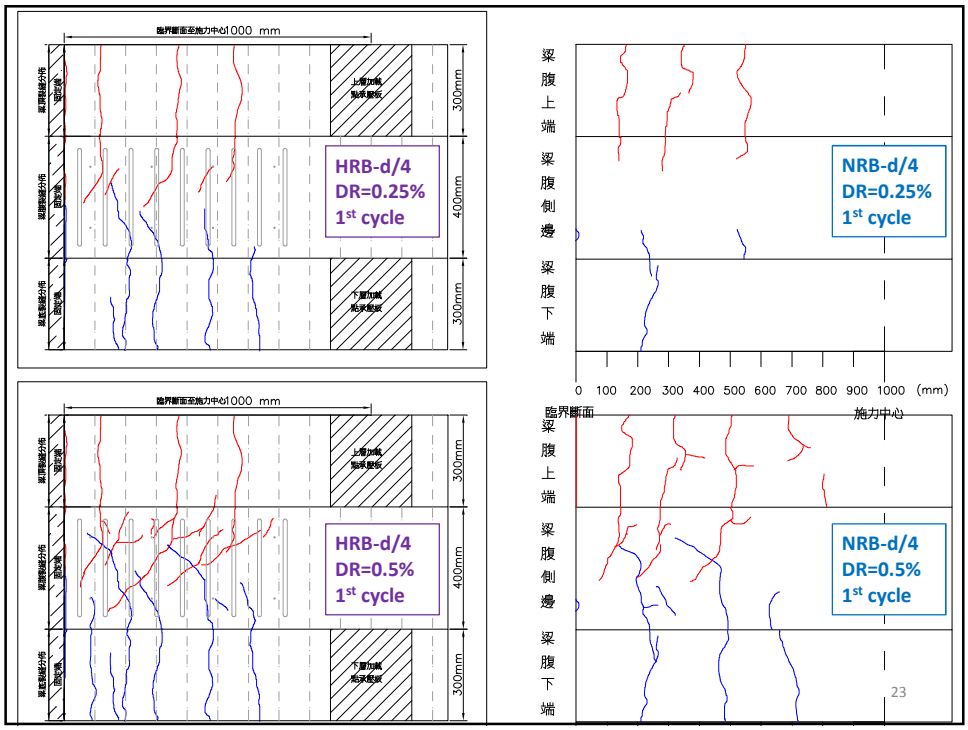
$\theta = 45°$

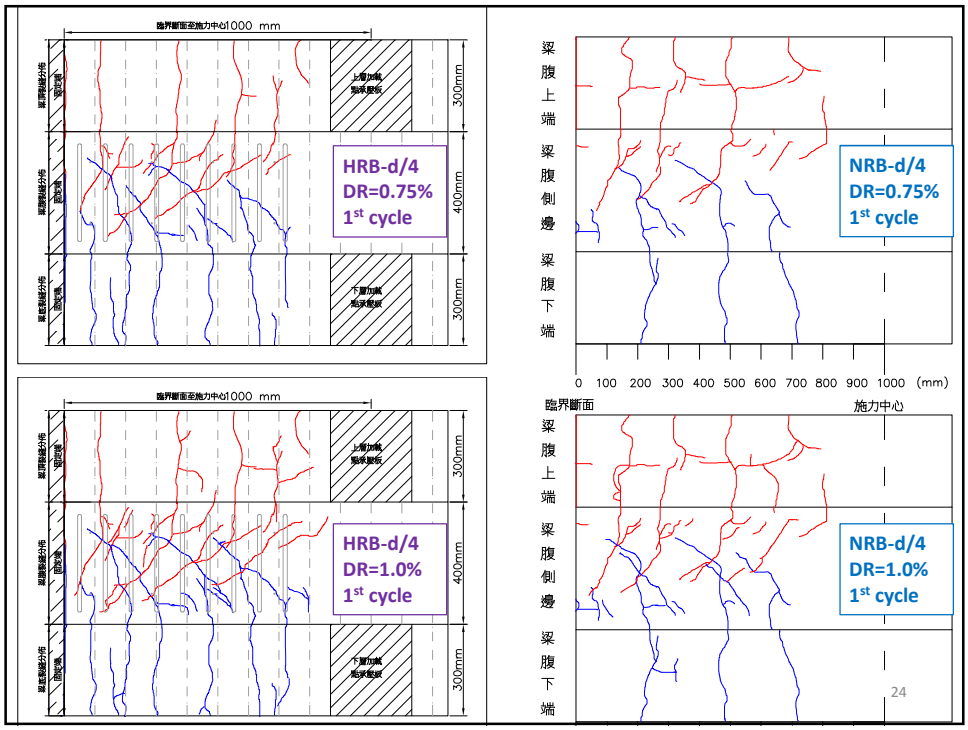
$f_{yt} \leq 600MPa$

間距規定: $s = \min(\frac{d}{2}, 300mm, s_v)$ [非塑鉸區]

$s = min(\frac{d}{4}, 150mm, 6d_{bl}, s_v)$ [塑鉸區]

報告完畢，謝謝聆聽


臨界斷面至施力中心1000 mm
HRB-d/4
DR=0.25%
1st cycle
NRB-d/4
DR=0.25%
1st cycle
HRB-d/4
DR=0.5%
1st cycle
NRB-d/4
DR=0.5%
1st cycle
300mm
400mm
300mm
梁腹上端
梁腹側邊
梁腹下端
0 100 200 300 400 500 600 700 800 900 1000 (mm)
臨界斷面
施力中心


臨界斷面至施力中心1000 mm
HRB-d/4
DR=0.75%
1st cycle
NRB-d/4
DR=0.75%
1st cycle
HRB-d/4
DR=1.0%
1st cycle
NRB-d/4
DR=1.0%
1st cycle
300mm
400mm
300mm
梁腹上端
梁腹側邊
梁腹下端
0 100 200 300 400 500 600 700 800 900 1000 (mm)
臨界斷面
施力中心

高強度鋼筋混凝土梁之裂縫發展與控制

邱建國 陳少謙 林芳慶

2014/12/05

國立台灣科技大學營建工程系
國家地震工程研究中心

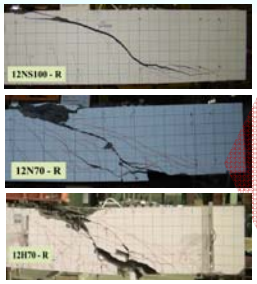
TAIWAN TECH
National Taiwan University of Science and Technology

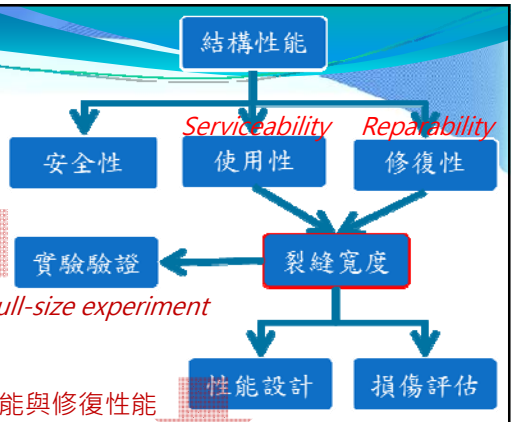
臺灣科大

簡報大綱

1. 研究目的
2. 實驗規劃
3. 實驗結果與分析
4. 設計公式建議
5. 結論

研究目的(1/3)





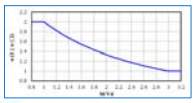
Full-size experiment

性能設計 – 使用性能與修復性能

AIJ, 2010

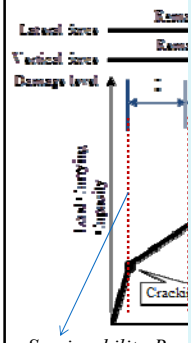
	剪力計算公式	容許狀況
使用性確保剪力強度(柱)	$bj\alpha f_s$	剪力裂縫發生點
使用性確保剪力強度(梁)	長期荷重; 靜載+活重 $bj\{\alpha f_s+0.5_w f_t(p_w-0.002)\}$	容許剪力裂縫寬度 0.3 mm
短期容許剪力強度	中小規模地震; 修復性能 $bj\left\{\frac{2}{3}\alpha f_s+0.5_w f_t(p_w-0.002)\right\}$	容許剪力裂縫寬度 1.0 mm 殘餘裂縫寬度 0.3 mm
安全性確保剪力強度	大規模地震; 安全性能 $bj\{\alpha f_s+0.5_w f_t(p_w-0.002)\}$	極限剪力強度

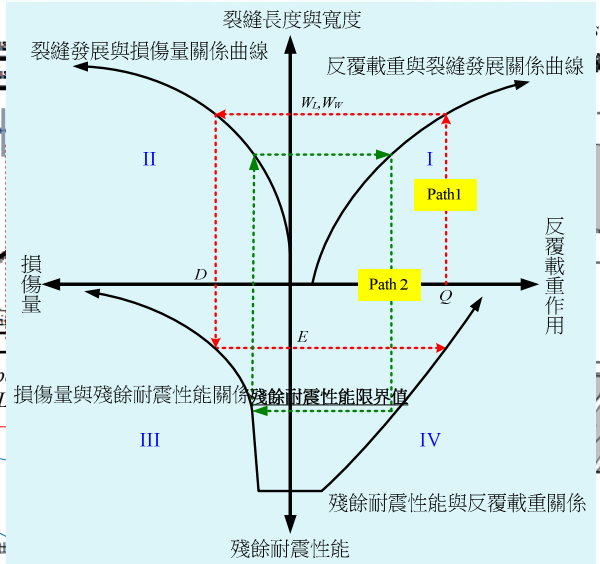
Allowable shear crack width

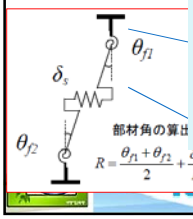


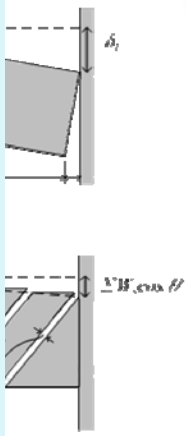
研究目的(2/3)

損傷評估(裂縫寬度與變形角)









構件非線性分析時之性能點決定

研究目的(3/3)

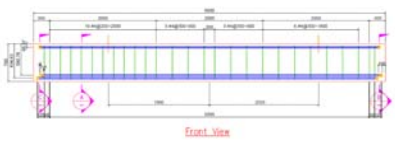
本試驗主要使用高強度鋼筋混凝土之足尺寸梁試體以了解剪力裂縫發展行為。各試體於單向載重作用下，分別量測並統計其於尖峰變形角時最大剪力裂縫寬度及殘留最大剪力裂縫寬度與試體斷面平均剪應力關係，再分別以0.3 mm (ACI: 0.4 mm) 及1.0 mm為上限並決定其對應之容許剪應力及使用性能確保之剪力設計公式。

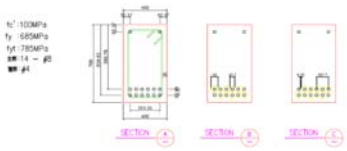
1. 平均斷面剪應力與尖峰變形角時最大剪力裂縫寬度
2. 平均斷面剪應力與殘留最大剪力裂縫寬度
3. 最大剪力裂縫寬度之殘留比(與剪力變形角關係)
4. **最大剪力裂縫寬度與容許應力(平均斷面剪應力)之關係**
5. **使用性能確保之剪力設計公式**

實驗規劃(1/6)

- 102~103年 共進行22組高強度鋼筋混凝土梁試驗

年度	試體名稱	破壞模式	拉力鋼筋根數	箍筋型式	箍筋間距		主筋降伏強度	箍筋降伏強度	混凝土抗壓強度	拉力鋼筋比 ρs	箍筋比ρw (%)		剪深比	保護層厚度	斷面尺寸	
	Specimen		N*	T*	Left	Right	fy (MPa)	fyt (MPa)	f'c (MPa)	(%)	Left	Right	a/d	c (cm)	寬(mm)	深(mm)
102年	8H70	撓剪	8	II	20	30	685	785	70	1.45	0.32	0.21	3.3	4	400	700
	8H100	撓剪	8	II	20	30	685	785	100	1.45	0.32	0.21	3.3	4	400	700
	8N70	剪力	8	II	20	30	685	420	70	1.45	0.32	0.21	3.3	4	400	700
	8N100	剪力	8	II	20	30	685	420	100	1.45	0.32	0.21	3.3	4	400	700
	8NS100	剪力	8	-	-	-	685	-	100	1.45	-	-	3.3	5.5	400	700
	12H70	剪力	12	II	20	30	685	785	70	2.17	0.32	0.21	3.3	4	400	700
	12H100	剪力	12	II	20	30	685	785	100	2.17	0.32	0.21	3.3	4	400	700
	12N70	剪力	12	II	20	30	685	420	70	2.17	0.32	0.21	3.3	4	400	700
	12N100	剪力	12	II	20	30	685	420	100	2.17	0.32	0.21	3.3	4	400	700
	12NS100	剪力	12	-	-	-	685	-	100	2.17	-	-	3.3	5.5	400	700





12H100

實驗規劃(2/6)

- 102~103年 共進行22組高強度鋼筋混凝土梁試驗

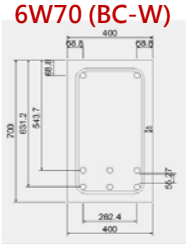
年度	試體名稱	破壞模式	拉力鋼筋根數	箍筋型式	箍筋間距		主筋降伏強度	箍筋降伏強度	混凝土抗壓強度	拉力鋼筋比 ρs	箍筋比ρw (%)		跨深比	保護層厚度	斷面尺寸	
	Specimen		N*	T*	Left	Right	fy (MPa)	fyt (MPa)	f'c (MPa)	(%)	Left	Right	a/d	c (cm)	寬(mm)	深(mm)
103年	6W70	撓剪	6	W	20	30	685	785	70	2	0.32	0.21	3.33	4	400	700
	6H70	撓剪	6	H	20	30	685	785	70	2	0.32	0.21	3.33	4	400	700
	175R70	剪力	6	H	30		685	785	70	3.5	0.24		1.75	4	350	500
	200R70	剪力	6	H	30		685	785	70	3.5	0.24		2	4	350	500
	275R70	剪力	6	H	30		685	785	70	3.5	0.24		2.75	4	350	500
	325R70	剪力	6	H	30		685	785	70	3.5	0.24		3.25	4	350	500
	175R100	剪力	6	H	30		685	785	100	3.5	0.24		1.75	4	350	500
	200R100	剪力	6	H	30		685	785	100	3.5	0.24		2	4	350	500
	275R100	剪力	6	H	30		685	785	100	3.5	0.24		2.75	4	350	500
	325R100	剪力	6	H	30		685	785	100	3.5	0.24		3.25	4	350	500
	2C100	剪力	12	H	20	30	685	785	100	2.17	0.32	0.21	3.3	2	400	700
	3C100	剪力	14	H	20	30	685	785	100	2.17	0.32	0.21	3.3	3	400	700

6m

4m

2m

6W70 (BC-W)



實驗規劃(3/6)

試體規劃與參數設定

	參數名稱		No.	類型	試體數量
103年	跨深比	a/d	1	1.75	2
			2	2	2
			3	2.75	2
			4	3.25	2
			5	3.33	14
	保護層厚度	c	1	2 cm	1
			2	3 cm	1
			3	4 cm	18
102年	拉力鋼筋比	ρs	1	1.45%	5
			2	2%	5
			3	2.17%	2
			4	3.50%	8
102年	箍筋比	ρw	1	0.21%	12
			2	0.24%	8
			3	0.32%	12
TTK	箍筋形式	T*	1	焊接箍筋	1
			2	傳統箍筋	19
			3	分段用鋼筋	0
	斷面尺寸	b · d	1	350mm · 500mm	8
			2	500mm · 700mm	14
	混凝土強度	f'c	1	70MPa (設計值)	10
			2	100MPa (設計值)	12

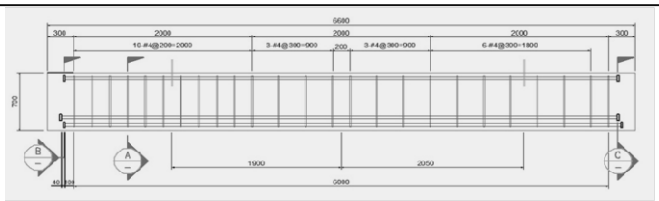
104年

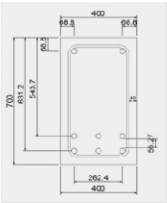
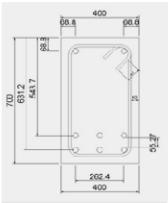
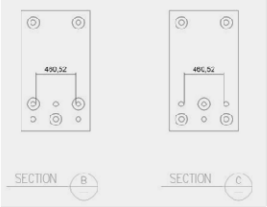
104年

實驗規劃(4/6)

試體設計與配筋細節

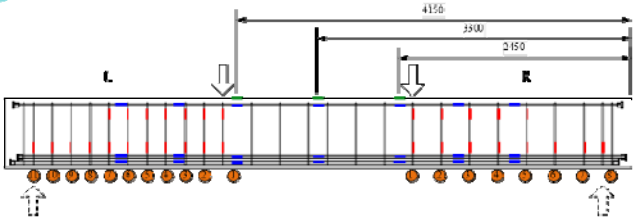
Specimen Design

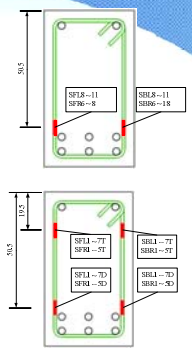


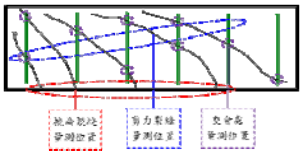
BC-W **6W70**		BC-T **6H70**		

實驗規劃(5/6)

應變設置與裂縫量測



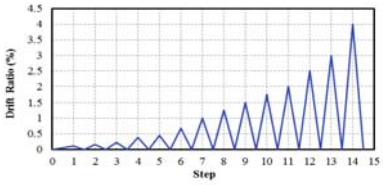


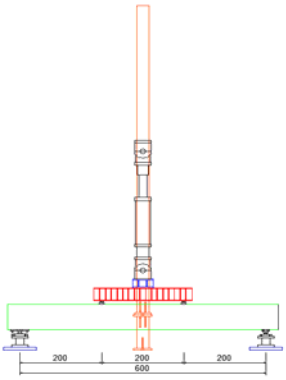


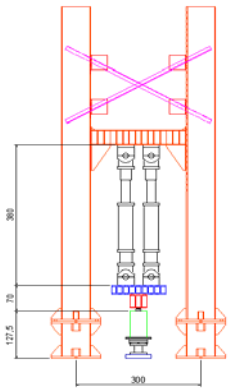
實驗規劃(6/6)

測試構架與載重歷時

Loading Procedure



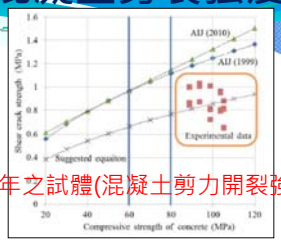




實驗結果與分析(1/8)

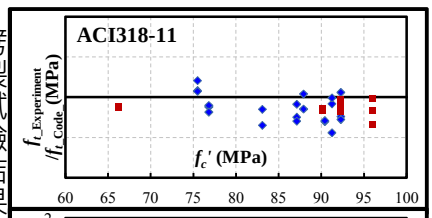
混凝土劈裂強度

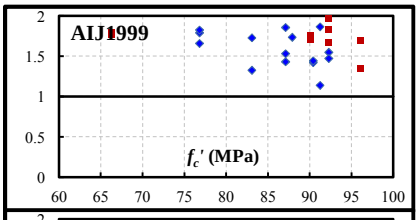
ACI	$0.67\sqrt{f_c'}$	MPa
AIJ1999	$0.33\sqrt{f_c'}$	MPa
AIJ2010	$0.56\sqrt{f_c'}$	MPa
JSCE2007	$0.23f_c'^{2/3}$	MPa

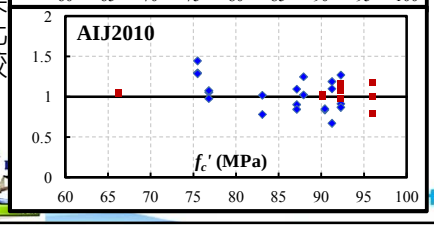


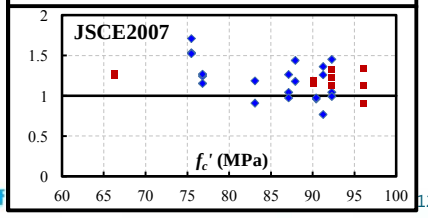
102 年之試體(混凝土剪力開裂強度)

劈裂試驗結果比較









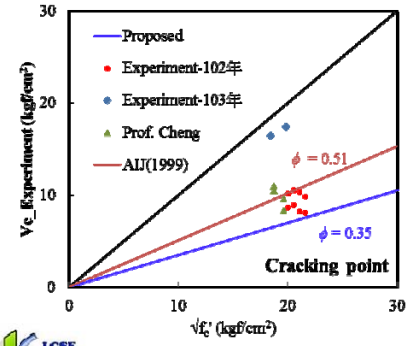
實驗結果與分析(2/8)

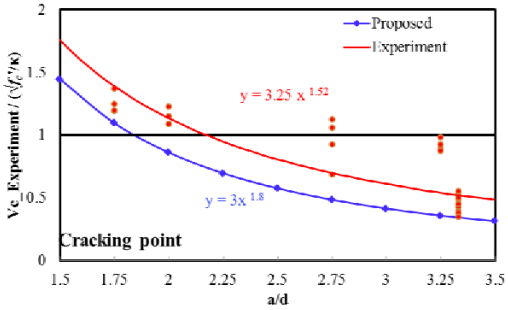
混凝土剪力開裂強度

$$v_{cr} = \eta_{cr} \times 0.33\sqrt{f'_c}/\kappa \text{ (MPa)}$$

$$\eta_{cr} = 3\left(\frac{a}{d}\right)^{-1.8} \quad (0.35 \le \eta_{cr} \le 1.0)$$

- 剪力開裂強度定義：觀測到第一條剪力裂縫時所對應之斷面平均剪應力
- 用以定義長期荷重下之混凝土容許剪應力





實驗結果與分析(3/8)

混凝土剪力極限強度

混凝土剪力極限強度

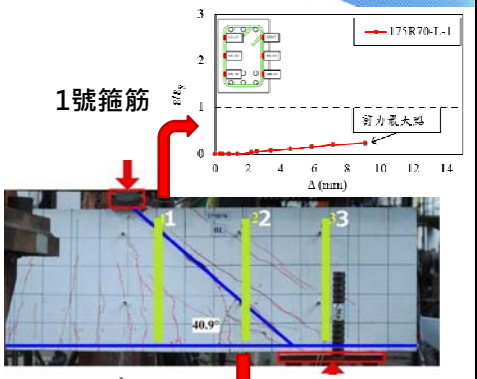
- 試體之極限剪力扣除鋼筋貢獻之剪力
- 用以定義短期荷重下之混凝土容許剪應力
- **箍筋貢獻之剪力強度計算**

如右圖所示，裂縫通過1號及2號箍筋
1號箍筋未降伏→應變為ε_{s1}
2號箍筋已降伏→應變為ε_y

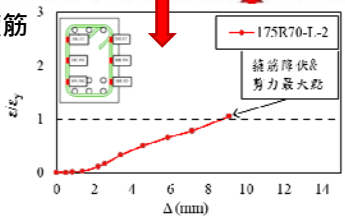
箍筋貢獻強度加總

$$V_{s1} = A_v E_s \varepsilon_{s1}$$
$$V_{s2} = A_v E_s \varepsilon_y$$
$$V_s = V_{s1} + V_{s2}$$

1號箍筋



2號箍筋



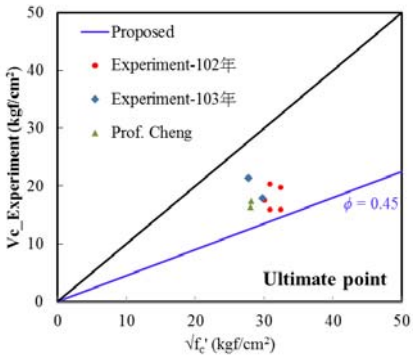
實驗結果與分析(4/8)

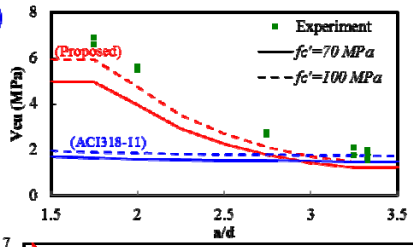
混凝土剪力極限強度

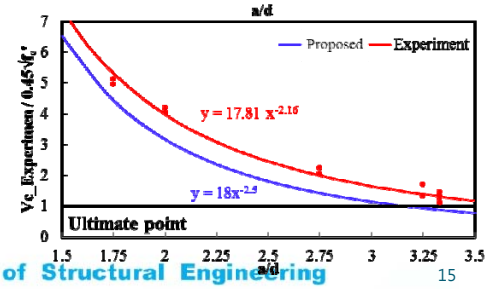
$$v_{cu} = 0.45\eta_{cu} \times 0.33\sqrt{f'_c}\ (\text{MPa})$$

$$\eta_{cu} = 18\left(\frac{a}{d}\right)^{-2.5};\ (1 \le \eta_{cu} \le 4)\ (\text{MPa})$$

ACI318-11 $V_c = \left(0.53\sqrt{f'_c} + 175\rho_w \frac{V_u d}{M_u}\right) bd$







實驗結果與分析(5/8)

使用性確保剪力裂縫控制

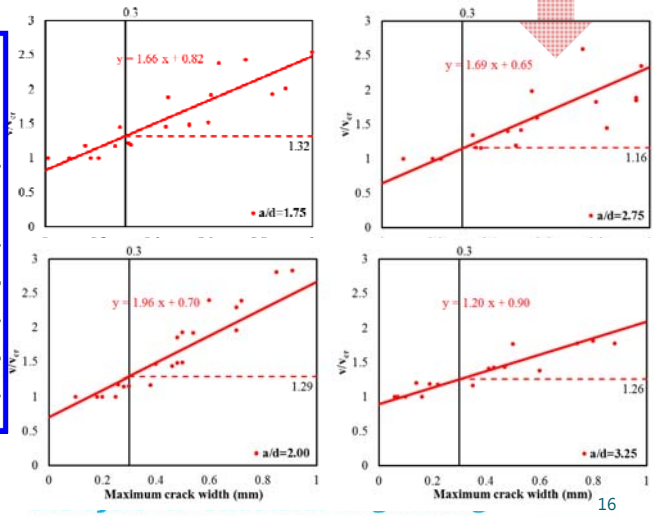
剪力裂縫寬度與斷面平均剪應力之關係

跨深比

剪力裂縫最大寬度 0.3 mm

斷面平均剪應力比

a/d	shear stress ratio (v/v_{cr})
1.75	1.32
2	1.29
2.75	1.16
3.25	1.26



實驗結果與分析(6/8)

使用性確保剪力裂縫控制

剪力裂縫寬度與箍筋應變之關係

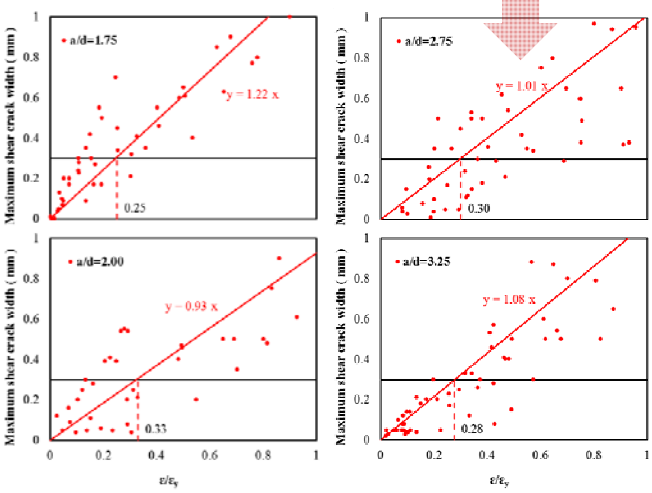
跨深比

長期荷重下使用性能確保之鋼筋容許應力：

$$0.3 f_{s\_allow} = 0.25 f_y$$

鋼筋容許應變比

a/d	$\varepsilon/\varepsilon_y$
1.75	0.25
2	0.33
2.75	0.3
3.25	0.28



實驗結果與分析(7/8)

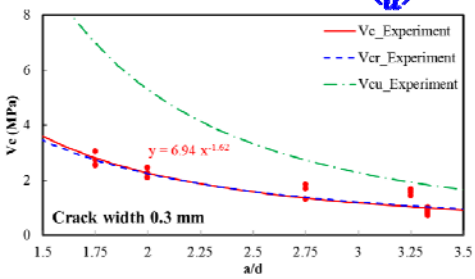
使用性確保剪力裂縫控制

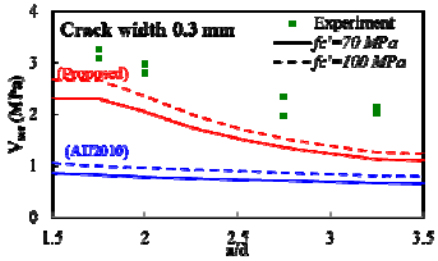
跨深比與混凝土容許剪應力

$$v_{c\_ser} = v_{cr}$$

$$v_{cr} = \eta_{cr} \times 0.33\sqrt{f'_c}/\kappa \text{ (MPa)}$$

$$\eta_{cr} = 3\left(\frac{a}{d}\right)^{-1.8} \quad (0.35 \le \eta_{cr} \le 1.0)$$





實驗結果與分析(8/8)

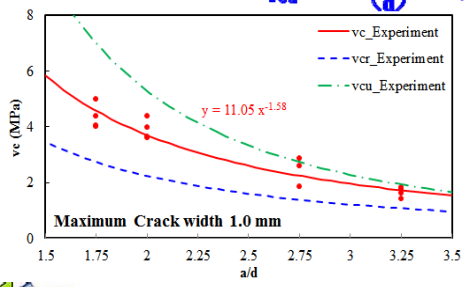
修復性確保剪力裂縫控制

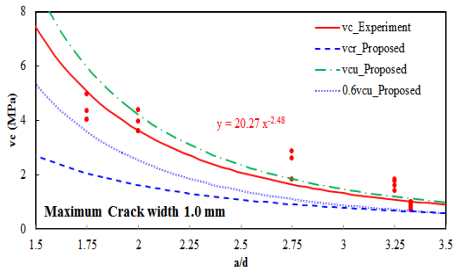
跨深比與混凝土容許剪應力

$$v_{c\_rep} = 0.6v_{cu}$$

$$v_{cu} = 0.45\eta_{cu} \times 0.33\sqrt{f'_c}\ (\text{MPa})$$

$$\eta_{cu} = 18\left(\frac{a}{d}\right)^{-2.5};\ (1 \le \eta_{cu} \le 4)\ (\text{MPa})$$





設計公式建議(1/9)

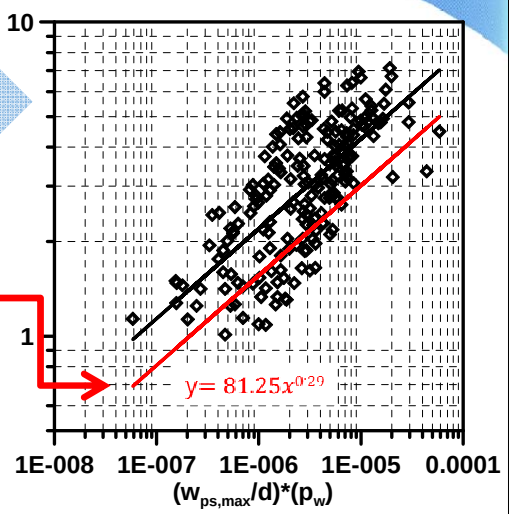
使用性確保剪力裂縫控制

斷面平均剪應力與剪力裂縫最大寬度(箍筋785MPa)

$$V = 81.25 \times \left\{\left(\frac{W_{ps,max}}{d}\right) \times \rho_{ts}\right\}^{0.29} \times \left(\eta_{cr} \times \left(0.33\sqrt{f'_c}\right) \times bD/1.5\right)$$

$$\eta_{cr} = 3 \times \left(a/d\right)^{-1.8};\ 0.35 \le \emptyset \le 1.0$$

$$\rho_{ts} = \frac{A_v}{bd}$$



設計公式建議(2/9)

使用性確保剪力裂縫控制

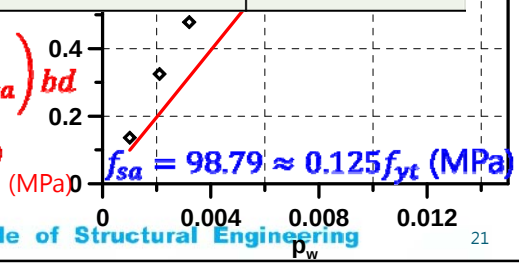
(剪

$V_c =$ (V

	剪力計算公式	對應狀態
使用性確保剪力強度(材)	$bj\alpha f_s$	剪力裂縫發生點
使用性確保剪力強度(梁)	$bj\{\alpha f_s + 0.5\,{}_w f_t(p_w - 0.002)\}$ 長期荷重: 靜載+活重 195MPa	容許剪力裂縫寬度 0.3 mm
短期容許剪力強度	中小規模地震; 修復性能 $bj\left\{\frac{2}{3}\alpha f_s + 0.5\,{}_w f_s(p_w - 0.002)\right\}$	容許剪力裂縫寬度 1.0 mm 殘餘裂縫寬度 0.3 mm
安全性確保剪力強度	大規模地震; 安全性能 $bj\{\alpha f_s + 0.5\,{}_w f_t(p_w - 0.002)\}$	極限剪力強度

$$V_{sa} = \left(\frac{\eta_{cr} \times 0.33\sqrt{f_c'}}{1.5} + \rho_{ts} f_{sa}\right) bd$$

$$\eta_{cr} = 3 \times \left({}^{a}/_{d}\right)^{-1.0}; 0.35 \le \emptyset \le 1.0$$

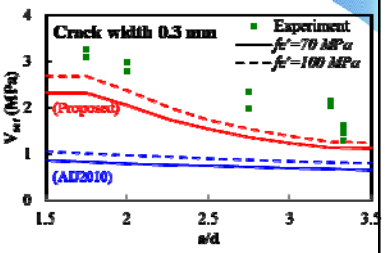


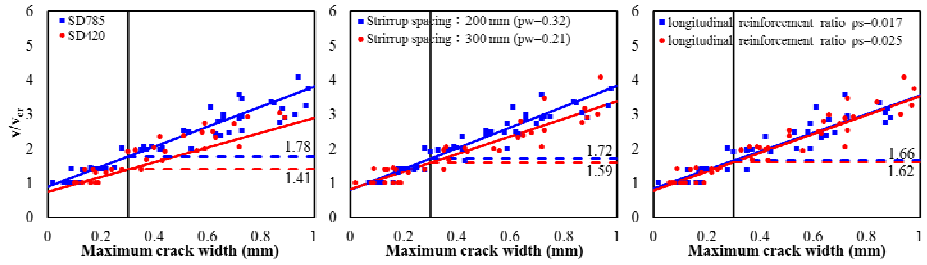
$f_{sa} = 98.79 \approx 0.125 f_{yt}$ (MPa)

設計公式建議(3/9)

梁試驗結果驗證(I)-跨深比3.33 (單向載重)

shear stress ratio (v/v_{cr})	Stirrup strength		Stirrup spacing		Longitudinal reinforcement ratio	
Maximum crack width (mm)	SD785	SD420	200 mm	300 mm	p_s=0.025	p_s=0.017
0.3	1.78	1.41	1.72	1.59	1.62	1.66





設計公式建議(4/9)

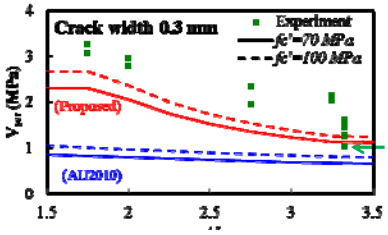
梁試驗結果驗證(II)- 反覆載重

斷面尺寸：400 mm × 700 mm

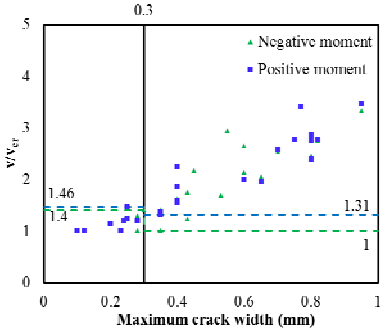
跨深比：3.33

箍筋間距：120 mm (ρ_{ts}=0.53%)

主筋比：ρ_s=0.02



Allowable shear stress ratio for the peak maximum crack width (v/v_{cr})		
shear stress ratio (v/v_{cr})	Moment type	
Maximum crack width (mm)	Negative	Positive
0.3	1	1.31



設計公式建議(5/9)

使用性確保剪力裂縫控制

使用性確保與耐震需求之箍筋比

$$\rho_{ts,a} = \left(\frac{w_a}{2} \times \left(\frac{B}{b}\right)\left(\frac{L}{d}\right)\left(1-\frac{B}{2L}\right) - \frac{V_{sc}}{bd}\right)/f_{sa}$$

$$\rightarrow \rho_{ts,a} = \left(\frac{w_a}{2} \times \left(\frac{L/D}{b/D}\right)\left(\frac{B}{L}\right)\left(\frac{L}{d}\right)\left(1-\frac{B}{2L}\right) - \frac{V_{sc}}{bd}\right)/f_{sa}$$

$$V_u = \frac{2M_{pr}}{L} + \frac{w_u LB}{2} = \frac{2 \times 0.9 \times 1.25 f_y \times \rho_t \times bd^2}{L} + \frac{w_u LB}{2}\left(1-\frac{B}{2L}\right)$$

$$v_u = \frac{2 \times 0.9\rho_t \times 1.25 f_y\, d}{L} + \frac{w_u LB}{2bd}\left(1-\frac{B}{2L}\right)$$

$$\rightarrow \rho_{ts,u} = \left(\frac{2 \times 0.9\rho_t \times 1.25 f_y\, d}{L} + \frac{w_u LB}{2bd}\left(1-\frac{B}{2L}\right)\right)/f_{su}$$

$$\rho_{ts,u} = \left(C_1/\left(\frac{L}{D}\right) + C_2\left(\frac{L}{D}\right)^2\right)/f_{su}$$

$$\rho_{ts,a} = \left(C_3\left(\frac{L}{D}\right)^2 - v_{cr}\right)/f_{sa}$$

設計公式建議(6/9)

使用性確保之剪力裂縫控制 – L/D

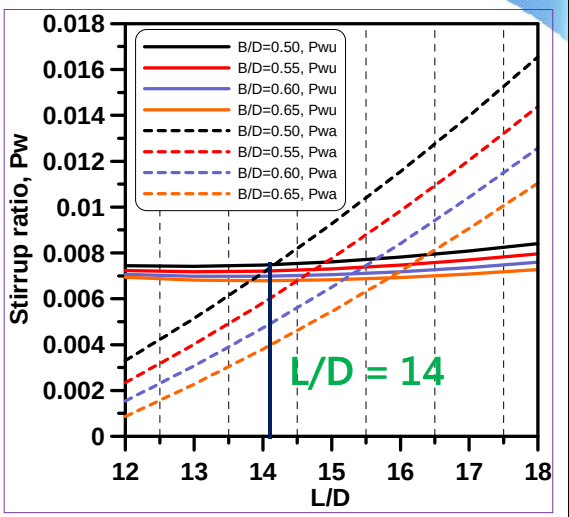
Assume:

ρ_s=0.025; d=0.9D;
f_y=685 MPa;
w_a=1000+300 kgf/m^2;
w_u=1000*1.2 + 300 kgf/m^2

f_{sa}=0.125f_{yt} (785 MPa);
f_{su}=550 MPa;
B/L=1.0.

	B/L (fc'=70 MPa)		
b/D	1	0.8	0.6
0.5	14	14	15
0.55	14	15	15
0.6	15	15	16
0.65	15	16	17

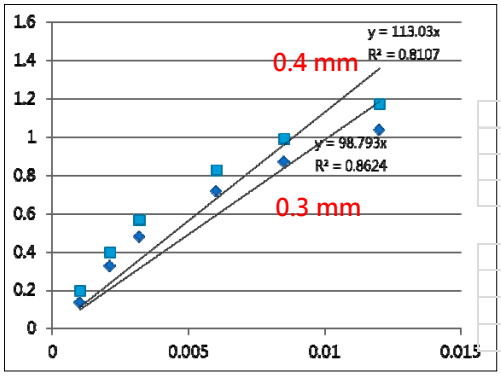
	B/L (fc' = 100 MPa)		
b/D	1	0.8	0.6
0.5	14	15	16
0.55	15	15	16
0.6	16	16	17
0.65	16	16	17



設計公式建議(7/9)

使用性確保之剪力裂縫控制 – L/D

剪力裂縫最大寬度值 = 0.3 及 0.4 mm



	Pt			
fc'	0.01	0.015	0.02	0.025
70	12	13	13	14
100	13	13	14	14

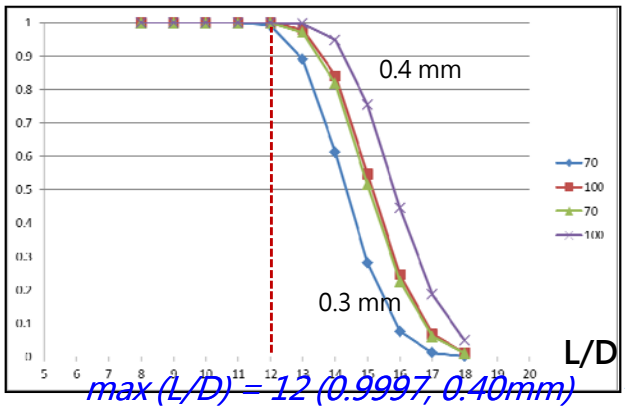
	Pt			
fc'	0.01	0.015	0.02	0.025
70	12	12	13	14
100	13	14	14	15

$$f_{sa}\ (0.3\ mm) = 0.125 f_{yt}\ \text{(MPa)}$$
$$f_{sa}\ (0.4\ mm) = 0.150 f_{yt}\ \text{(MPa)}$$

設計公式建議(8/9)

可靠度分析

使用性確保之剪力裂縫控制 – L/D



Random variables
b/D: 0.45-0.55
B/L: 0.5-1.0
Dead Load : 800-1100 kgf/m^2
Live Load: 200-400 kgf/m^2

f_y: 685 Mpa
f_{yt}: 785 Mpa
f_{su}: 550 MPa
f_{sa}: 0.125or 0.15 f_{yt}

max (L/D) = 12 (0.9997, 0.40mm)

MCS

$$\rho_{ts,u} = \left(C_1/\left(\frac{L}{D}\right) + C_2\left(\frac{L}{D}\right)^2\right)/f_{su}$$

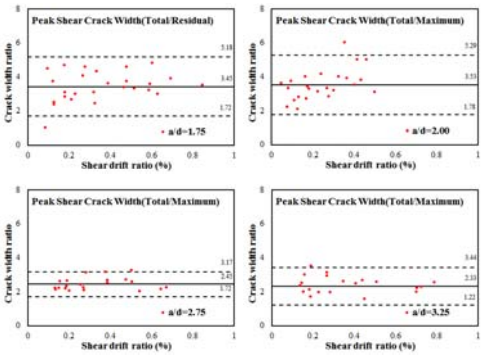
$$\rho_{ts,a} = \left(C_3\left(\frac{L}{D}\right)^2 - v_{cr}\right)/f_{sa}$$

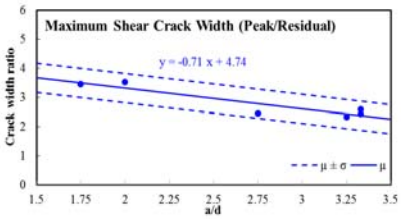
$$G(L/D) = \frac{\rho_{ts,u}}{\rho_{ts,a}} - 1$$

Reliability

設計公式建議(9/9)

尖峰變形角時剪力裂縫最大寬度與殘留剪力裂縫最大寬度之比值分佈





$$n_{s\_Maximum} = -0.71 \times \left(\frac{a}{d}\right) + 4.74$$

本研究考量構件於地震後之殘留剪力裂縫最大值以不超0.4 mm為原則，以建立修復性能確保下之箍筋容許應力值。若取尖峰變形角時剪力裂縫最大寬度與殘留剪力裂縫最大寬度之比值2.5計算尖峰變形角時剪力裂縫最大寬度，則約為1.0 mm。若以剪力裂縫最大寬度1.0 mm為控制目標，則其對應之容許箍筋應力可定為162.9 MPa (約為0.20倍之箍筋規定降伏應力)

結論

梁構件性能點			容許剪應力計算式 (MPa)
長期荷重	使用性	剪力裂縫發生點	$v_{cr} = \eta_{cr} \times 0.33\sqrt{f_c'}/1.5$ $\eta_{cr} = 3\left(\frac{a}{d}\right)^{-1.8} \quad (0.35 \le \eta_{cr} \le 1.0)$
		剪力裂縫最大寬度 0.4mm	$v_{ser} = \eta_{cr} \times 0.33\sqrt{f_c'}/1.5 + 0.15\rho_{ts}f_{yt}$ *Or max (L/D) = 12*
短期荷重	修復性	剪力裂縫最大寬度 1.0 mm	$v_{rep} = 0.27\eta_{cu} \times 0.33\sqrt{f_c'} + 0.20\rho_{ts}f_{yt}$ $\eta_{cu} = 18\left(\frac{a}{d}\right)^{-2.5}; \ (1 \le \eta_{cu} \le 4)$

謝謝....

高強度鋼筋混凝土梁之
裂縫發展與控制

ACI 及AIJ撓曲裂縫寬度限制值

規範		限制值(mm)
ACI	Z-Factor	0.33
ACI	Spacing Control	0.41~0.53
AIJ	曲げひび割れ幅算定式	0.30

ACI – Z Factor

$$w_{max} = 11 \cdot 10^{-6} \beta f_s \sqrt[3]{d_c A}$$

ACI – Spacing Control

$$w_c = \frac{f_s}{E_s} \cdot 2\sqrt{d_c^2 + (\frac{s}{2})^2}$$

AIJ – 曲げひび割れ幅算定式

$$W_{max} = 1.5 W_{av}$$

$$W_{av} = l_{av}\varepsilon_{s,av} + l_{av}\varepsilon_{sh}$$

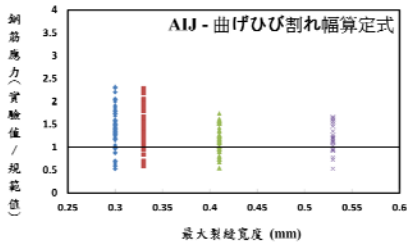
$$l_{av} = 2\times(c + \frac{s}{10}) + k\frac{\phi}{p_e}$$

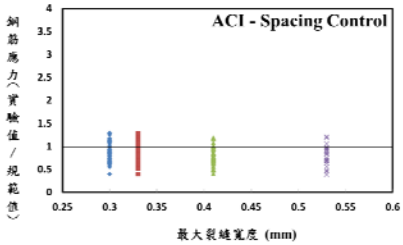
$$\varepsilon_{s,av} = \frac{1}{E_s}(\sigma_t - k_1 k_2 \times \frac{F_t}{p_e})$$

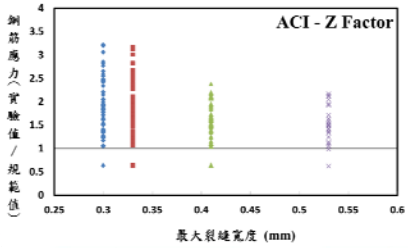
實驗結果比較

觀察範圍：四點荷重簡支梁試體等彎矩段內裂縫寬度

裂縫寬度種類：0.3/0.33/0.41/0.53 mm







鋼筋應力 f_s

ACI Spacing Control

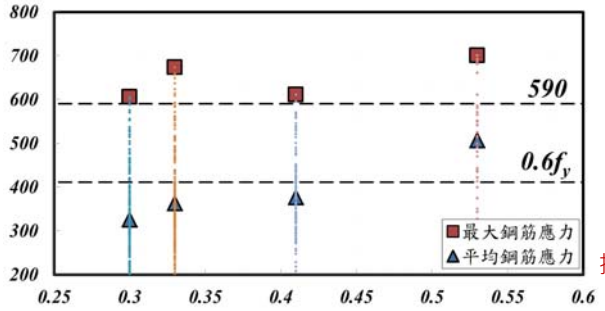
$$w_c = \frac{f_s}{E_s} \cdot 2\sqrt{d_c^2 + (\frac{s}{2})^2}$$

建議 f_s 取 $0.6f_y$

$f_y = 685\ MPa$

$f_s = 0.6f_y = 411\ \text{MPa}$

鋼筋應力 (MPa)



撓曲裂縫寬度 (mm)

AIJ 2010 容許應力檢核

表 7 混凝土容許應力 (MPa)

長期容許應力			短期容許應力		
抗壓	抗拉	抗剪	抗壓	抗拉	抗剪
$\frac{1}{3}f_c'$	-	$\min\left\{\frac{1}{30}f_c', \left(0.49+\frac{1}{100}f_c'\right)\right\}$	長期容許值之 2.0 倍	-	長期容許值之 1.5 倍

*f_c' 為混凝土抗壓強度 (MPa)

剪力開裂應力

$M_{ser} \leq M_a$

靜載+活載

表 8 鋼筋容許應力 (MPa)

鋼筋型號	長期容許應力		短期容許應力	
	抗壓及抗拉	抗剪	抗壓及抗拉	抗剪
SR 235	155	155	235	235
SR 295	155	195	295	295
SD 295	195	195	295	295
SD 345	215 (*195)	195	345	345
SD 390	215 (*195)	195	390	390
SD 490	215 (*195)	195	490	490
SD 785	580	580	785	785

*D29 以上之鋼筋為( )內之數值