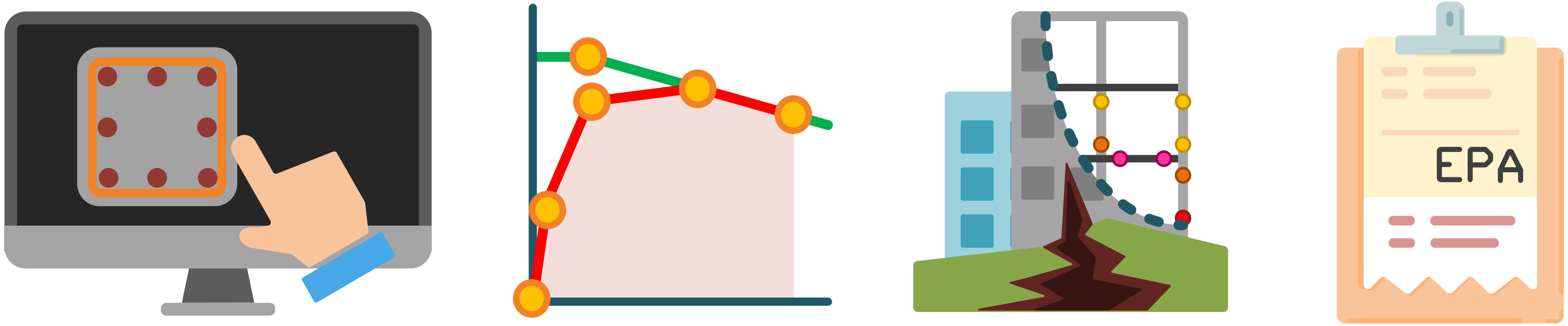

建築物耐震能力詳細評估SERCB之介紹與評估結果判讀

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國立臺北科技大學土木工程系 教授

報告內容

- 鋼筋混凝土建築結構非線性行為
- 鋼筋混凝土構件補強理論與分析驗證
- 鋼斜撐框架補強理論探討與分析驗證
- 鋼筋混凝土建築結構耐震能力評估

鋼筋混凝土建築結構非線性行為

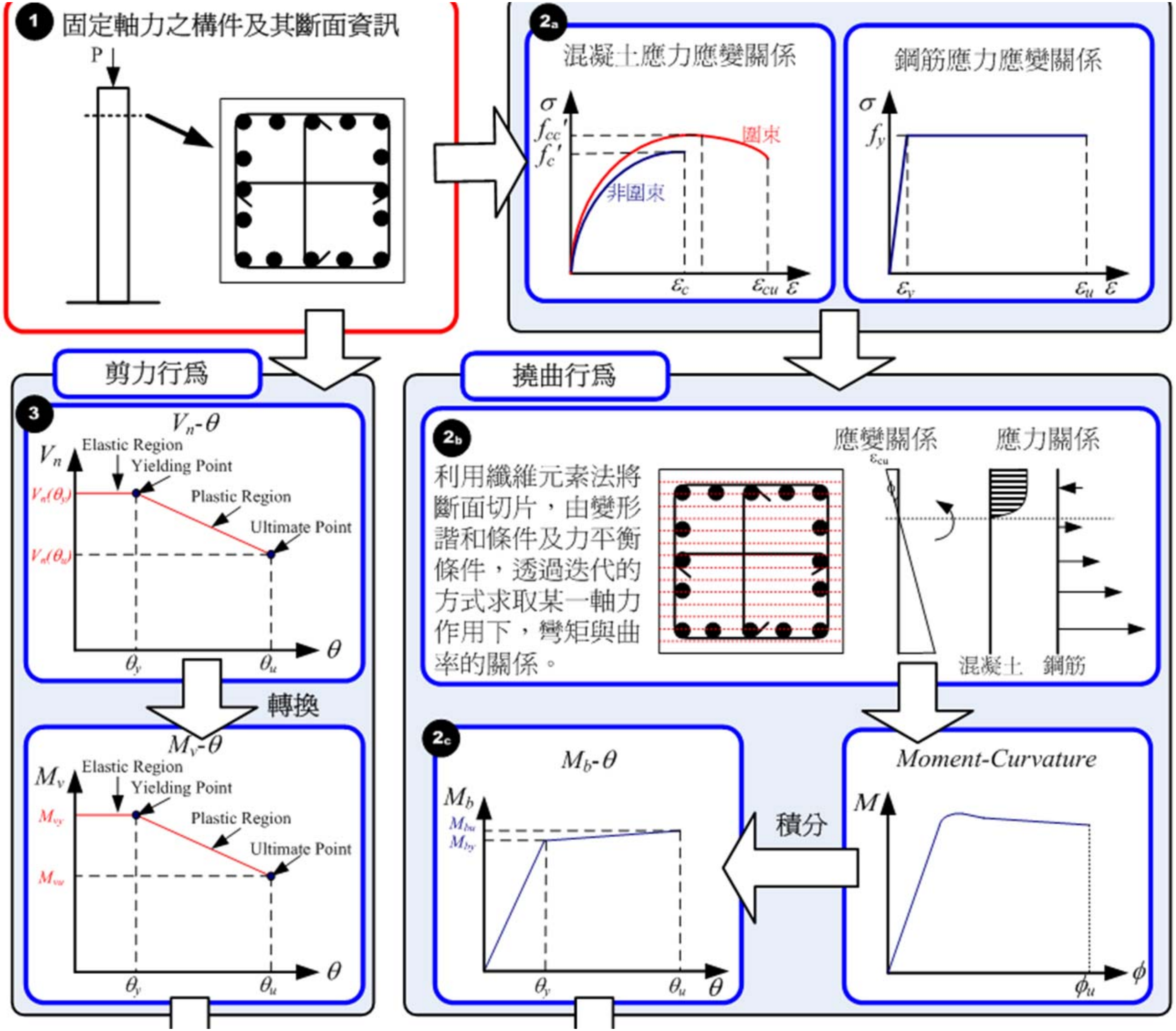
靜態側推分析
(Static Pushover Analysis)

動態歷時分析
(Dynamic Time-history Analysis)

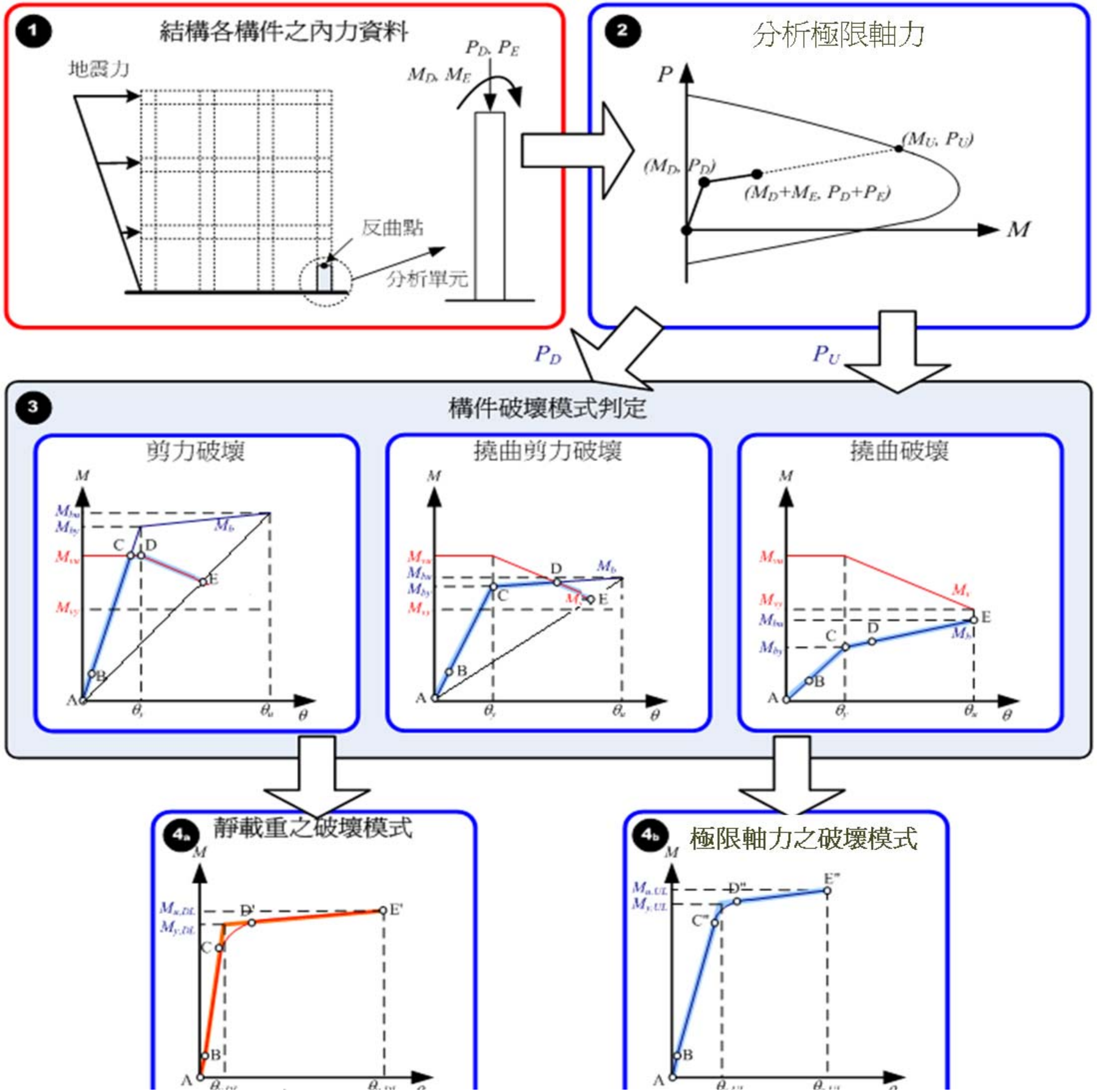
靜態側推分析
(Pushover Analysis)

❶ 藉由施加側向力於結構物上，隨著力量緩慢增加，記錄構材開裂、降伏、塑性變形和結構失敗等發展行為，並在一連串的迭代過程中，依各不同受力階段之結構行為，修正構材有效勁度與不平衡力，採用階段線性分析的方式，直到塑性較發展至崩塌機制或到達極限塑性變形為止。

❷ 能清楚地提供結構物在各性能階段的受力與變形行為。

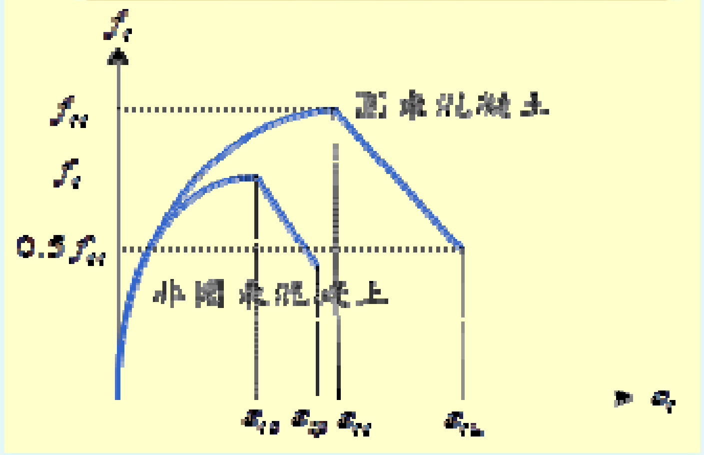


利用纖維元素法將斷面切片，由變形諧和條件及力平衡條件，透過迭代的方式求取某一軸力作用下，彎矩與曲率的關係。


1
結構各構件之內力資料
地震力
P_D, P_E
M_D, M_E
反曲點
分析單元
2
分析極限軸力
P
M
(M_U, P_U)
(M_D, P_D)
(M_D+M_E, P_D+P_E)
P_D
P_U
3
構件破壞模式判定
剪力破壞
撓曲剪力破壞
撓曲破壞
M
M_{bu}
M_{by}
M_{vu}
M_{vy}
M_b
M_v
A
B
C
D
E
θ_y
θ_u
θ
4a
靜載重之破壞模式
$M_{u,DL}$
$M_{y,DL}$
B'
C'
D'
E'
$\theta_{y,DL}$
$\theta_{u,DL}$
4b
極限軸力之破壞模式
$M_{u,UL}$
$M_{y,UL}$
B''
C''
D''
E''
$\theta_{y,UL}$
$\theta_{u,UL}$

混凝土組成律

Kawashima組成律



圍束區

$$f_c = E_c\varepsilon_c\left[1-\frac{1}{n}\left(\frac{\varepsilon_c}{\varepsilon_{cc}}\right)^{n-1}\right], 0\le\varepsilon_c<\varepsilon_{cc}$$

$$f_c = f'_{cc}-E_{des}\left(\varepsilon_c-\varepsilon_{cc}\right), \varepsilon_{cc}\le\varepsilon_c<\varepsilon_{cu}$$

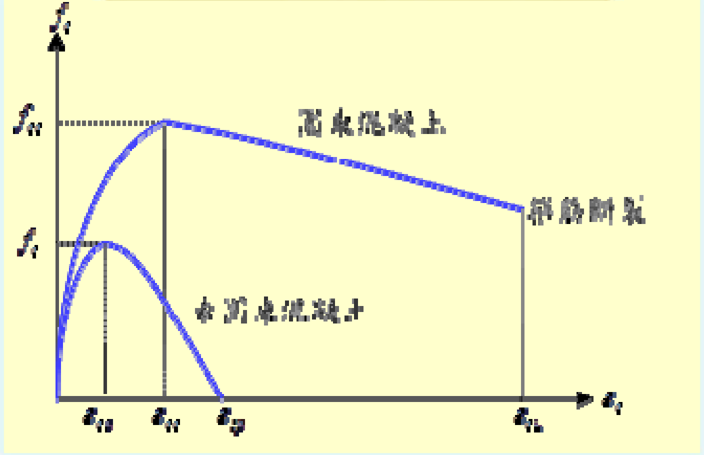
$$f_c = 0.5f'_{cc}, \varepsilon_{cu}\le\varepsilon_c$$

非圍束區

$$f_c = E_c\varepsilon_c\left[1-\frac{1}{n}\left(\frac{\varepsilon_c}{\varepsilon_{co}}\right)^{n-1}\right], 0\le\varepsilon_c<\varepsilon_{co}$$

$$f_c = f'_c-E_{des}\left(\varepsilon_c-\varepsilon_{co}\right), \varepsilon_{co}\le\varepsilon_c<\varepsilon_{sp}$$

Mander組成律



圍束區

$$f_c = \frac{f'_{cc}xr}{r-1+x^r}$$

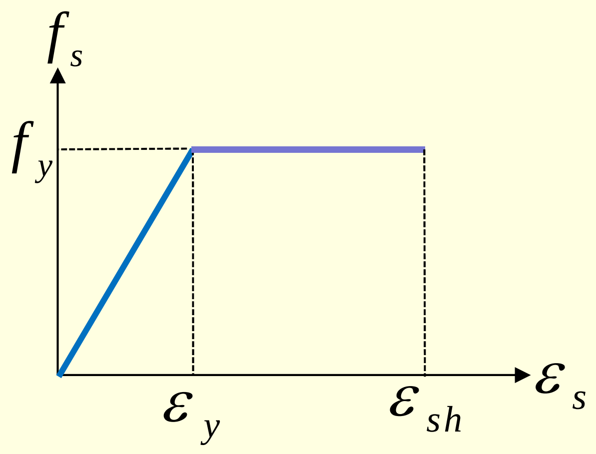
$$f'_{cc} = f'_c\left(-1.254+2.254\sqrt{1+\frac{7.94f'_l}{f'_c}}-2\frac{f'_l}{f'_c}\right)$$

非圍束區

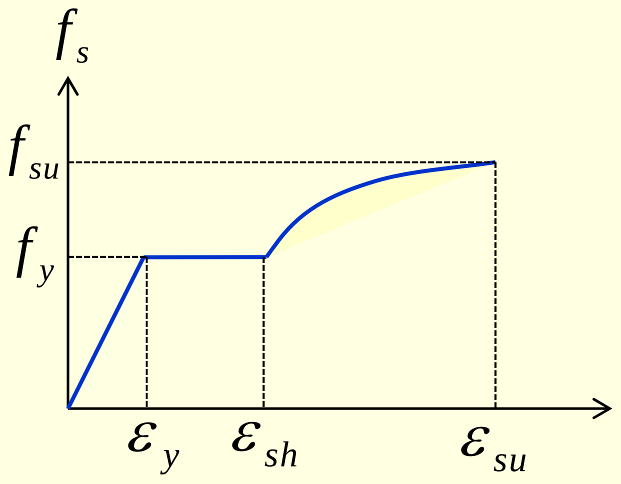
$$f_c = \frac{f'_{cc}xr}{r-1+x^r} \qquad f'_{cc} = f'_c$$

鋼筋組成律

完全彈塑性



應變硬化模式



$$f_s = E_s \varepsilon_s \;, \varepsilon_s \le \varepsilon_y$$

$$f_s = f_y \;, \varepsilon_y < \varepsilon_s \le \varepsilon_{sh}$$

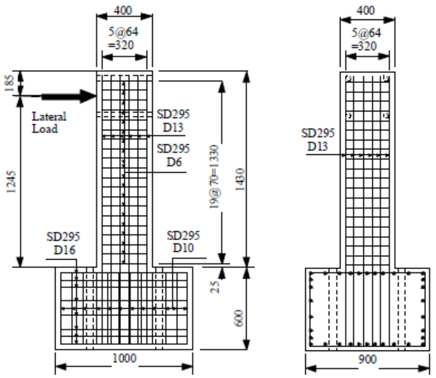
$$f_s = E_s \varepsilon_s \;, \varepsilon_s \le \varepsilon_y$$

$$f_s = f_y \;, \varepsilon_y < \varepsilon_s \le \varepsilon_{sh}$$

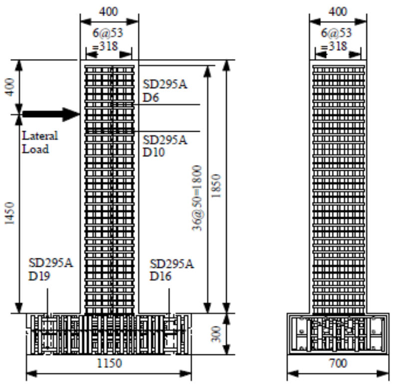
$$\varepsilon_{sh} < \varepsilon_s \le \varepsilon_{su}$$

$$f_s = \left[\frac{m(\varepsilon_s - \varepsilon_{sh}) + 2}{60(\varepsilon_s - \varepsilon_{sh}) + 2} + \frac{(\varepsilon_s - \varepsilon_{sh})(60 - m)}{2(30 r_s + 1)^2} \right]$$

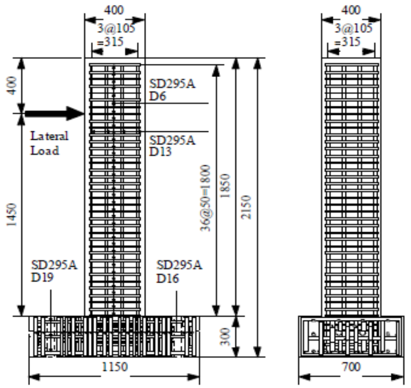
日本土木學會試驗資料　分析比對



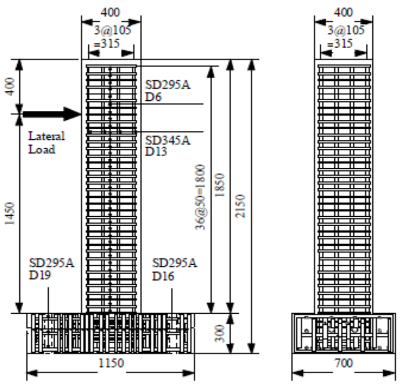
TP2



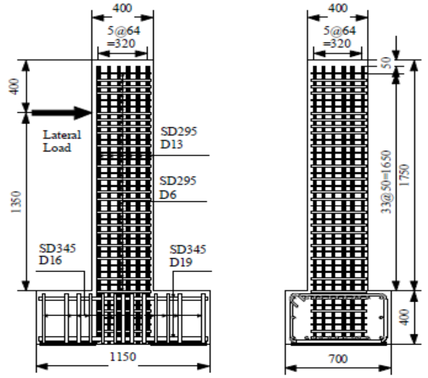
TP10



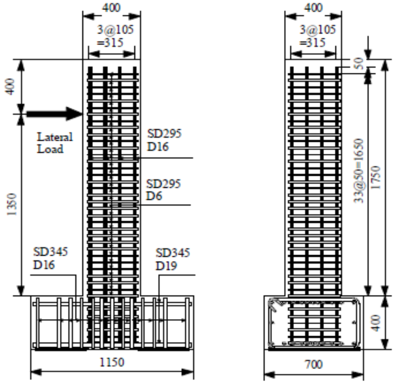
TP11



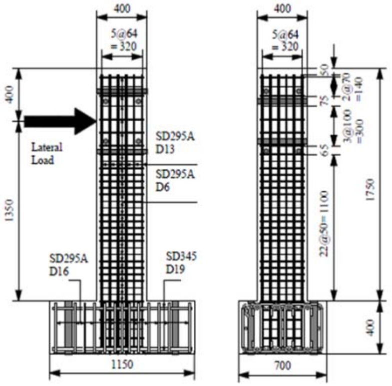
TP13



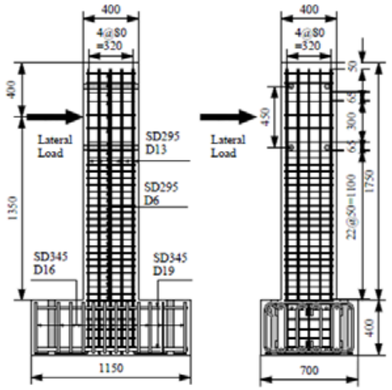
TP29



TP30

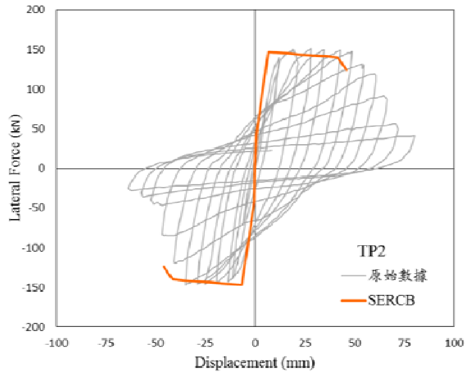


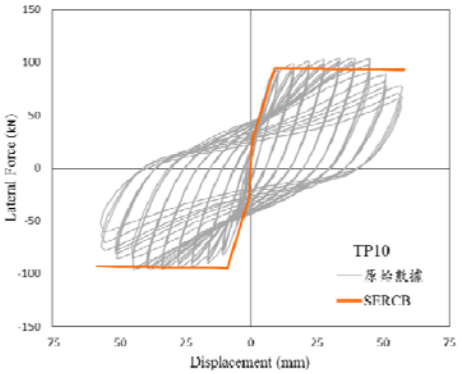
TP68

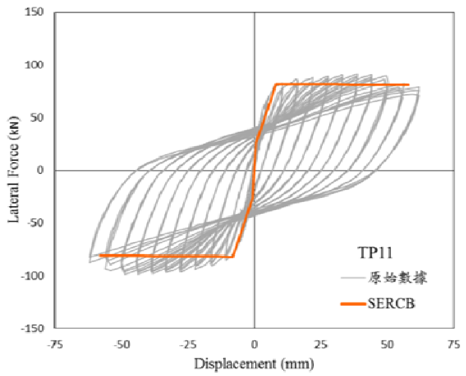


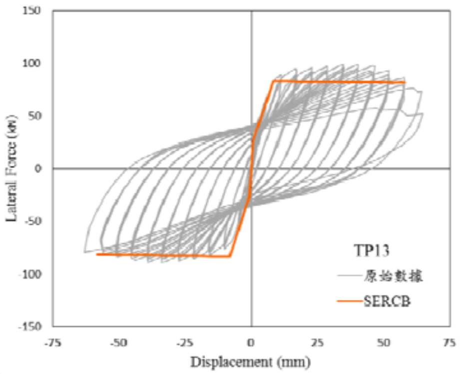
TP74

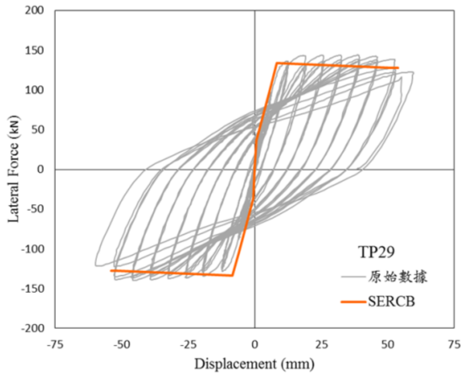
日本土木學會試驗資料　分析比對

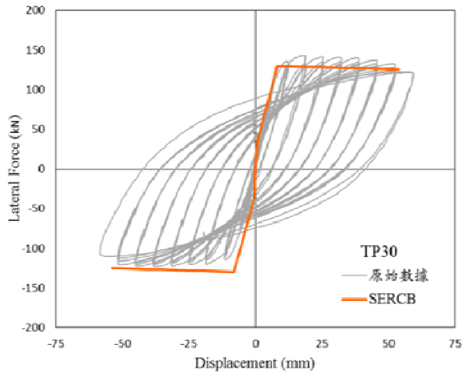


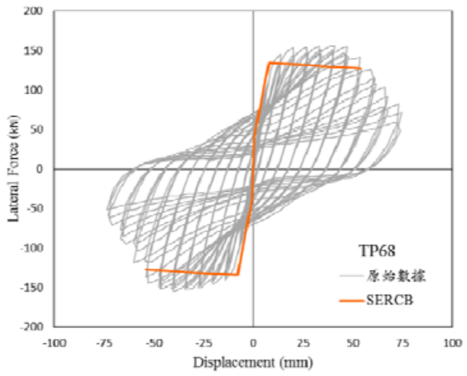


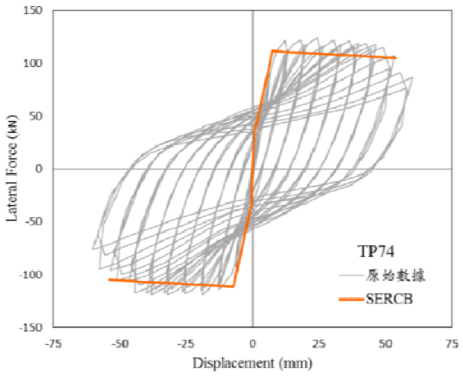






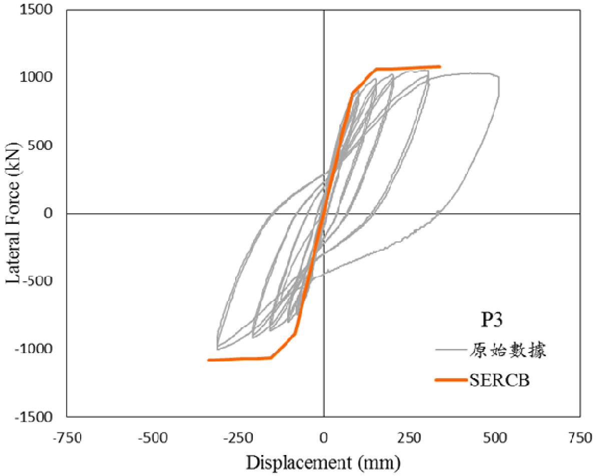


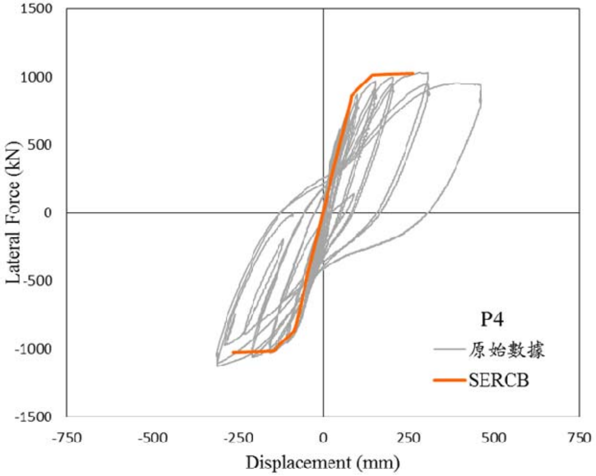




牛鬥橋單柱試體

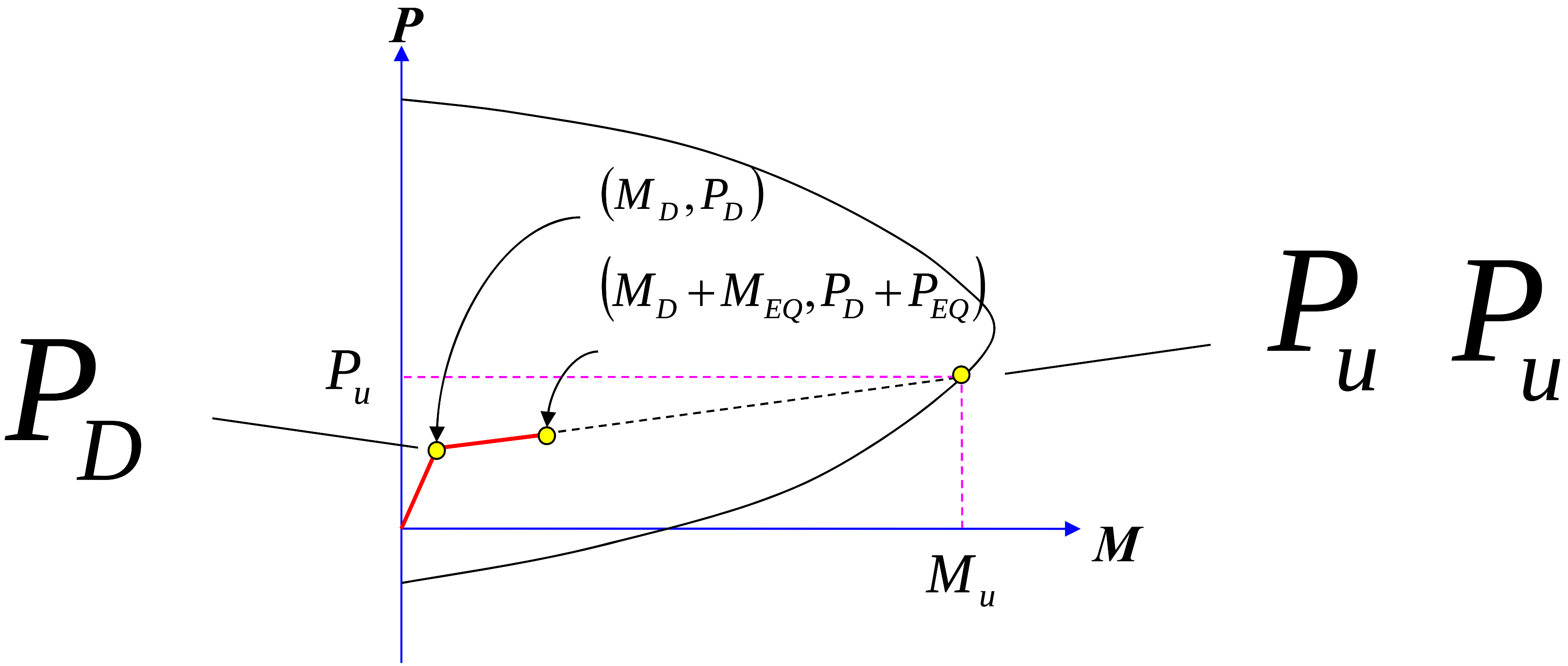

NCREE
NCREE


1500
1000
500
0
-500
-1000
-1500
Lateral Force (kN)
-750
-500
-250
0
250
500
750
Displacement (mm)
P3
原始數據
SERCB


1500
1000
500
0
-500
-1000
-1500
Lateral Force (kN)
-750
-500
-250
0
250
500
750
Displacement (mm)
P4
原始數據
SERCB

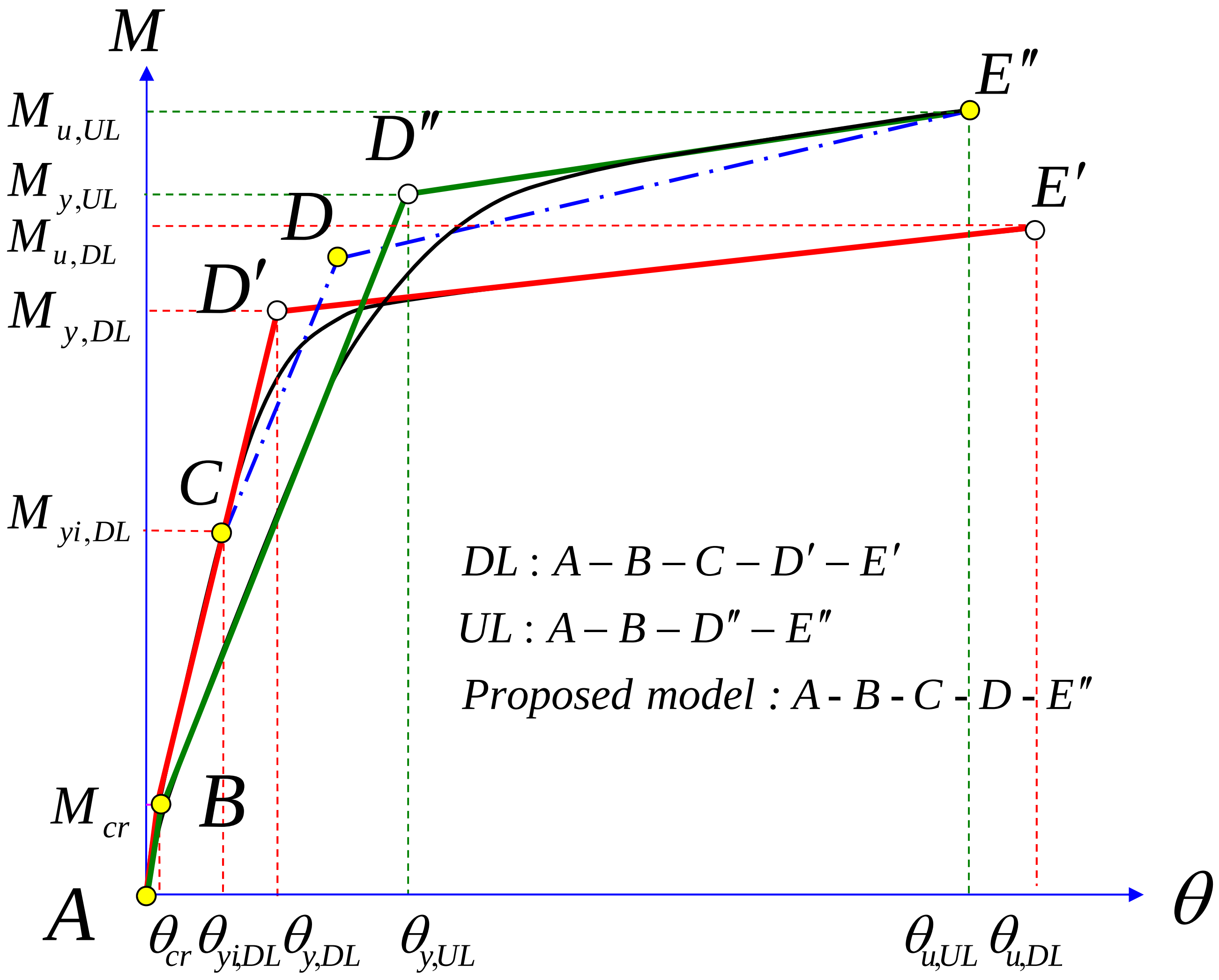
改良式構架鋼筋混凝土柱
塑鉸之設定

【軸力-彎矩交互關係圖】



尋找柱斷面軸力

考慮軸力變化之塑鉸定義方式



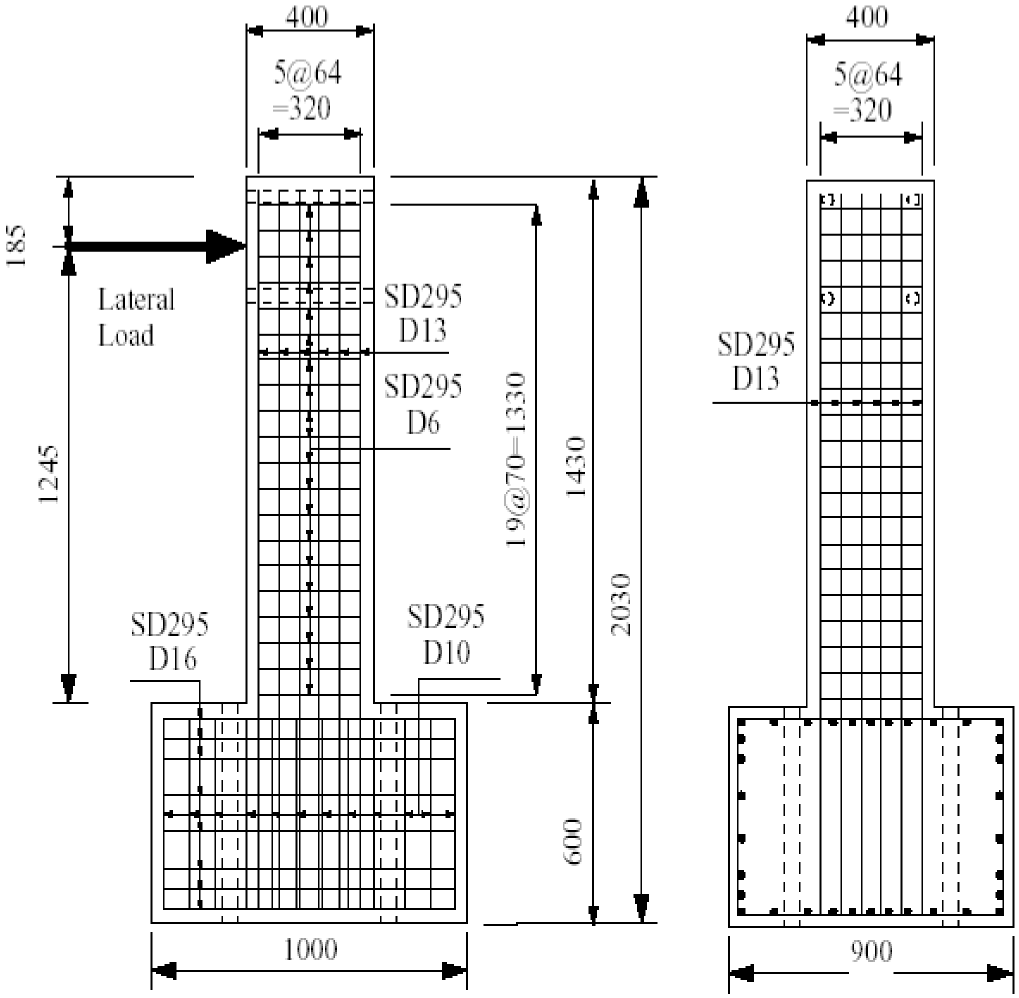
$DL: A - B - C - D' - E'$

$UL: A - B - D'' - E''$

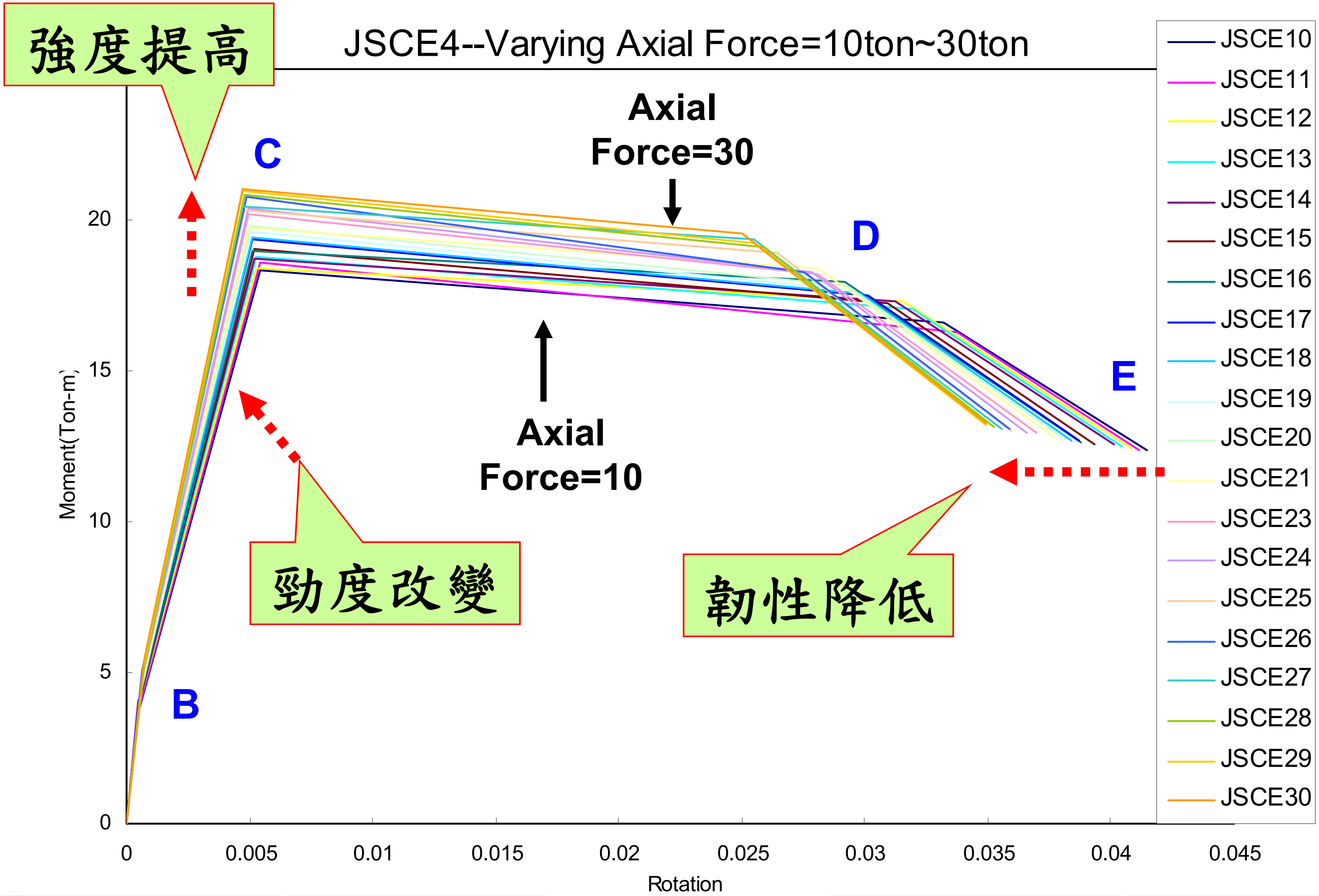
Proposed model : A - B - C - D - E''

Case1.混凝土固定(fc`=350)-軸力變化(10ton~30ton)

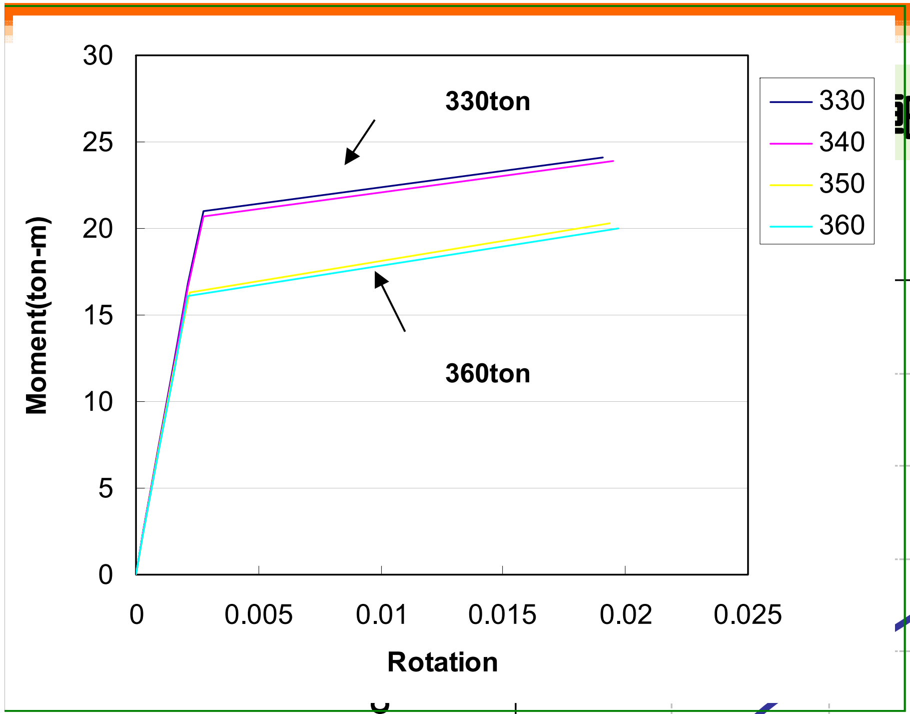
JSCE4

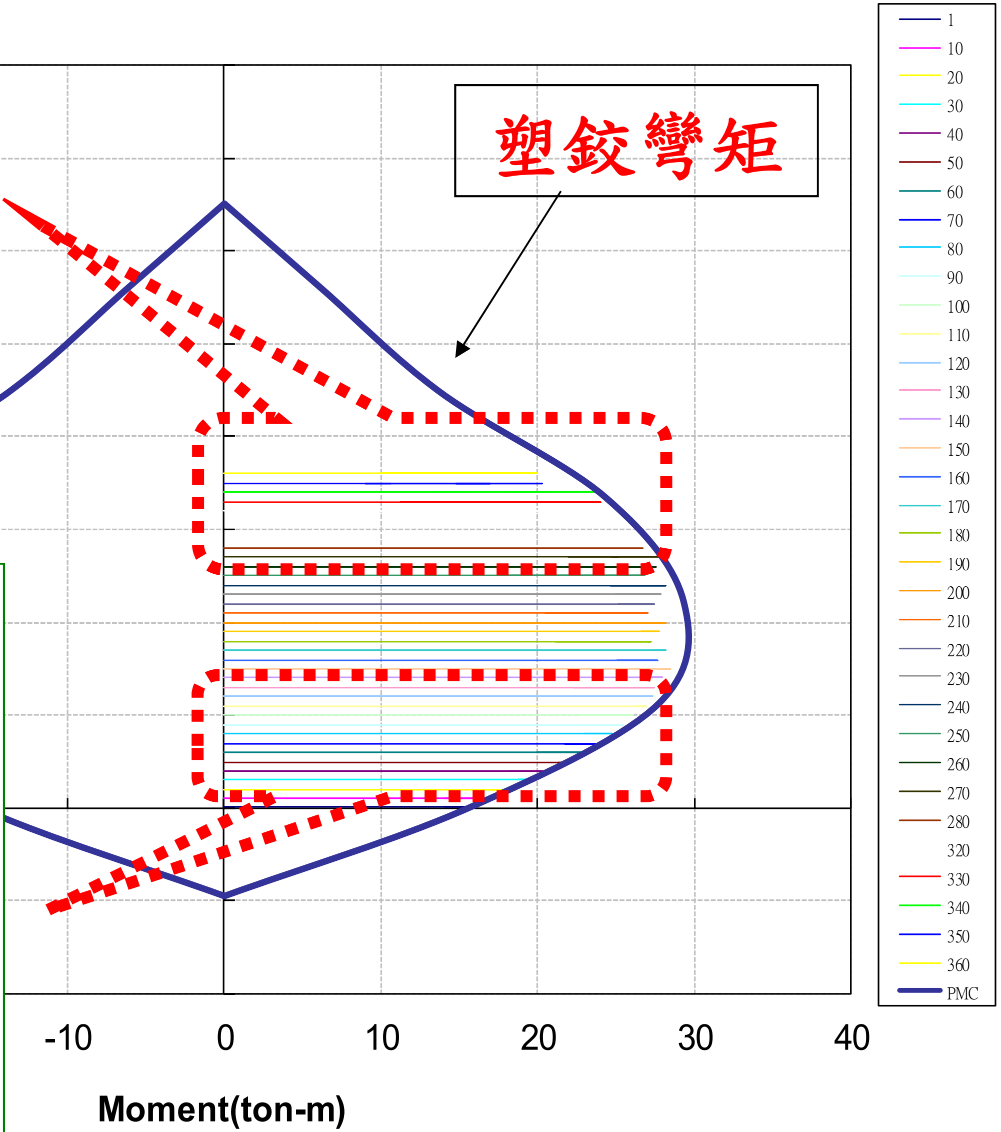


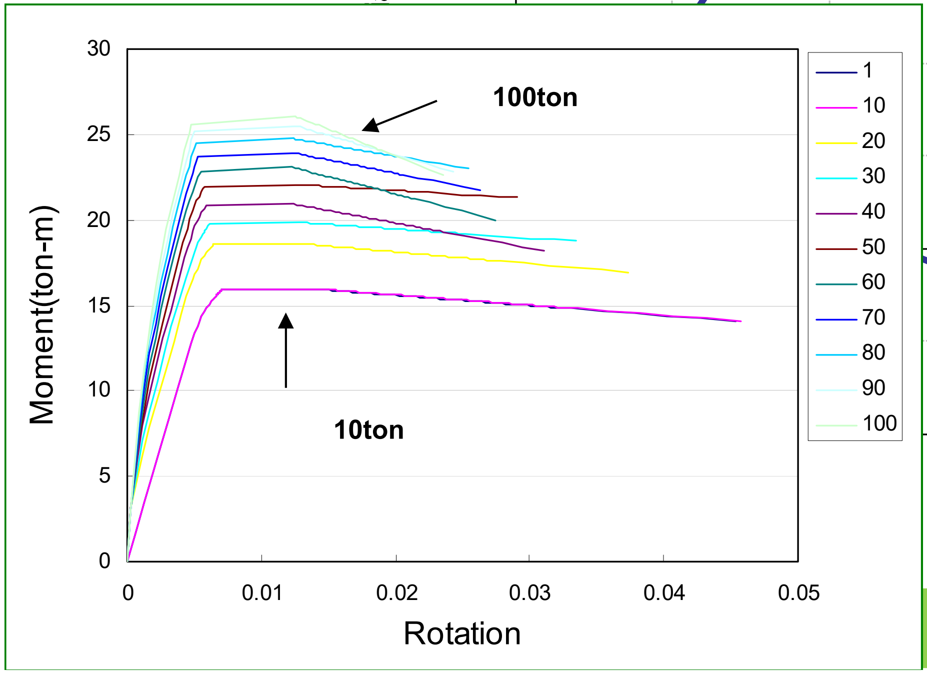
Case1.混凝土固定(fc`=350)-軸力變化(10ton~30ton)



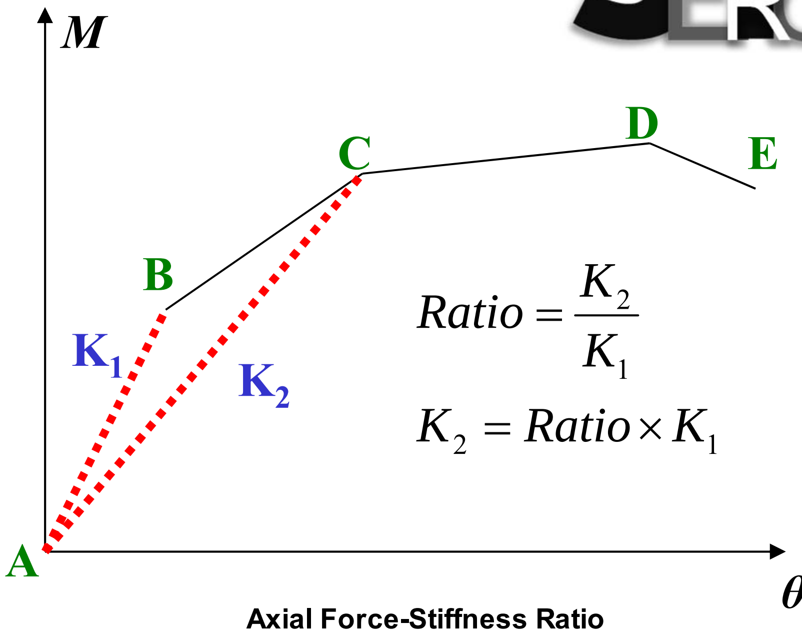
力變化(1ton~360ton)





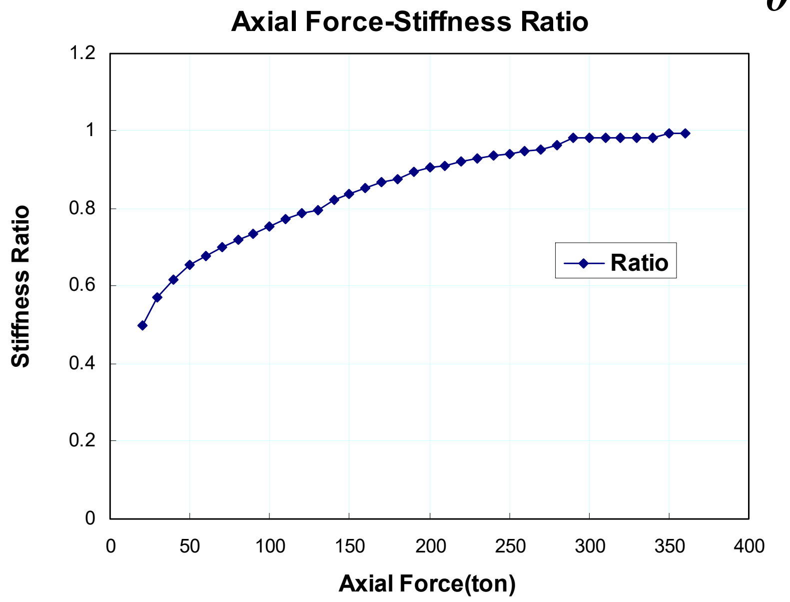


Axial Force	Ratio
20(ton)	0.500126
30	0.572653
40	0.617576
50	0.654692
60	0.677131
70	0.701033
80	0.721649
90	0.737022
100	0.752531
110	0.773794
120	0.788129
130	0.797386
140	0.821175
150	0.838802
160	0.851769
170	0.869229
180	0.875656
190	0.895279
200(ton)	0.905517
210	0.910601
220	0.923186
230	0.93012
240	0.936486
250	0.940281
260	0.94852
270	0.953338
280	0.962182
290	0.983667
300	0.983783
310	0.983838
320	0.984352
330	0.984071
340	0.98372
350	0.994867
360	0.993796

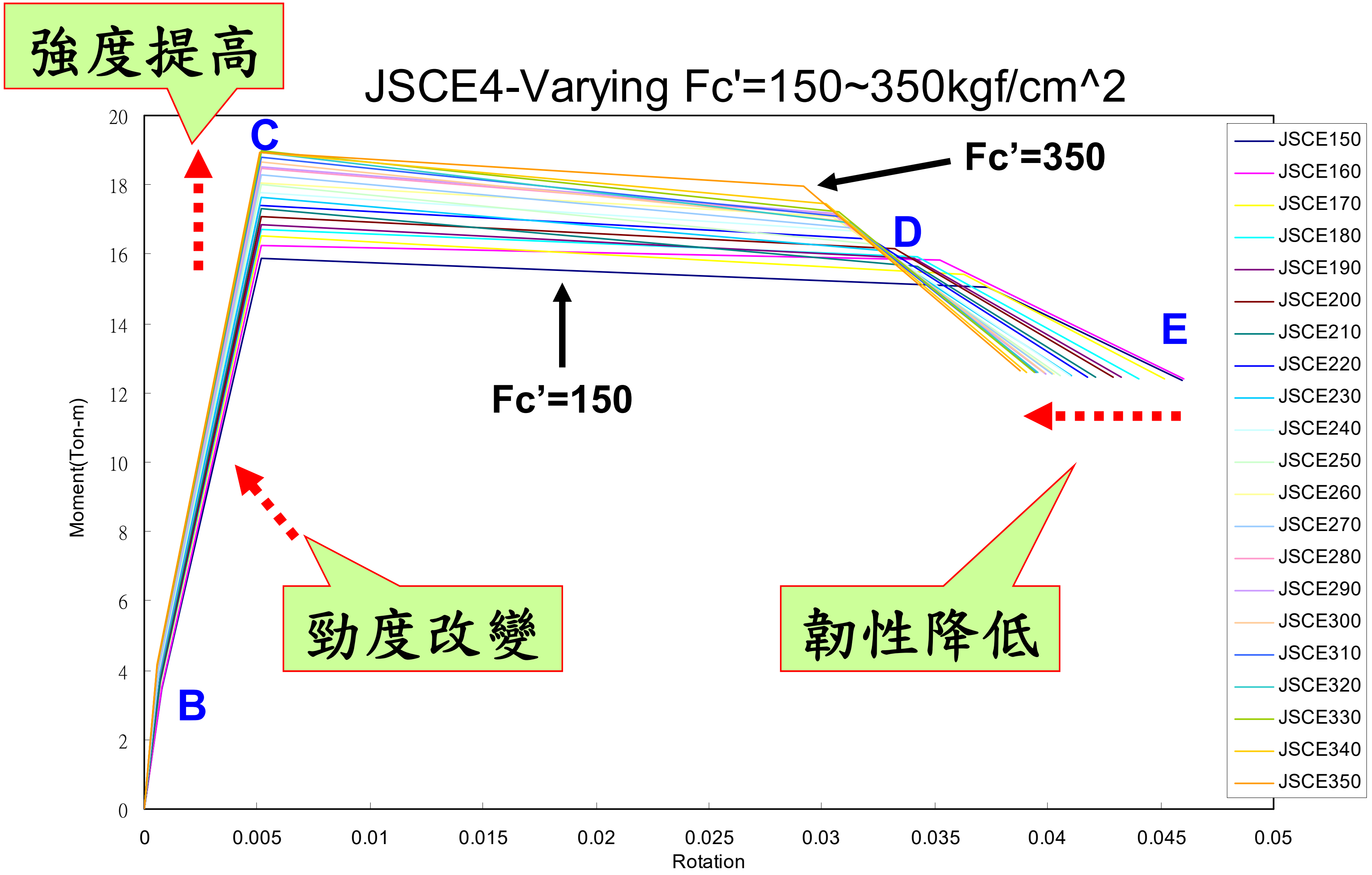


$$Ratio = \frac{K_2}{K_1}$$

$$K_2 = Ratio \times K_1$$



Case2.混凝土變化(fc`=150~350)-軸力固定(16ton)



Fc'=350

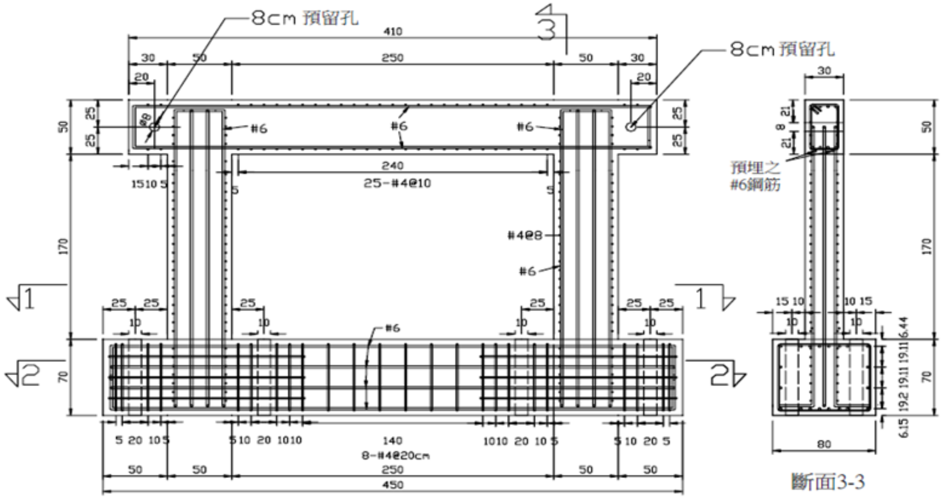
Fc'=150

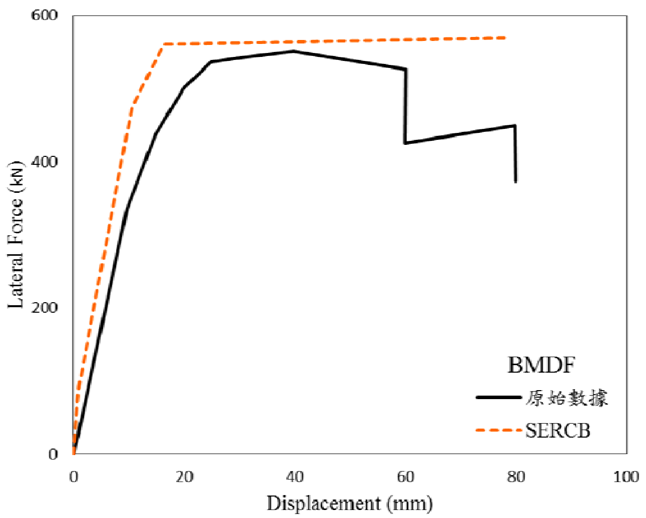
強度提高

勁度改變

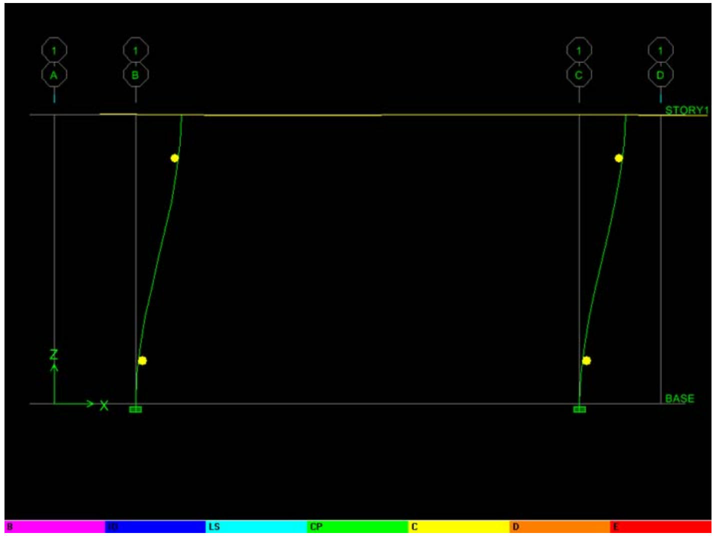
韌性降低

構架BMDF試體

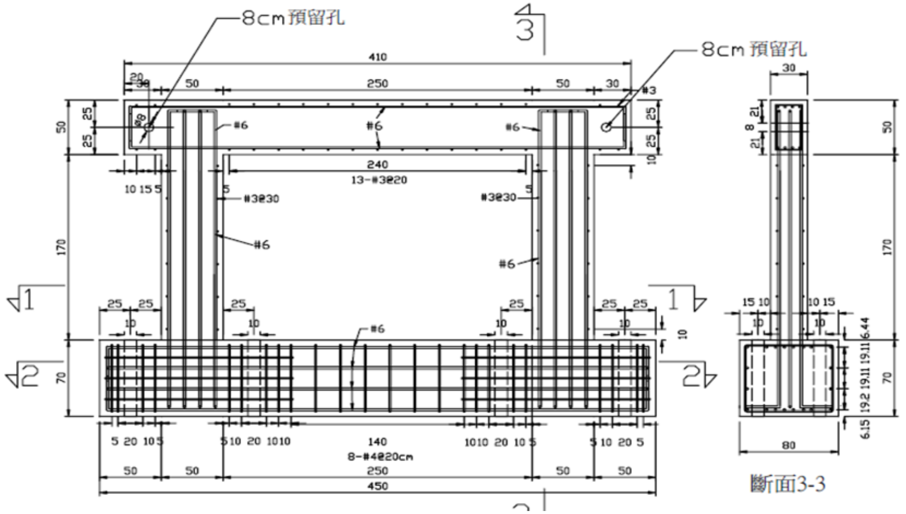


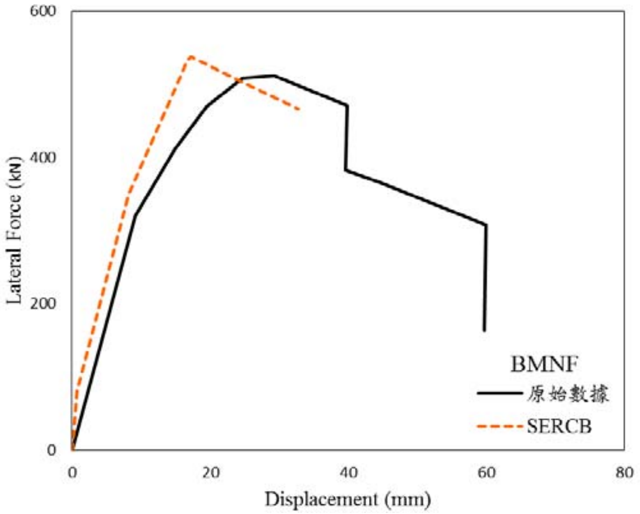


(含開口RC牆非韌性構架之耐震行為研究，黃世建、陳力平、陳俊宏，2003)

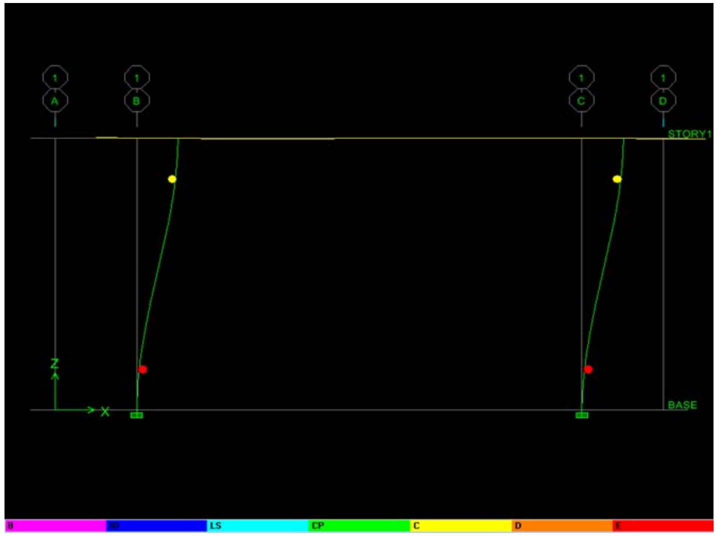


構架BMNF試體

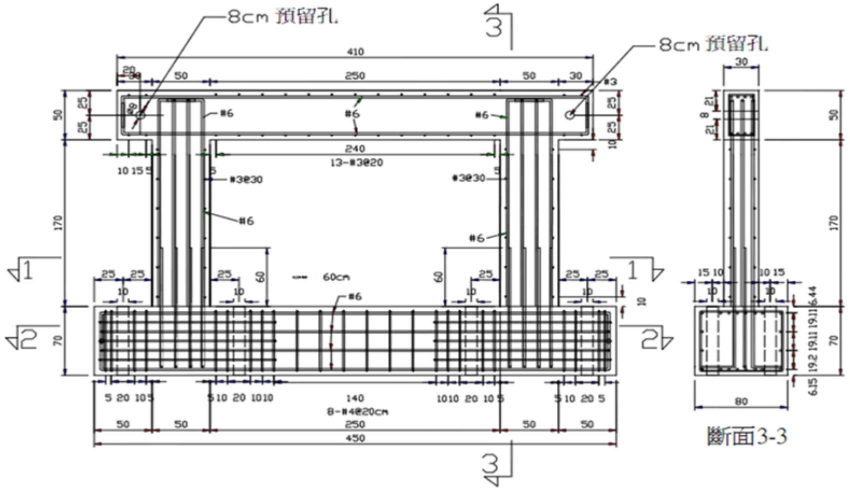


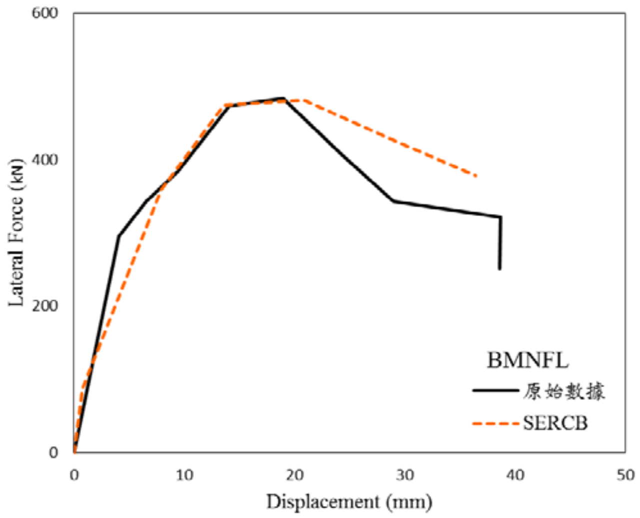


(含開口RC牆非韌性構架之耐震行為研究，黃世建、陳力平、陳俊宏，2003)

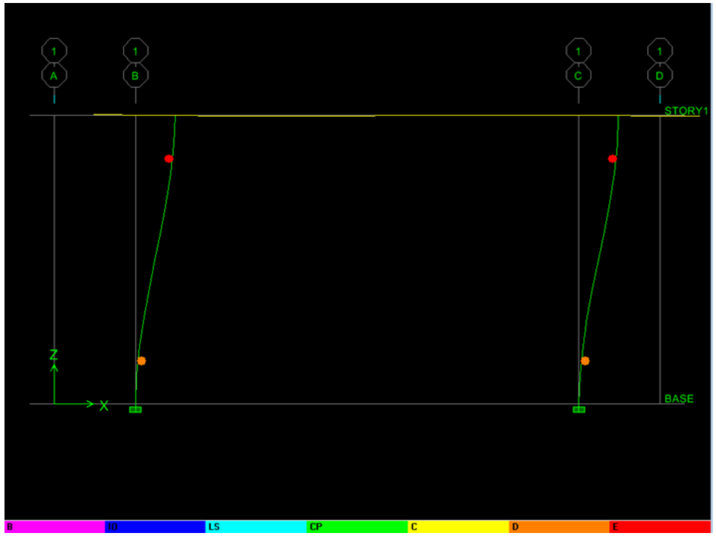


構架BMNFL試體

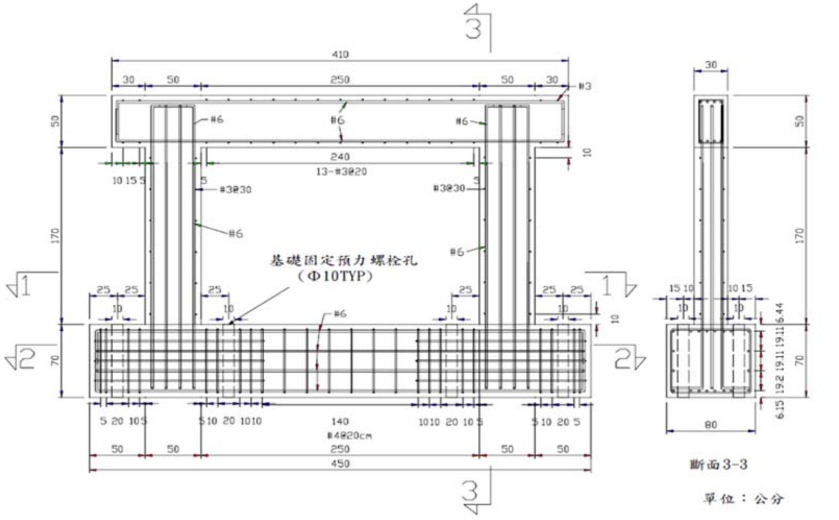


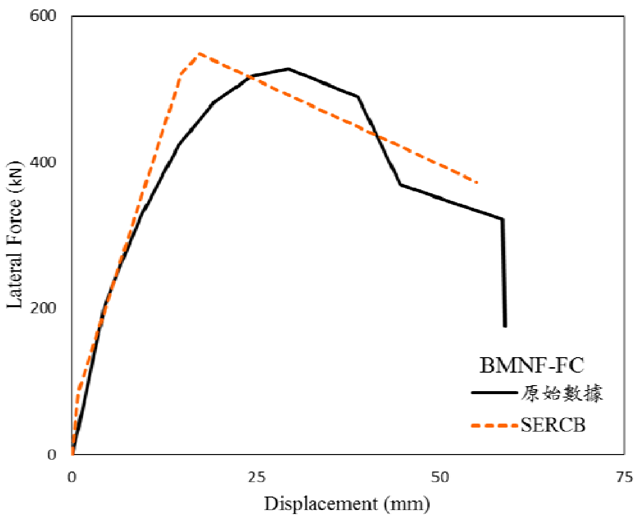


(SCFRP於含磚牆非韌性構架之耐震補強實驗與分析，陳振華、黃炯憲、張國鎮、陳俊宇，2003)

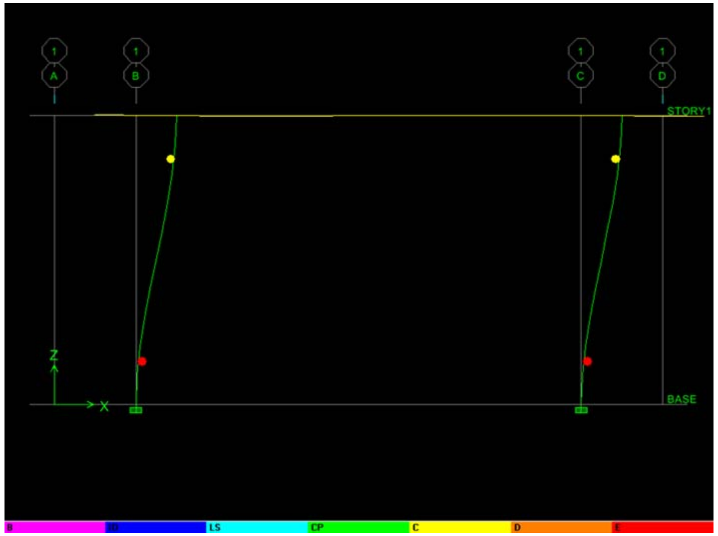


構架BMNF-FC試體

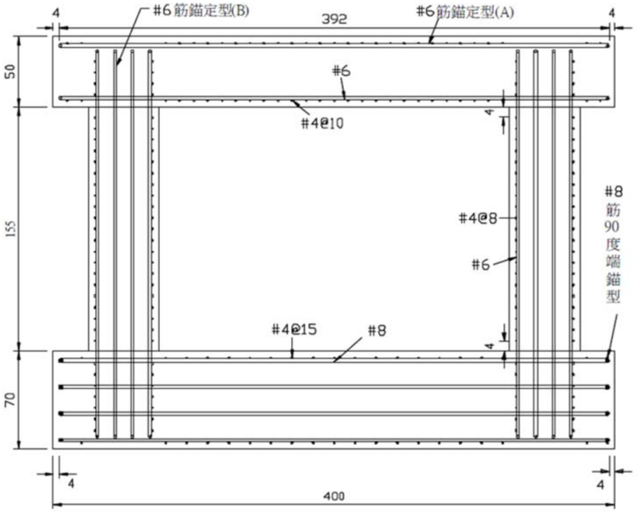


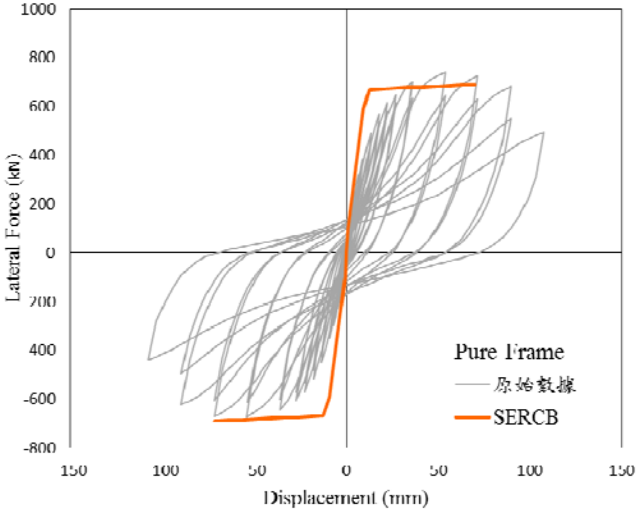


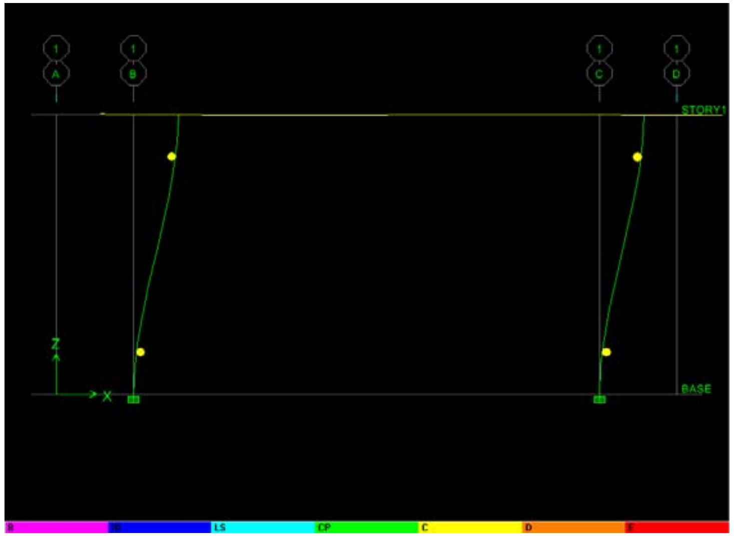
(含牆RC構架之耐震分析及設計研究，黃昭勳、蔡驥鑫，2003)



構架Pure Frame試體

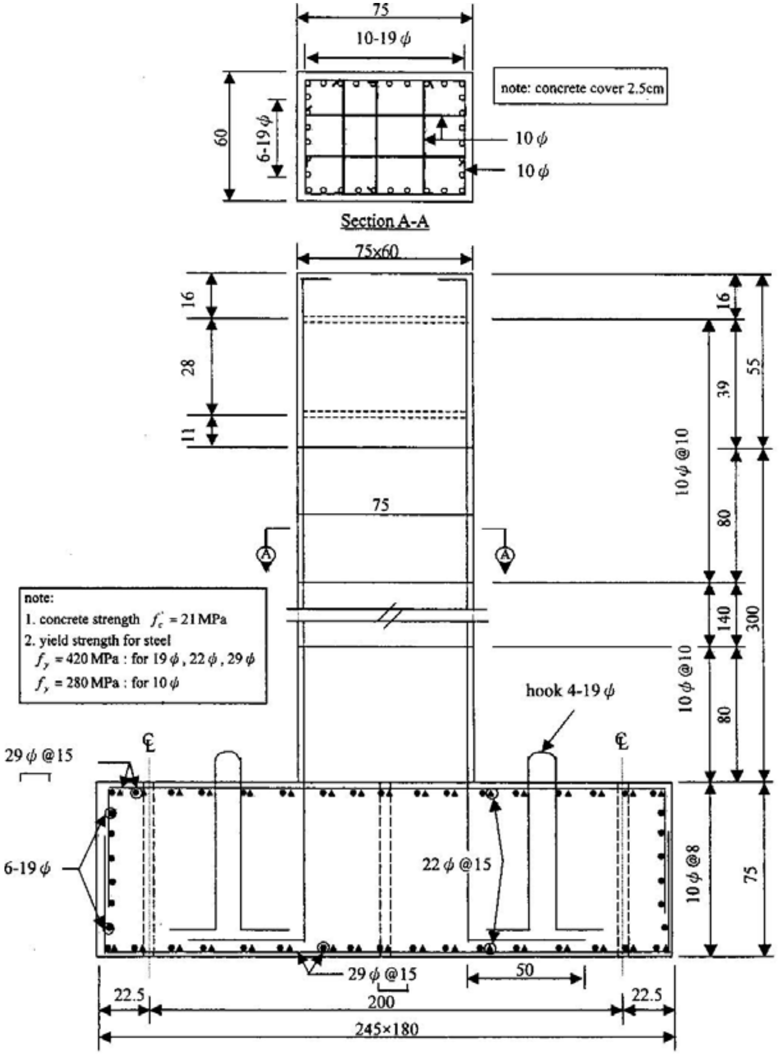


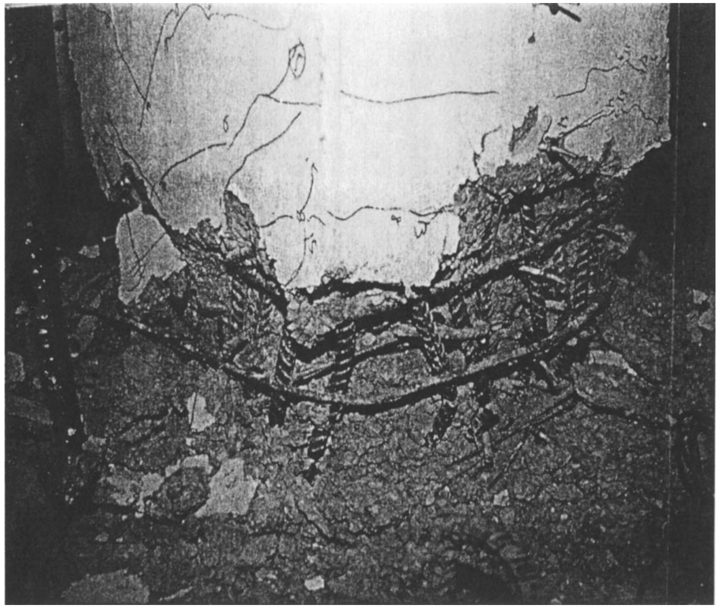




(鋼筋混凝土牆之碳纖FRP
耐震補強研究，黃世建、
葉永信，2001)

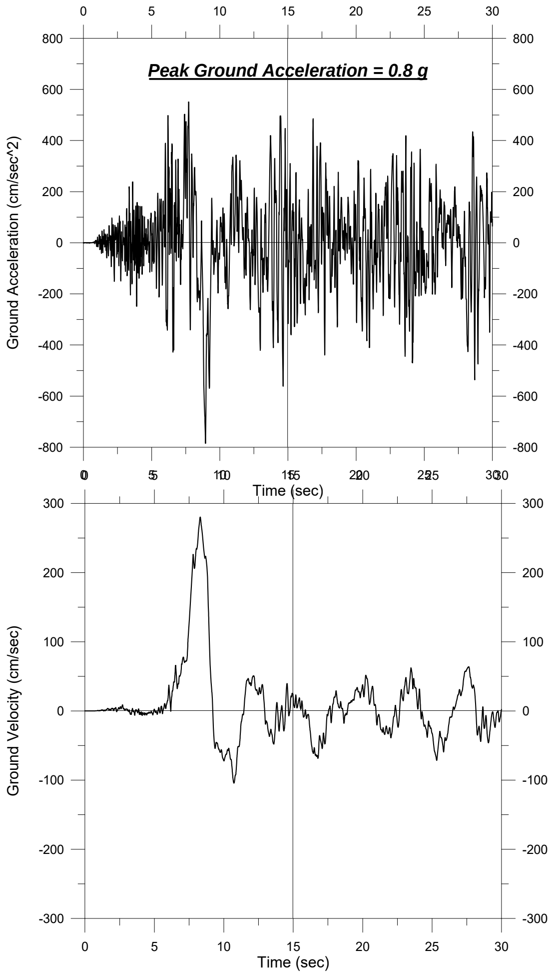
Pseudodynamic test of reinforced concrete columns



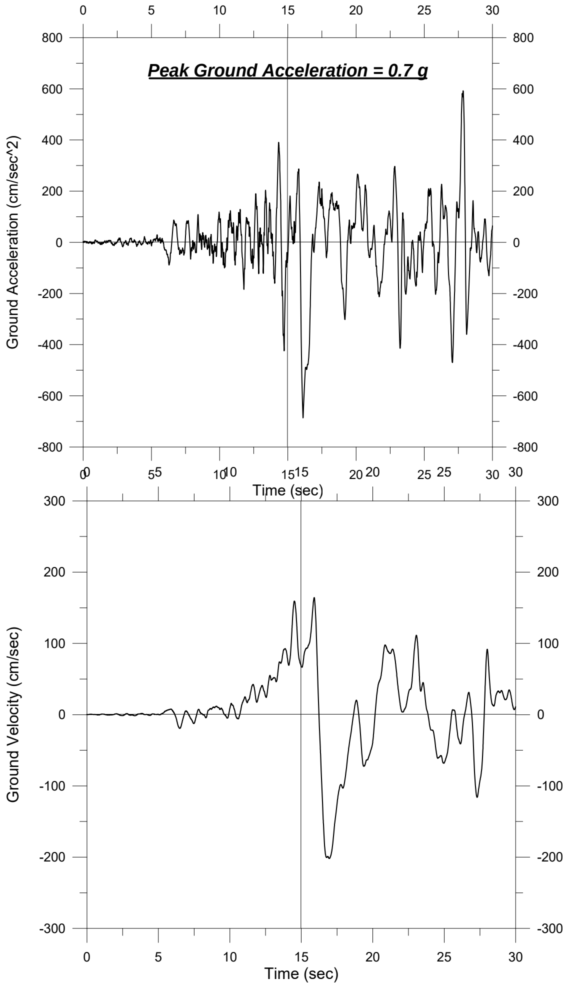


Ground acceleration and velocity for TCU075、TCU102

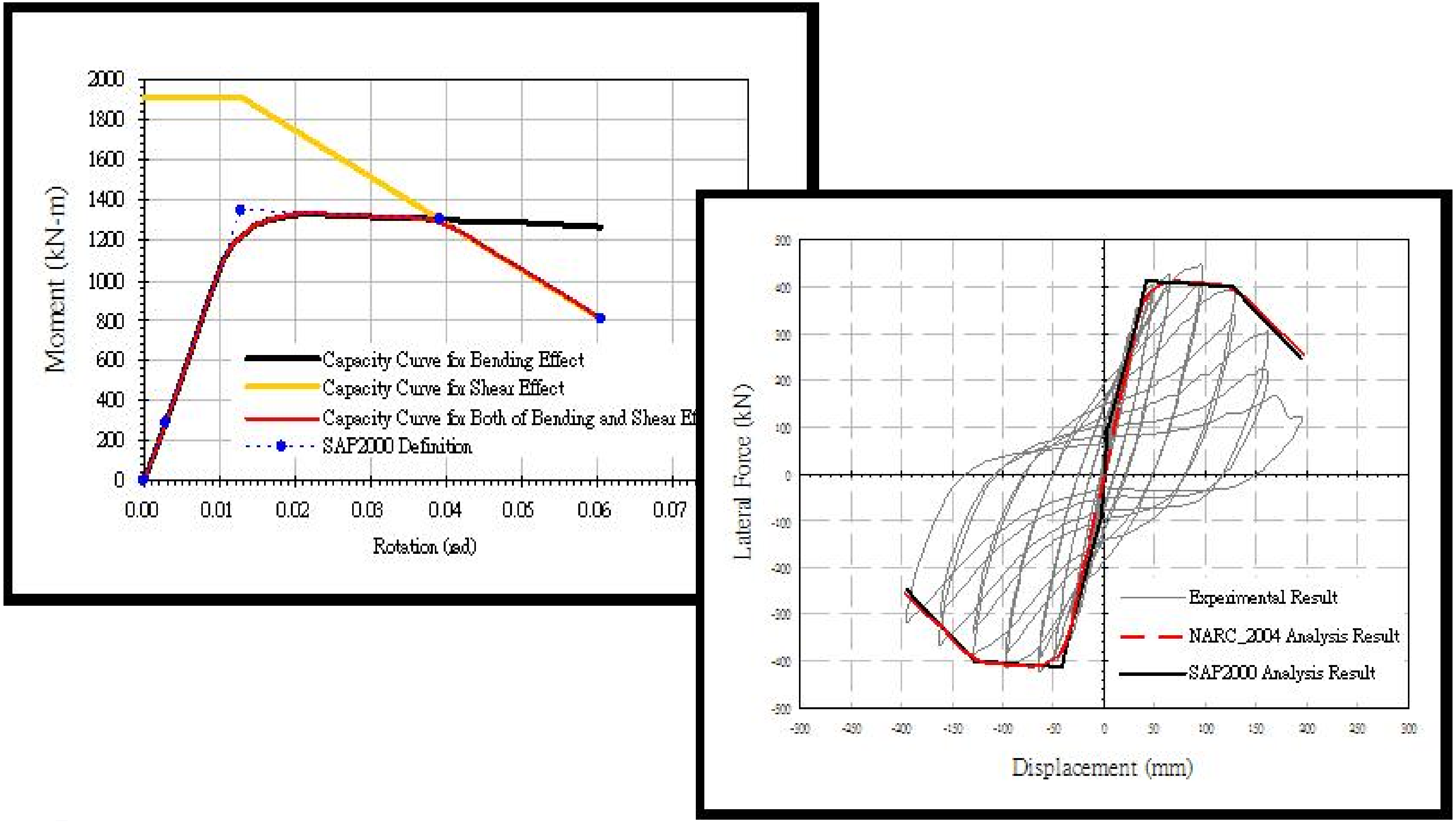
Ground acceleration and velocity for TCU075



Ground acceleration and velocity for TCU102

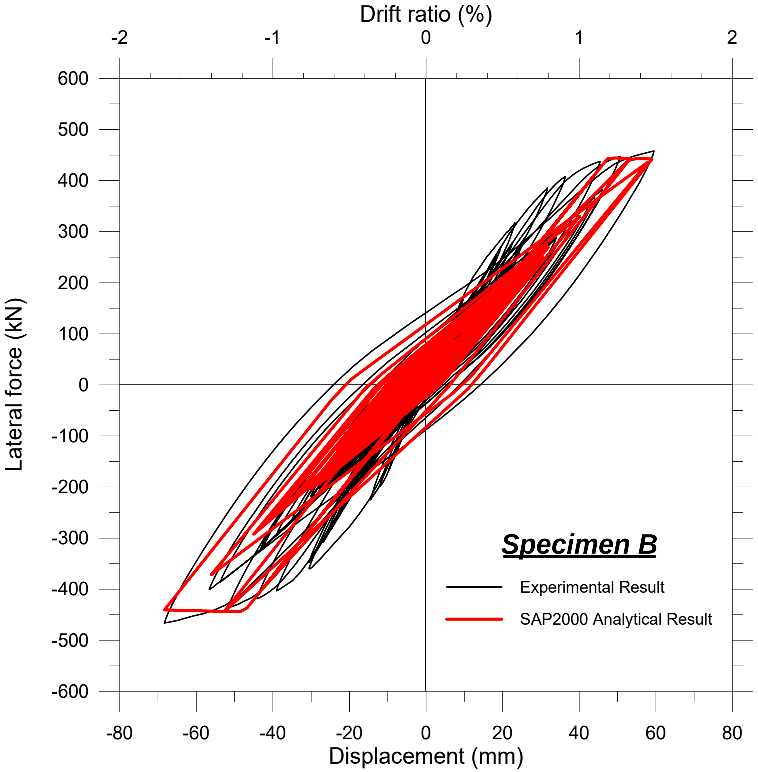


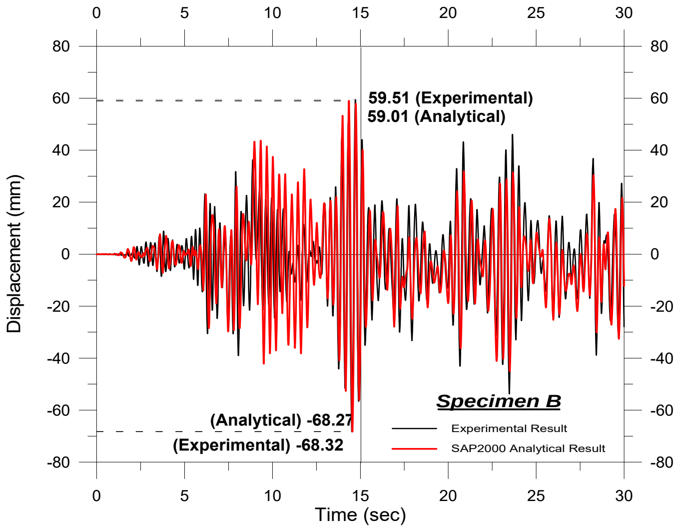
Specimen A -
cyclic loading test and analysis result

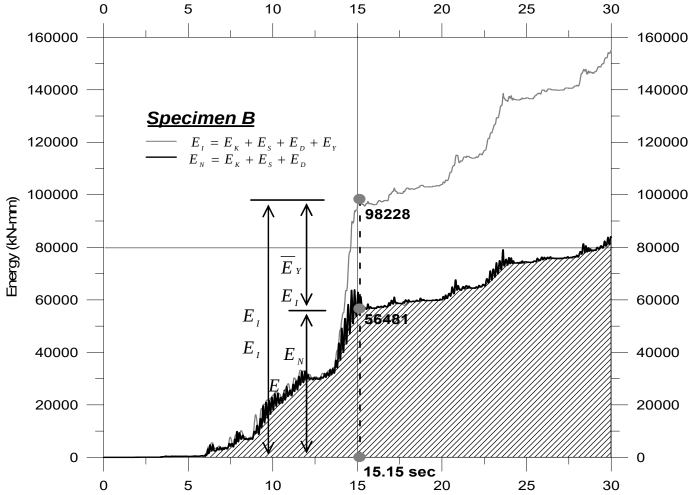


Specimen B -

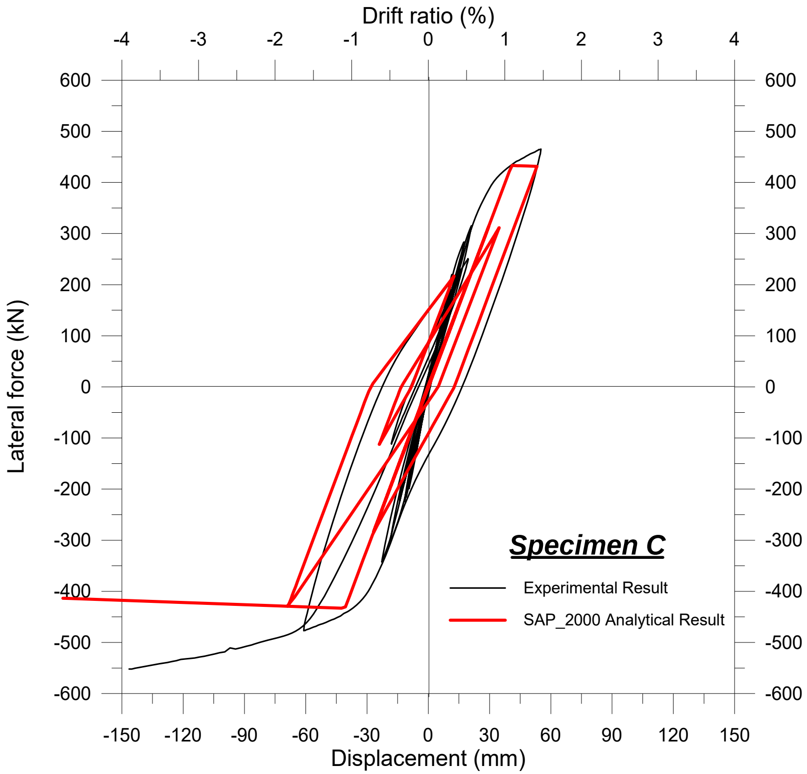
Pseudodynamic and analysis results for as-built Specimen subject to TCU075

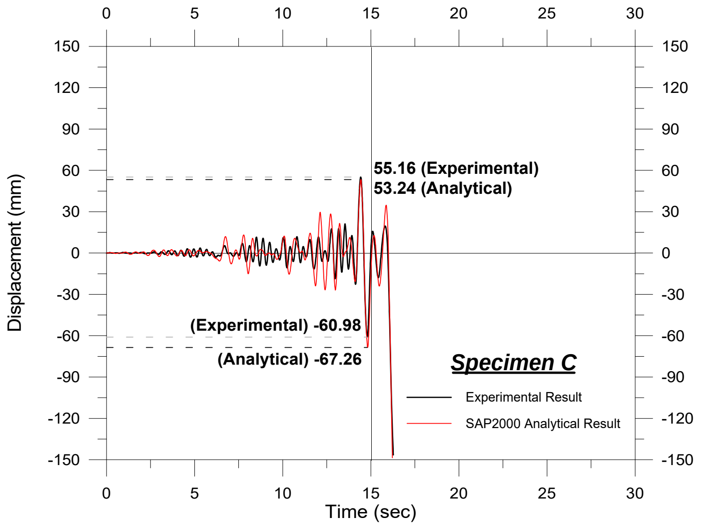


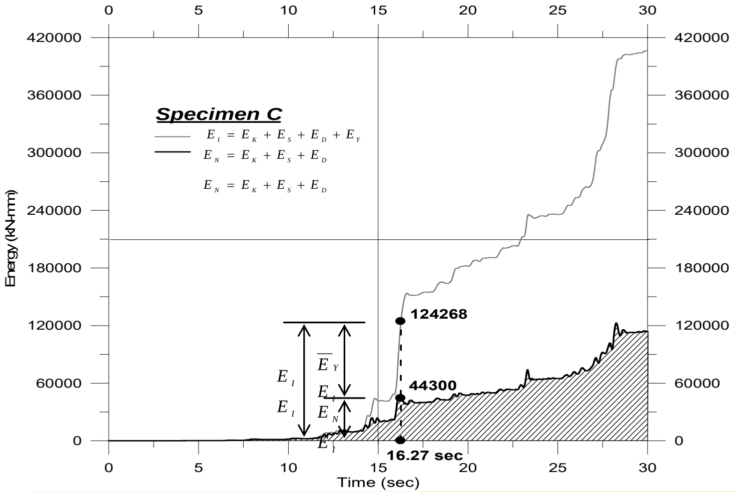




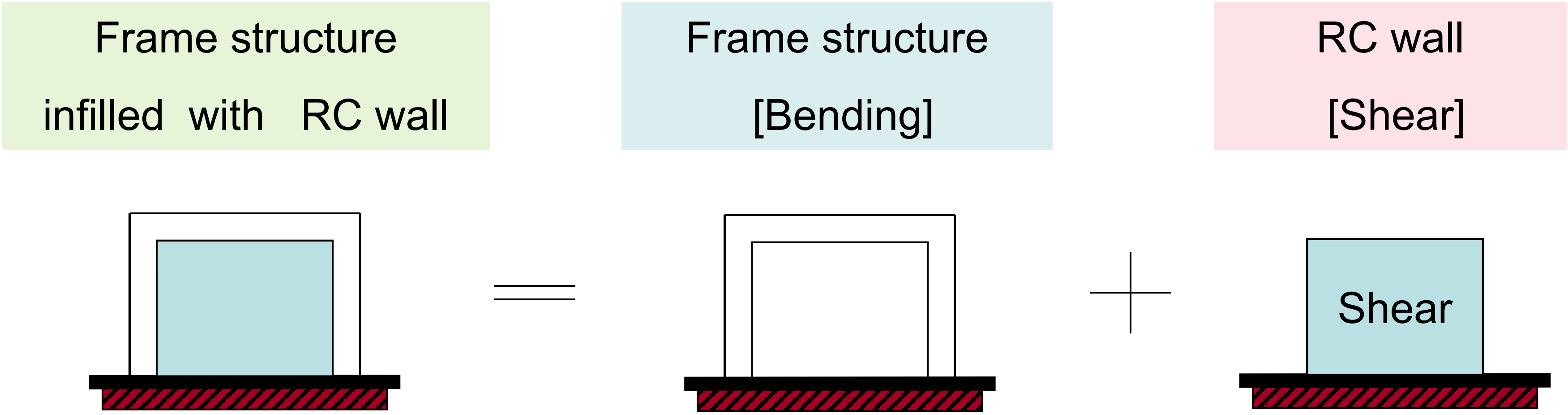
Specimen C –
Pseudodynamic and analysis results for as-built Specimen subject to TCU102







RC牆模擬分析



Frame structure

infilled with RC wall

=

Frame structure

[Bending]

+

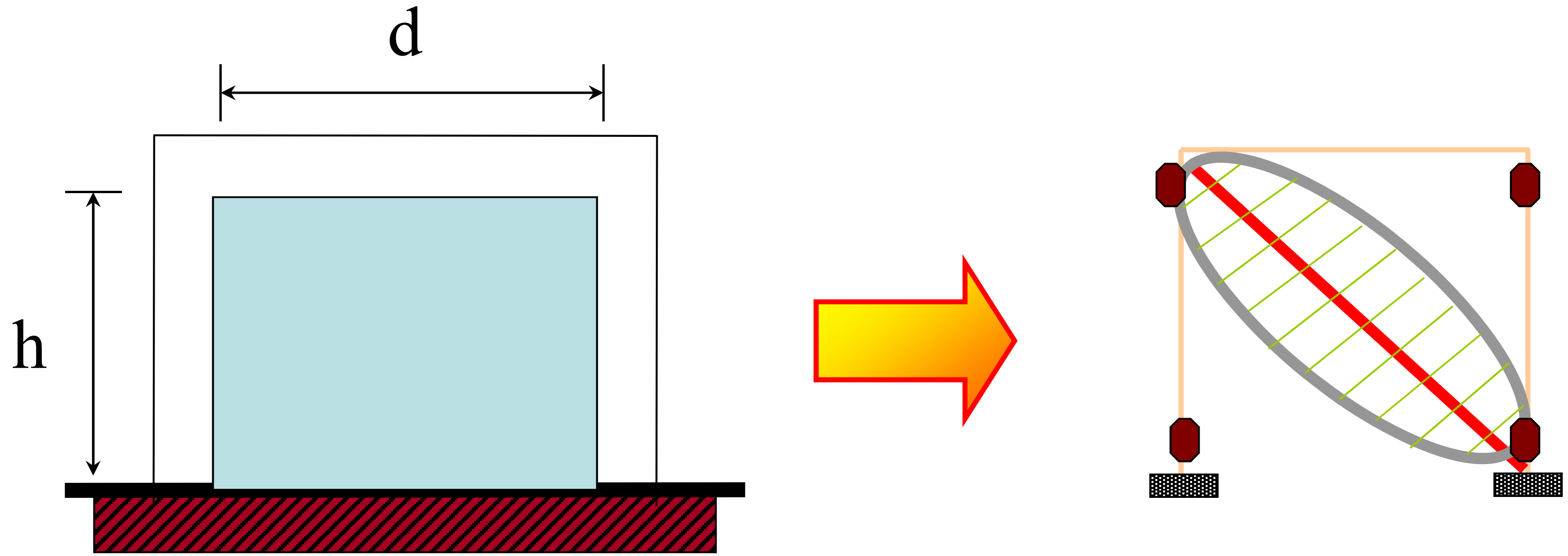
RC wall

[Shear]

Pushover analysis is highly sensitive to the structural nonlinearity of frame and RC wall

Lai, M.C., **Sung, Y.C.**, "**A Study on Pushover Analysis of Frame Structure Infilled with Low-rise Reinforced Concrete Wall**", *Journal of Mechanics*, Vol. 24, 2008, p.p. 437-449

RC Structural Wall



Soften Strut-Tie Model

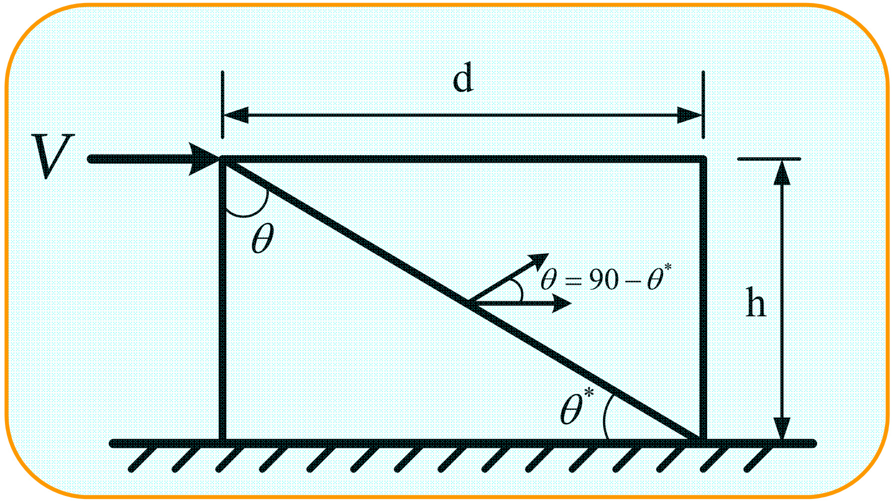
Equivalent diagonal structural strut of an RC wall

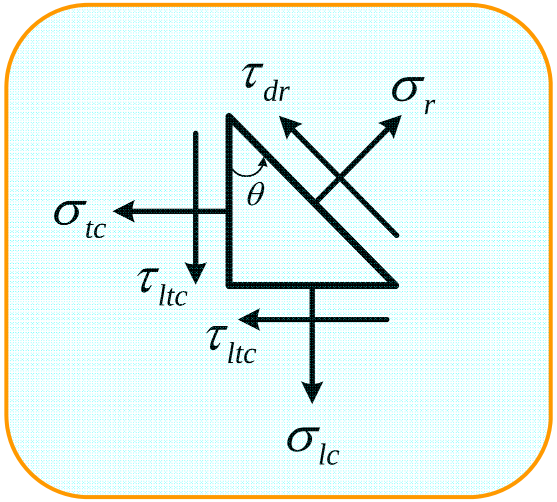
Lai, M.C., **Sung, Y.C.**, “**A Study on Pushover Analysis of Frame Structure Infilled with Low-rise Reinforced Concrete Wall**”, *Journal of Mechanics*, Vol. 24, 2008, p.p. 437-449

Equilibrium (1/2)

Based on the theory of fixed angle softened truss model, the angle of cracks in the postcracking concrete coincides with the angle θ. Otherwise, the horizontal angle of shear wall is θ*.

$$\theta^{*} = \tan^{-1}\left(\frac{h}{d}\right) \qquad \theta = 90 - \theta^{*}$$

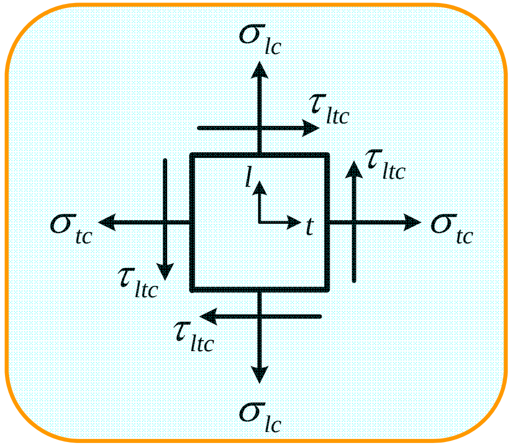




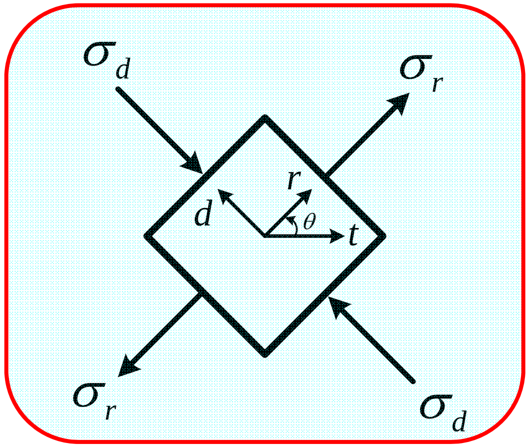
Equilibrium (2/2)

The equilibrium can be expressed as following Eqs., according to the Mohr circle of stress.

$(\sigma_d - \sigma_r)$



$(\sigma_{lc}, \sigma_{tc}, \tau_{ltc})$



$$\sigma_{lc} = \sigma_d \cos^2\theta + \sigma_r \sin^2\theta$$

$$\sigma_{tc} = \sigma_d \sin^2\theta + \sigma_r \cos^2\theta$$

$$\tau_{ltc} = (-\sigma_d + \sigma_r)\sin\theta\cos\theta$$

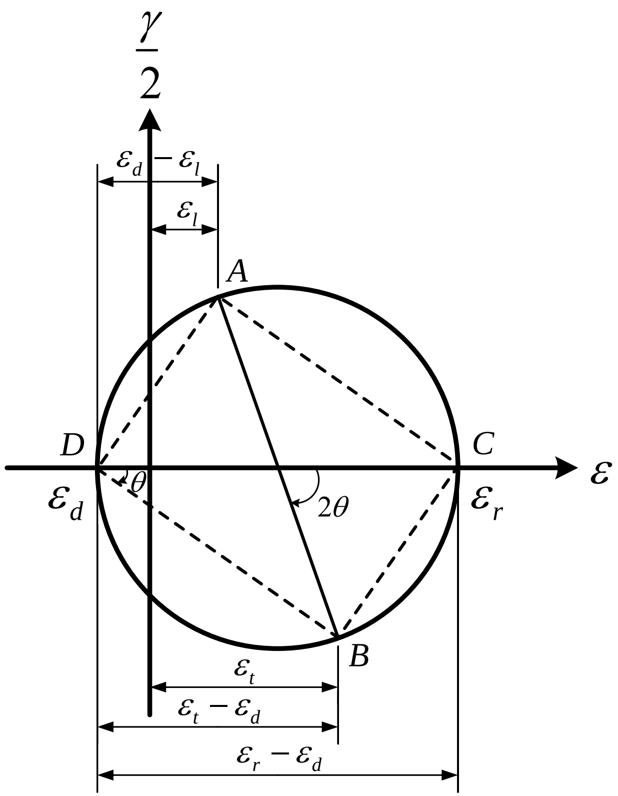
$$V_c = \tau_{ltc} \times b_w \times d$$

The total shear force applied on the RC wall

$$V = V_c + V_s = \tau_{ltc} \times b_w \times d + \rho_t \times b_w \times h \times f_t$$

Compatibility

Based on the theory of fixed angle softened truss model, the compatibility can be expressed as following Eqs., according to the Mohr circle of strain.



$$\varepsilon_l = \varepsilon_d \cos^2\theta + \varepsilon_r \sin^2\theta$$

$$\varepsilon_t = \varepsilon_d \sin^2\theta + \varepsilon_r \cos^2\theta$$

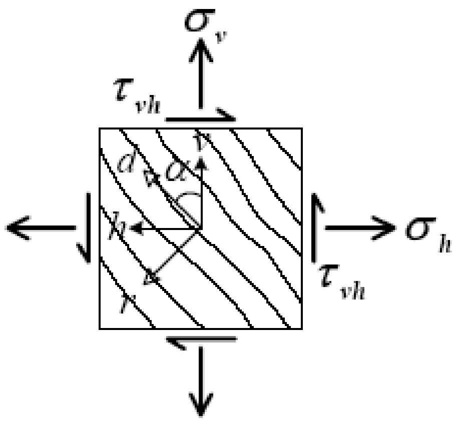
$$\gamma_{lt}/2 = (-\varepsilon_d + \varepsilon_r)\sin\theta\cos\theta$$

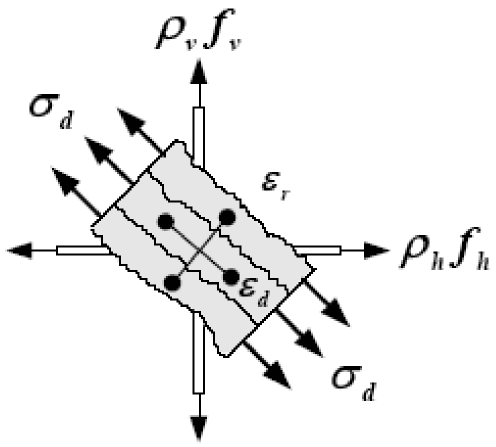
Constitutive Laws of Concrete(1/3)

Both the strength and stiffness of cracked reinforced concrete in compression are lower than those of uniaxial compressed concrete.

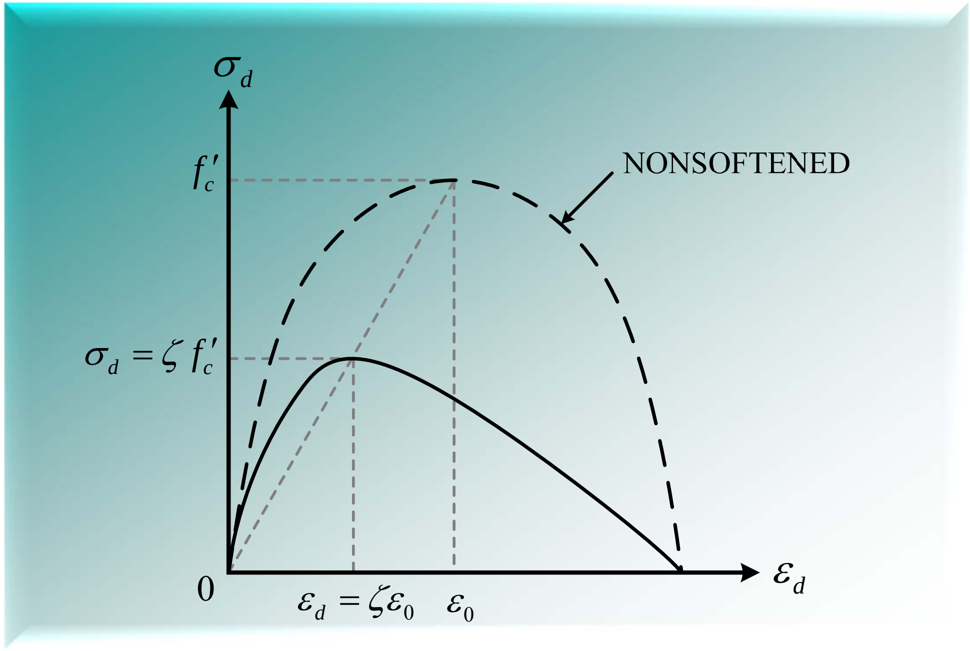
Softening effect on the biaxial constitute laws of concrete

$$\zeta = \frac{0.9}{\sqrt{1+600\varepsilon_r}}$$





Constitutive Laws of Concrete(2/3)



Belarbi and Hsu

$$\varepsilon_d / \zeta\varepsilon_0 \le 1$$

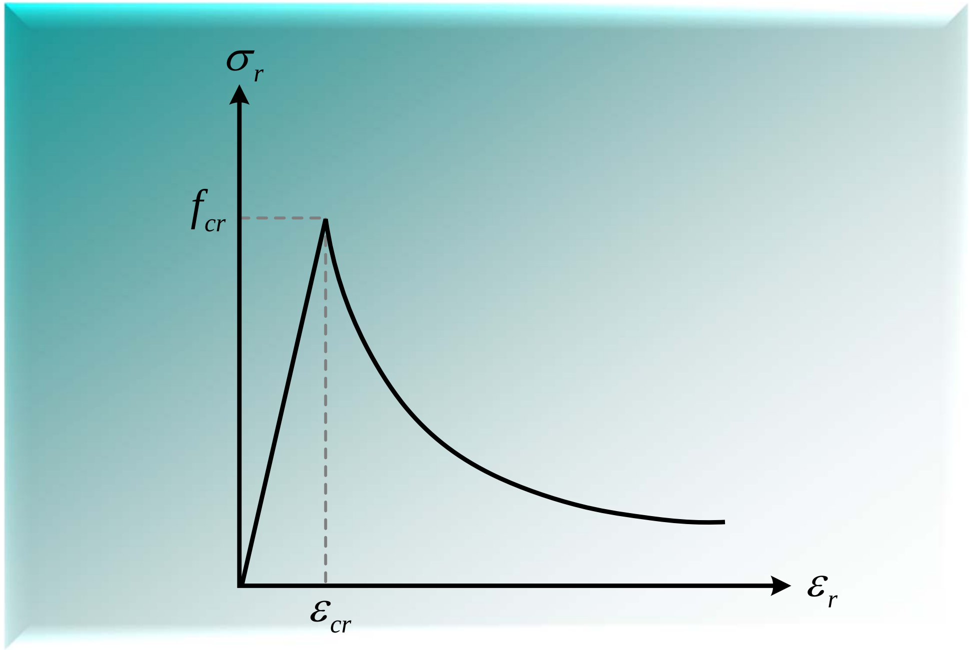
$$\sigma_d = \zeta f'_c \left[2\left(\frac{\varepsilon_d}{\zeta\varepsilon_0}\right) - \left(\frac{\varepsilon_d}{\zeta\varepsilon_0}\right)^2 \right]$$

$$\varepsilon_d / \zeta\varepsilon_0 > 1$$

$$\sigma_d = \zeta f'_c \left[1 - \left(\frac{\varepsilon_d / \zeta\varepsilon_0 - 1}{2/\zeta - 1}\right)^2 \right]$$

$$\zeta = \frac{0.9}{\sqrt{1 + 600\varepsilon_r}}$$

Constitutive Laws of Concrete(3/3)



Vecchio and Collins

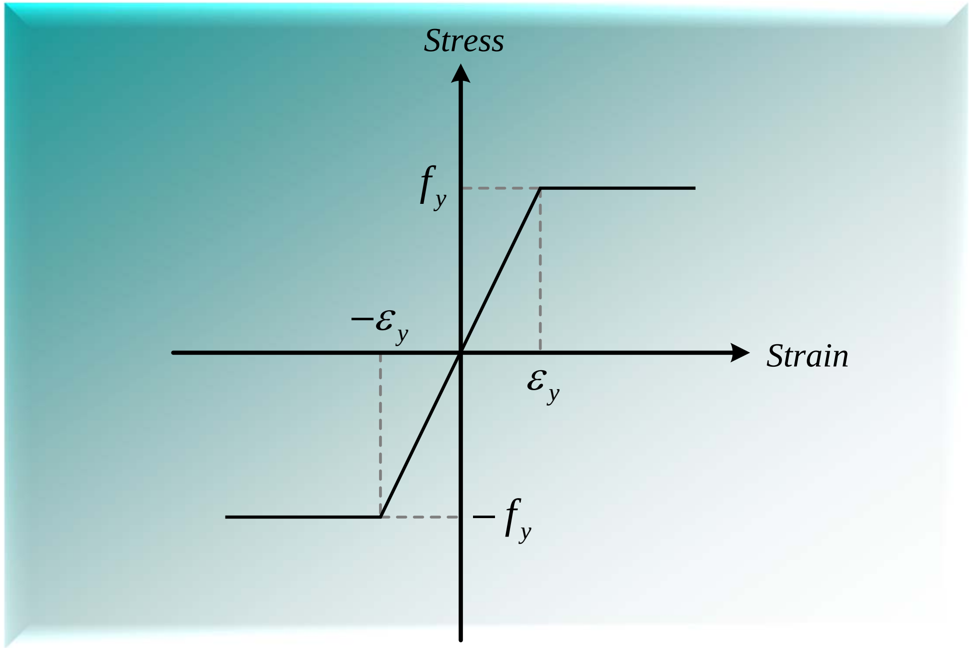
$$\varepsilon_r \le \varepsilon_{cr}$$

$$\sigma_r = E_c \varepsilon_r$$

$$\varepsilon_r > \varepsilon_{cr}$$

$$\sigma_r = f_{cr}\left(\frac{\varepsilon_{cr}}{\varepsilon_r}\right)^{0.4}$$

Constitutive Laws of Reinforcement



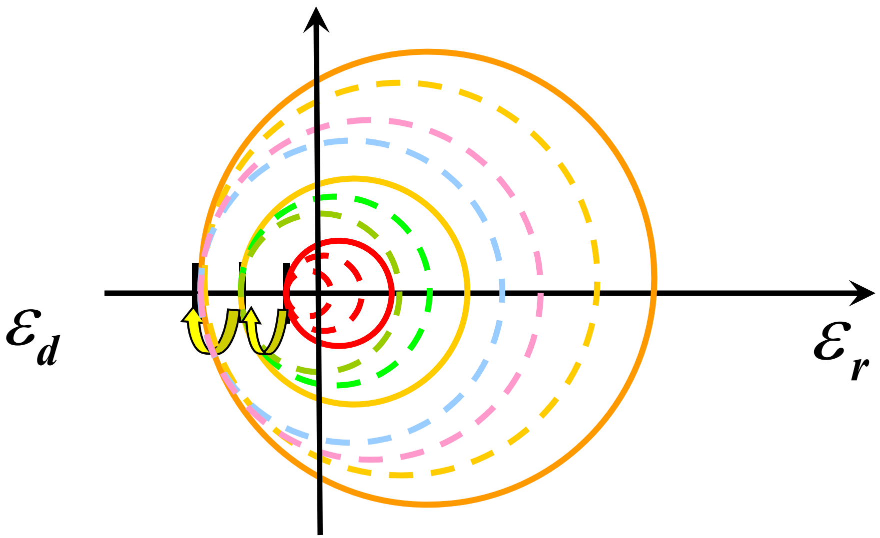
$$\varepsilon_s \le \varepsilon_y$$

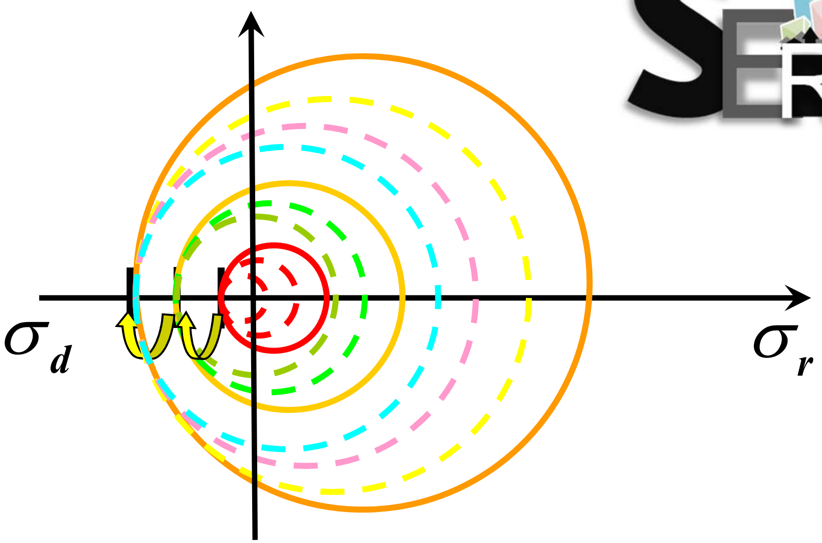
$$f_s = E_s \varepsilon_s$$

$$\varepsilon_s > \varepsilon_y$$

$$f_s = f_y$$

$$V_s = \rho_t \times b_w \times h \times f_t$$





$\varepsilon_l = \varepsilon_d \cos^2\theta + \varepsilon_r$ sin

$\varepsilon_t = \varepsilon_d \sin^2\theta + \varepsilon_r$ cos

$\gamma_{lt}/2 = (-\varepsilon_d + \varepsilon_r)\sin\theta$ c

$$V = V_c + V_s \; ?$$

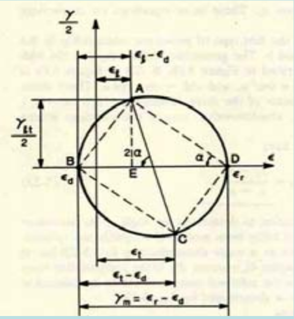
$$V_c = \tau_{ltc} \times b_w \times d$$

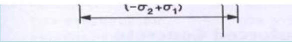
$$V_s = \rho_t \times b_w \times h \times f_t$$

s$^2\,\theta + \sigma_r \sin^2\theta$

n$^2\,\theta + \sigma_r \cos^2\theta$

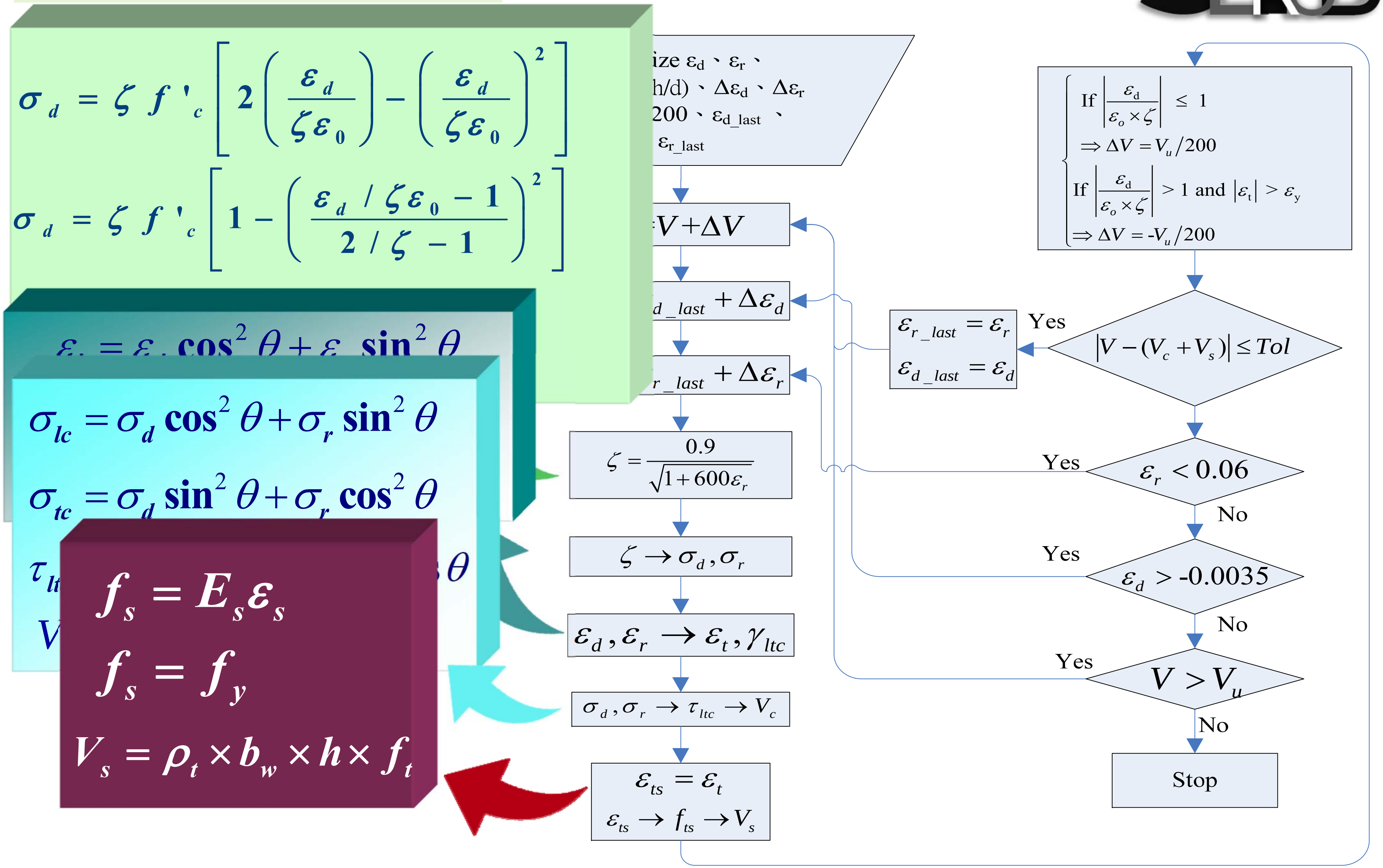
$+\sigma_r)\sin\theta\cos\theta$





Lai, M.C., **Sung, Y.C., "A Study on Pushover Analysis of Frame Structure Infilled with Low-rise Reinforced Concrete Wall"**, *Journal of Mechanics*, Vol. 24, 2008, p.p. 437-449

RC牆剪力分析流程



$$\sigma_d = \zeta f'_c \left[2\left(\frac{\varepsilon_d}{\zeta\varepsilon_0}\right) - \left(\frac{\varepsilon_d}{\zeta\varepsilon_0}\right)^2 \right]$$

$$\sigma_d = \zeta f'_c \left[1 - \left(\frac{\varepsilon_d/\zeta\varepsilon_0 - 1}{2/\zeta - 1}\right)^2 \right]$$

$$\varepsilon_l = \varepsilon_d \cos^2\theta + \varepsilon_r \sin^2\theta$$

$$\sigma_{lc} = \sigma_d \cos^2\theta + \sigma_r \sin^2\theta$$

$$\sigma_{tc} = \sigma_d \sin^2\theta + \sigma_r \cos^2\theta$$

τ_{lt} … θ

V

$$f_s = E_s\varepsilon_s$$

$$f_s = f_y$$

$$V_s = \rho_t \times b_w \times h \times f_t$$

ize ε_d、ε_r、
h/d)、$\Delta\varepsilon_d$、$\Delta\varepsilon_r$
200、ε_{d_last}、
ε_{r_last}

$V+\Delta V$

$_{d\_last}+\Delta\varepsilon_d$

$_{r\_last}+\Delta\varepsilon_r$

$$\zeta = \frac{0.9}{\sqrt{1+600\varepsilon_r}}$$

$$\zeta \rightarrow \sigma_d, \sigma_r$$

$$\varepsilon_d, \varepsilon_r \rightarrow \varepsilon_t, \gamma_{ltc}$$

$$\sigma_d, \sigma_r \rightarrow \tau_{ltc} \rightarrow V_c$$

$$\varepsilon_{ts} = \varepsilon_t$$
$$\varepsilon_{ts} \rightarrow f_{ts} \rightarrow V_s$$

$$\begin{cases} \text{If } \left|\dfrac{\varepsilon_d}{\varepsilon_o \times \zeta}\right| \le 1 \\ \Rightarrow \Delta V = V_u/200 \\ \text{If } \left|\dfrac{\varepsilon_d}{\varepsilon_o \times \zeta}\right| > 1 \text{ and } |\varepsilon_t| > \varepsilon_y \\ \Rightarrow \Delta V = -V_u/200 \end{cases}$$

$$|V - (V_c + V_s)| \le Tol$$

Yes

$$\varepsilon_{r\_last} = \varepsilon_r$$
$$\varepsilon_{d\_last} = \varepsilon_d$$

$$\varepsilon_r < 0.06$$

Yes

No

$$\varepsilon_d > -0.0035$$

Yes

No

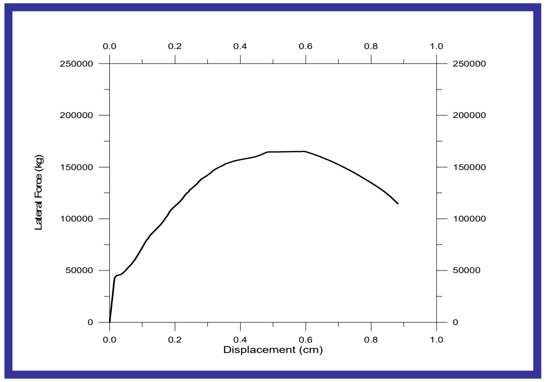
$$V > V_u$$

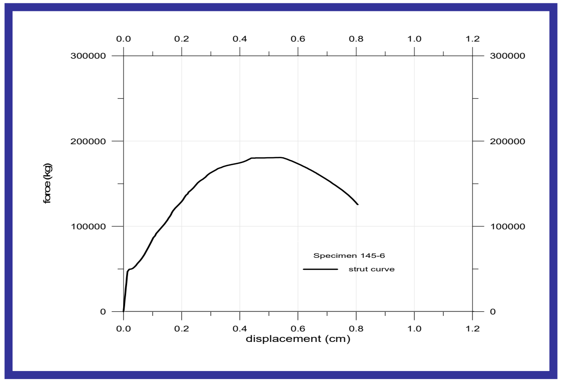
Yes

No

Stop

Aeff of equivalent diagonal structural strut



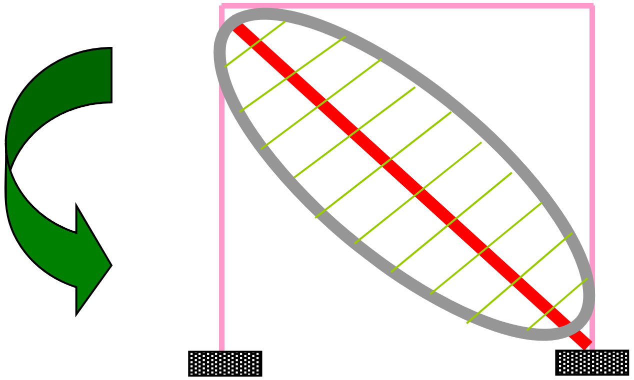


$$V = V_c + V_s$$

$$\delta = \gamma_{lt} \times h$$

$$V_{strut} = \frac{V}{\cos\theta}$$

$$\delta_{strut} = \delta \times \cos\theta$$



$$\Rightarrow A_{eff} = \frac{P}{\delta} \times \frac{L}{E_c}$$

$$\frac{E_c \times A_{eff}}{L} = \frac{P}{\delta}$$

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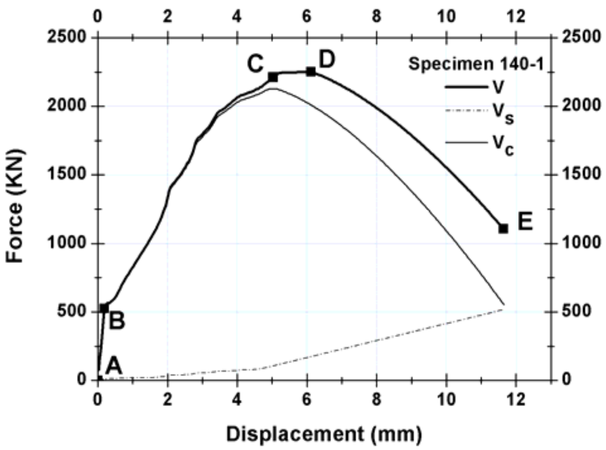
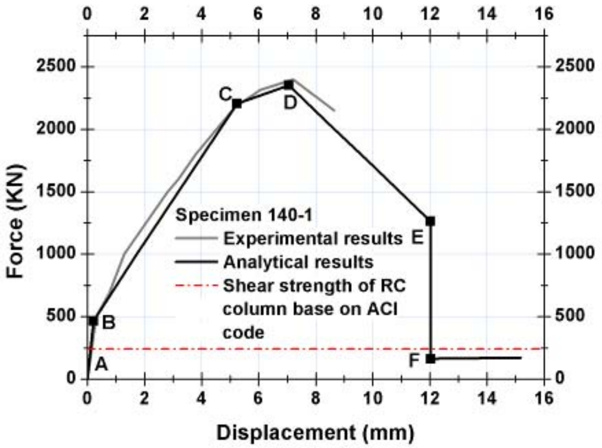
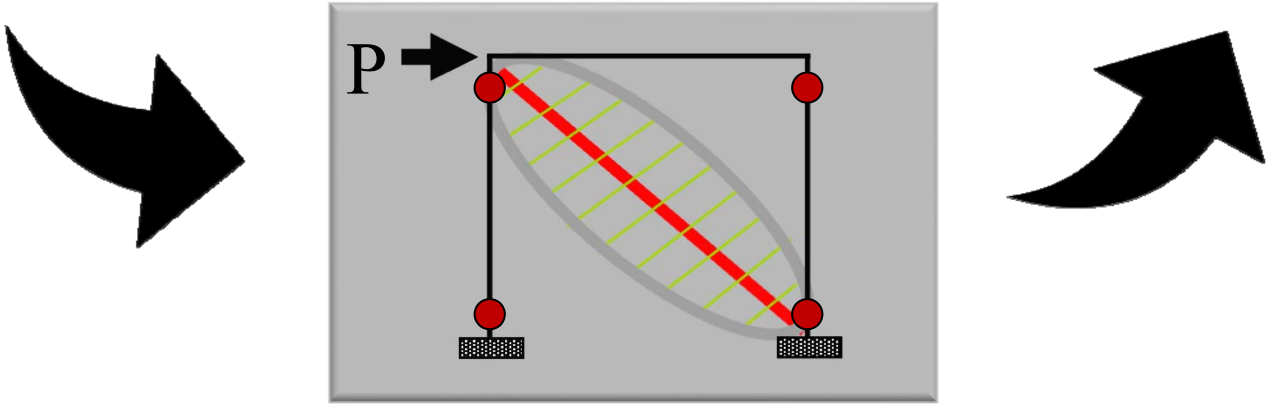
Pushover Analysis Results (specimen 140-1)





Shear strength of RC wall

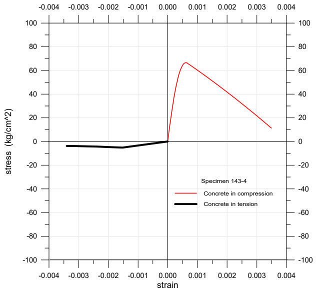
Pushover Analysis Results

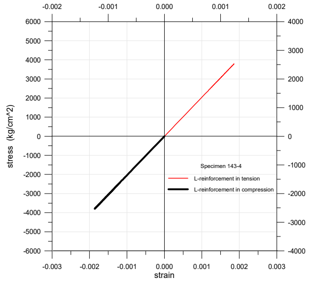


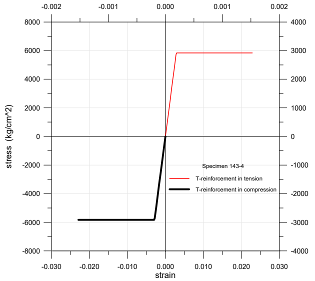
Lai, M.C., **Sung, Y.C.**, "**A Study on Pushover Analysis of Frame Structure Infilled with Low-rise Reinforced Concrete Wall**", *Journal of Mechanics,* Vol. 24, 2008, p.p. 437-449

分析方式

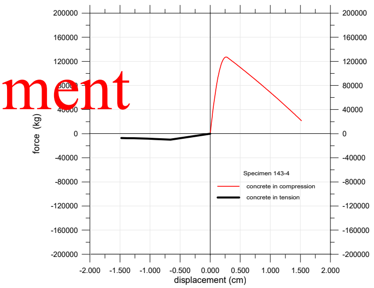
Stress-Strain

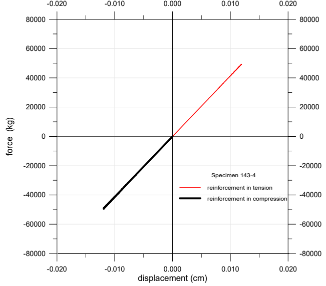


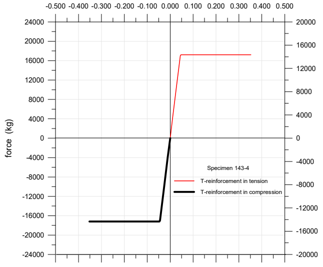




Force-Displacement





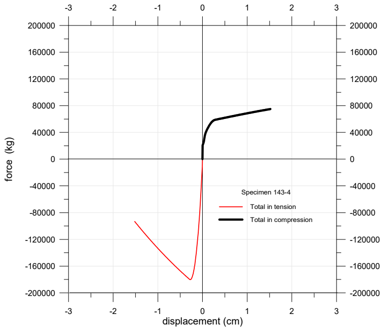


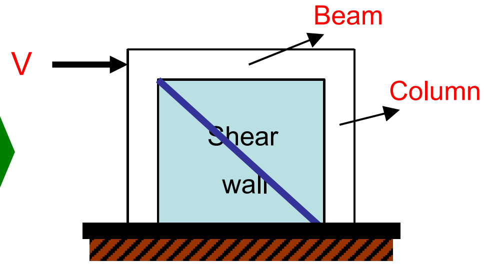
$\sum$

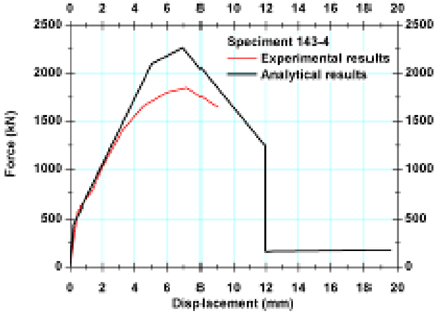
Concrete

Longitudinal reinforcements

Transverse reinforcements







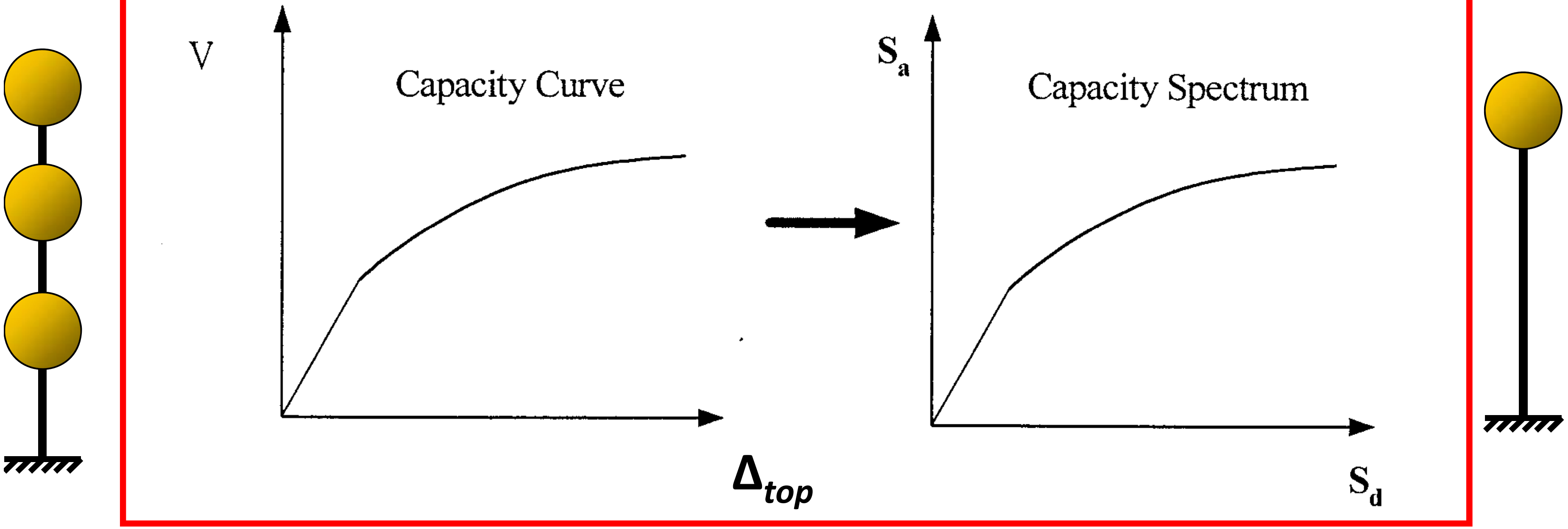
RC牆模擬-等值柱

考慮軸向、
剪力、撓曲

考慮軸向

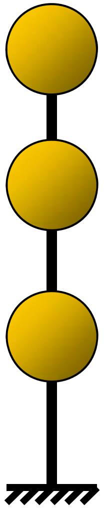
結構系統
改變

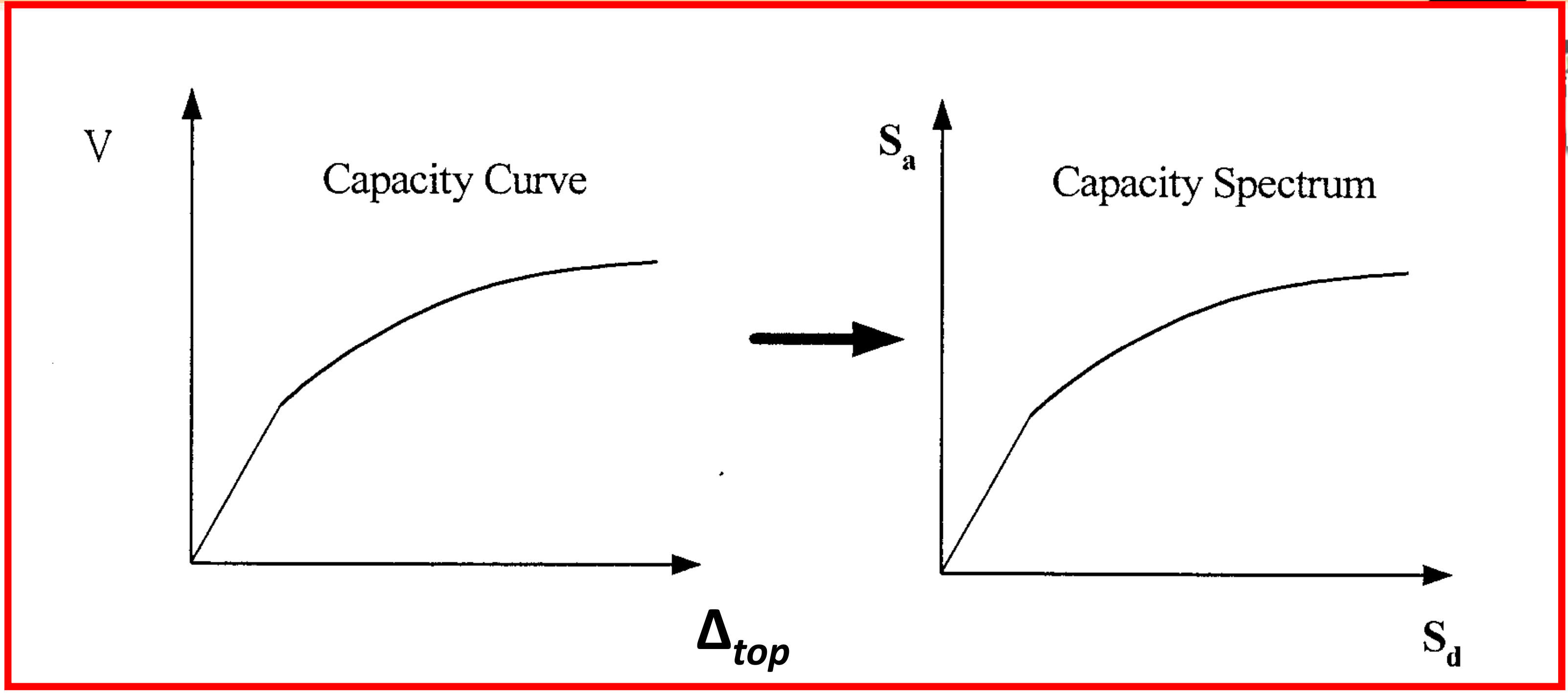
建立結構物的容量譜曲線(Capacity Spectrum)

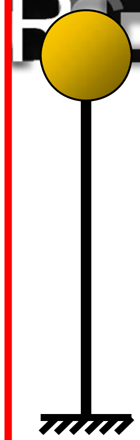


$$S_{ai} = \frac{V_i}{W\alpha_1}$$

$$\alpha_1 = \frac{\left[\sum_{i=1}^{N}(w_i\phi_{i1})/g\right]^2}{\left[\sum_{i=1}^{N} w_i/g\right]\left[\sum_{i=1}^{N}(w_i\phi_{i1}^2)/g\right]}$$







$$S_{di} = \frac{\Delta_{top,i}}{(PF_1 \times \phi_{1,top})}$$

$$PF_1 = \left[\frac{\sum_{i=1}^{N} (w_i \phi_{i1}) / g}{\sum_{i=1}^{N} (w_i \phi_{i1}^2) / g} \right]$$

α_1: 第一振態有效振態質量比

PF_1: 第一振態參與因子

$\phi_{1,top}$: 第一振態形狀在頂部之值

修正CSC檔

$$PF_1 = \left[\frac{\sum_{i=1}^{N} \frac{(W_i \varphi_{i1})}{g}}{\sum_{i=1}^{N} \frac{(W_i \varphi_{i1}^2)}{g}} \right] \qquad \alpha_1 = \frac{\left[\sum_{i=1}^{N} \frac{(W_i \varphi_{i1})}{g} \right]^2}{\left[\sum_{i=1}^{N} \frac{W_i}{g} \right] \left[\sum_{i=1}^{N} \frac{(W_i \varphi_{i1}^2)}{g} \right]}$$

$$S_d = \frac{\Delta_{roof}}{PF_1 \varphi_{roof,1}} \qquad S_a = \frac{V}{\alpha_1 W}$$

α_1 ：第一個自然振態的振態質量係數 **(Modal Mass Coefficient)**

PF_1 ：第一個自然振態的振態參與因 **(Modal Participation Factor)**

φ_{i1} ：第 i 層的樓層位移

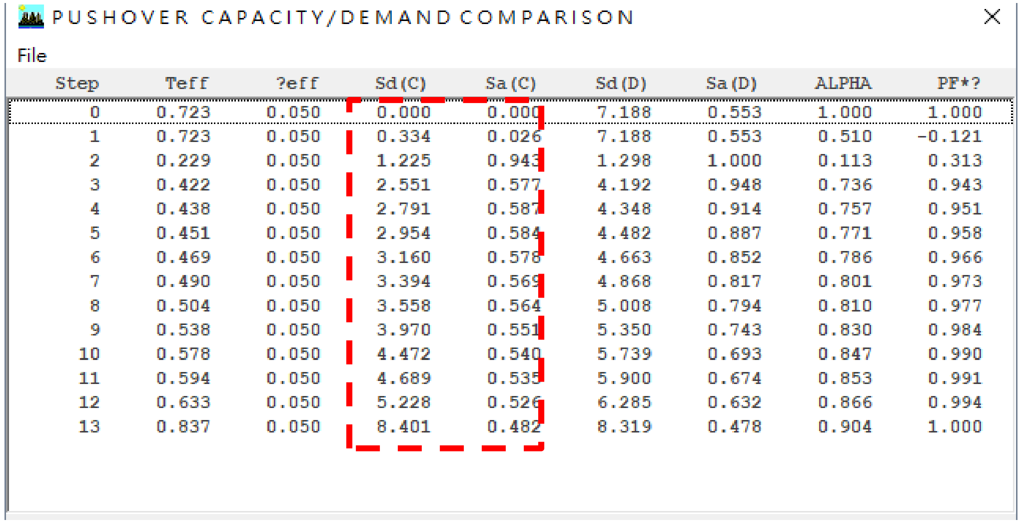
$\frac{W_i}{g}$ ：第 i 層的指定質量

W ：結構物的靜載重

N ：結構物的最高層數

V ：容量曲線的側力

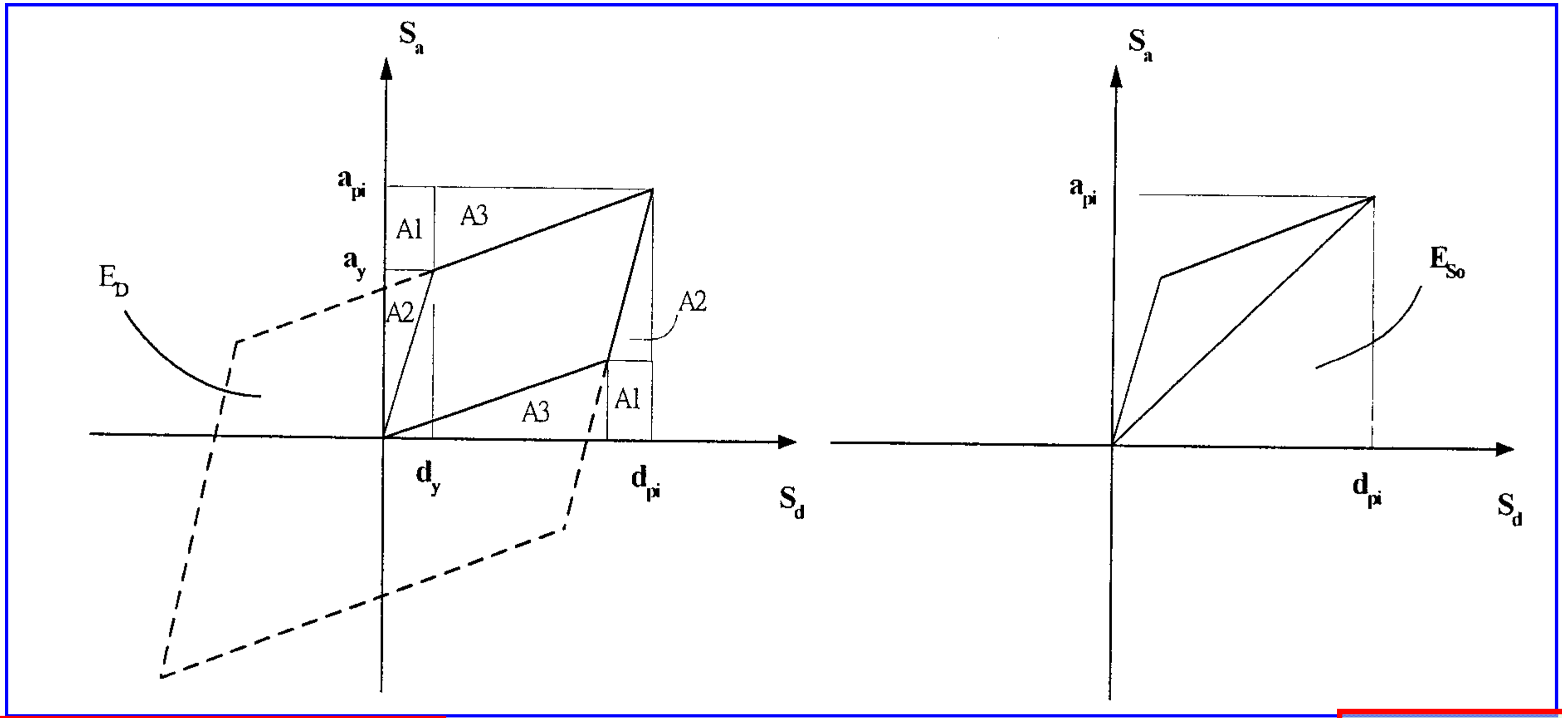
1.由模型內匯出計算PF$_1$ 、α$_1$使用參數
2.計算每一步Sa、Sd
3.將CSC檔中紅框位置替換成計算結果



PUSHOVER CAPACITY/DEMAND COMPARISON

Step	Teff	?eff	Sd(C)	Sa(C)	Sd(D)	Sa(D)	ALPHA	PF*?
0	0.723	0.050	0.000	0.000	7.188	0.553	1.000	1.000
1	0.723	0.050	0.334	0.026	7.188	0.553	0.510	-0.121
2	0.229	0.050	1.225	0.943	1.298	1.000	0.113	0.313
3	0.422	0.050	2.551	0.577	4.192	0.948	0.736	0.943
4	0.438	0.050	2.791	0.587	4.348	0.914	0.757	0.951
5	0.451	0.050	2.954	0.584	4.482	0.887	0.771	0.958
6	0.469	0.050	3.160	0.578	4.663	0.852	0.786	0.966
7	0.490	0.050	3.394	0.569	4.868	0.817	0.801	0.973
8	0.504	0.050	3.558	0.564	5.008	0.794	0.810	0.977
9	0.538	0.050	3.970	0.551	5.350	0.743	0.830	0.984
10	0.578	0.050	4.472	0.540	5.739	0.693	0.847	0.990
11	0.594	0.050	4.689	0.535	5.900	0.674	0.853	0.991
12	0.633	0.050	5.228	0.526	6.285	0.632	0.866	0.994
13	0.837	0.050	8.401	0.482	8.319	0.478	0.904	1.000

等值週期與等值遲滯阻尼比



$$S_{di} = \frac{T_i^2}{4\pi^2} S_{ai} g$$

$$E_D = 4(a_y d_{pi} - d_y a_{pi})$$

$$E_{So} = \frac{a_{pi} d_{pi}}{2}$$

$$\beta_{pi} = \frac{E_D}{4\pi E_{So}} = \frac{0.637(a_y d_{pi} - d_y a_{pi})}{a_{pi} d_{pi}}$$

阻尼比異於5%之修正係數

有效阻尼比 $\xi(\%)$	B_S	B_1
<2	0.80	0.80
5	1.00	1.00
10	1.33	1.25
20	1.60	1.50
30	1.79	1.63
40	1.87	1.70
>50	1.93	1.75

求性能點之EPA

$$\because a_{pi} = ZS_a(T_i，\beta_{pi}+0.05)$$

$$\therefore Z = \frac{a_{pi}}{S_a(T_i，\beta_{pi}+0.05)}$$

$$S_a(T_i，\beta_{pi}+0.05) = S_{aD}(T_i，\beta_{pi}+0.05)/0.4S_{DS}$$

一般工址

$$S_{aD}^{n} = \begin{cases} \dfrac{2.5}{B_S} & ; \quad 0 \le T \le T_0 \\[2ex] \dfrac{2.5S_{D1}}{B_1 S_{DS} T} & ; \quad T_0 \le T \end{cases}$$

台北盆地

$$S_{aD}^{n} = \begin{cases} \dfrac{2.5}{B_s} & ; \quad 0 \le T \le T_0 \\[2ex] \dfrac{2.5T_0^{D}}{B_1 T} & ; \quad T > T_0 \end{cases}$$

方法1 由等值週期與阻尼比求性能點之EPA

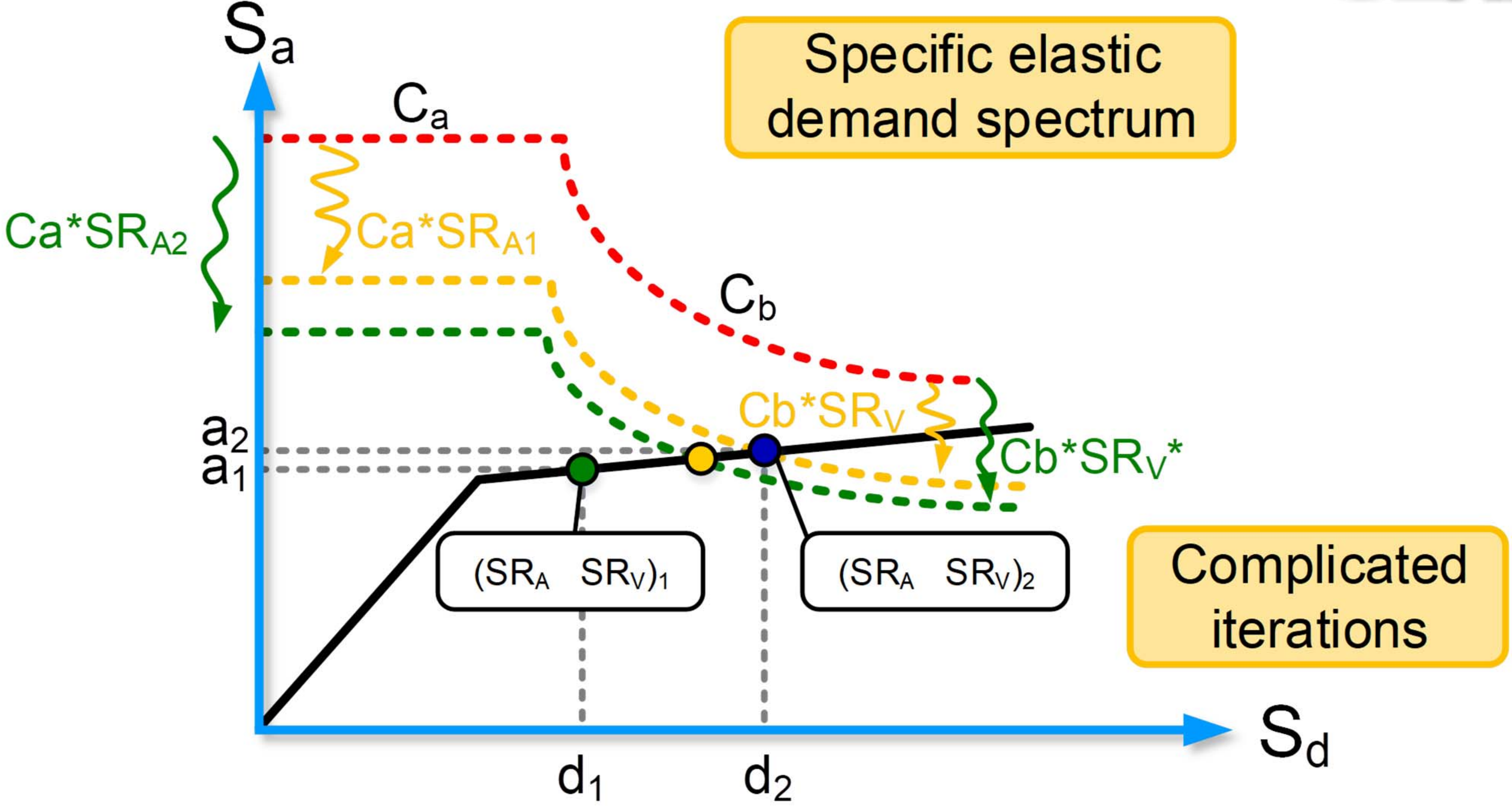
一般工址　有效最大加速度(EPA)

較短及短週期	中週期	長週期
$T \le T_0^D \dfrac{B_s(\beta_{eff})}{B_1(\beta_{eff})}$	$T_0^D \dfrac{B_s(\beta_{eff})}{B_1(\beta_{eff})} < T \le 2.5T_0^D \dfrac{B_s(\beta_{eff})}{B_1(\beta_{eff})}$	$2.5T_0^D \dfrac{B_s(\beta_{eff})}{B_1(\beta_{eff})} < T$
$EPA = \dfrac{a_{pi} \times B_s(\beta_{eff})}{(\dfrac{2.5S_{aD}(T)}{S_{DS}})} = \dfrac{a_{pi} \times B_s(\beta_{eff})}{2.5}$	$EPA = \dfrac{a_{pi} \times B_1(\beta_{eff})}{(\dfrac{2.5S_{aD}(T)}{S_{DS}})}$	$EPA = \dfrac{a_{pi} \times B_1(\beta_{eff})}{(\dfrac{2.5S_{D1}(T)}{S_{DS}T})}$

方法1 由等值週期與阻尼比求性能點之EPA

台北盆地　有效最大加速度(EPA)

較短及短週期	中週期	長週期
$T \le T_0^D \dfrac{B_s(\beta_{eff})}{B_1(\beta_{eff})}$	$T_0^D \dfrac{B_s(\beta_{eff})}{B_1(\beta_{eff})} < T \le 2.5T_0^D \dfrac{B_s(\beta_{eff})}{B_1(\beta_{eff})}$	$2.5T_0^D \dfrac{B_s(\beta_{eff})}{B_1(\beta_{eff})} < T$
$EPA = \dfrac{a_{pi} \times B_s(\beta_{eff})}{(\frac{2.5S_{aD}(T)}{S_{DS}})} = \dfrac{a_{pi} \times B_s(\beta_{eff})}{2.5}$	$EPA = \dfrac{a_{pi} \times B_1(\beta_{eff})}{(\frac{2.5S_{aD}(T)}{S_{DS}})}$	$EPA = \dfrac{a_{pi} \times B_1(\beta_{eff})}{(\frac{2.5T_0^D}{T})}$

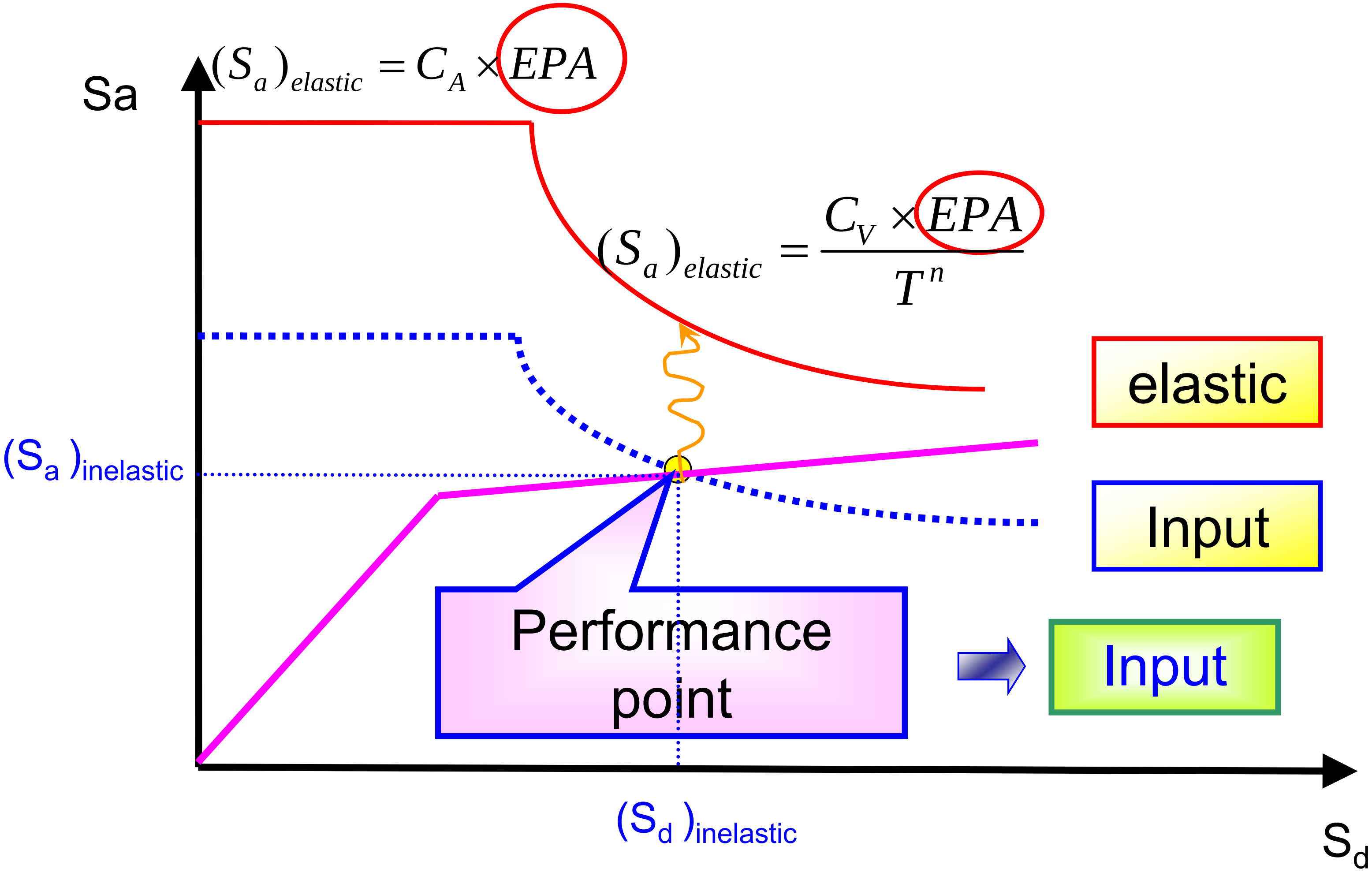


Specific elastic demand spectrum

Complicated iterations

ATC40由EPA求性能點

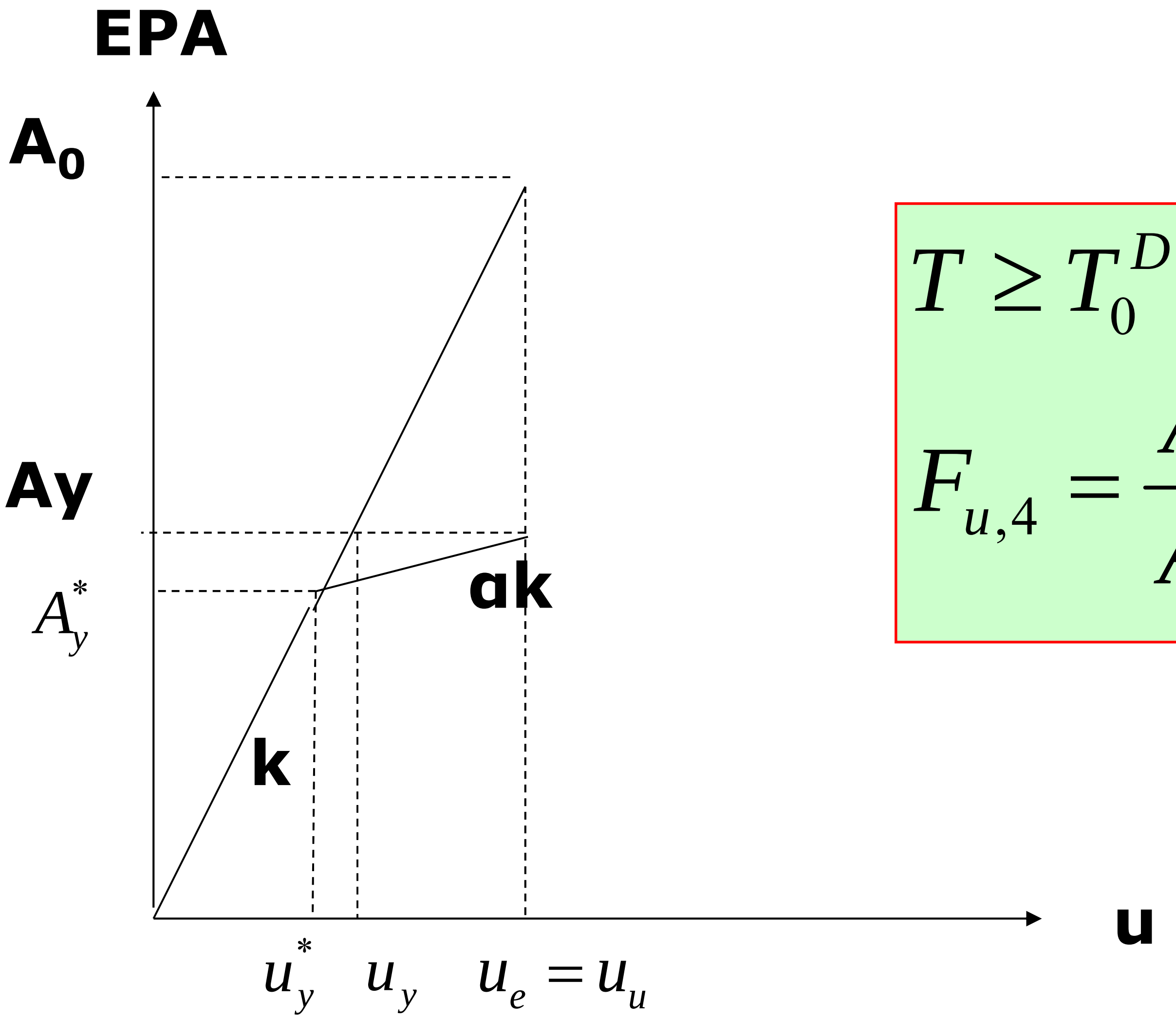
Find EPA which every performance is corresponded ⇒ output



$$(S_a)_{elastic} = C_A \times EPA$$

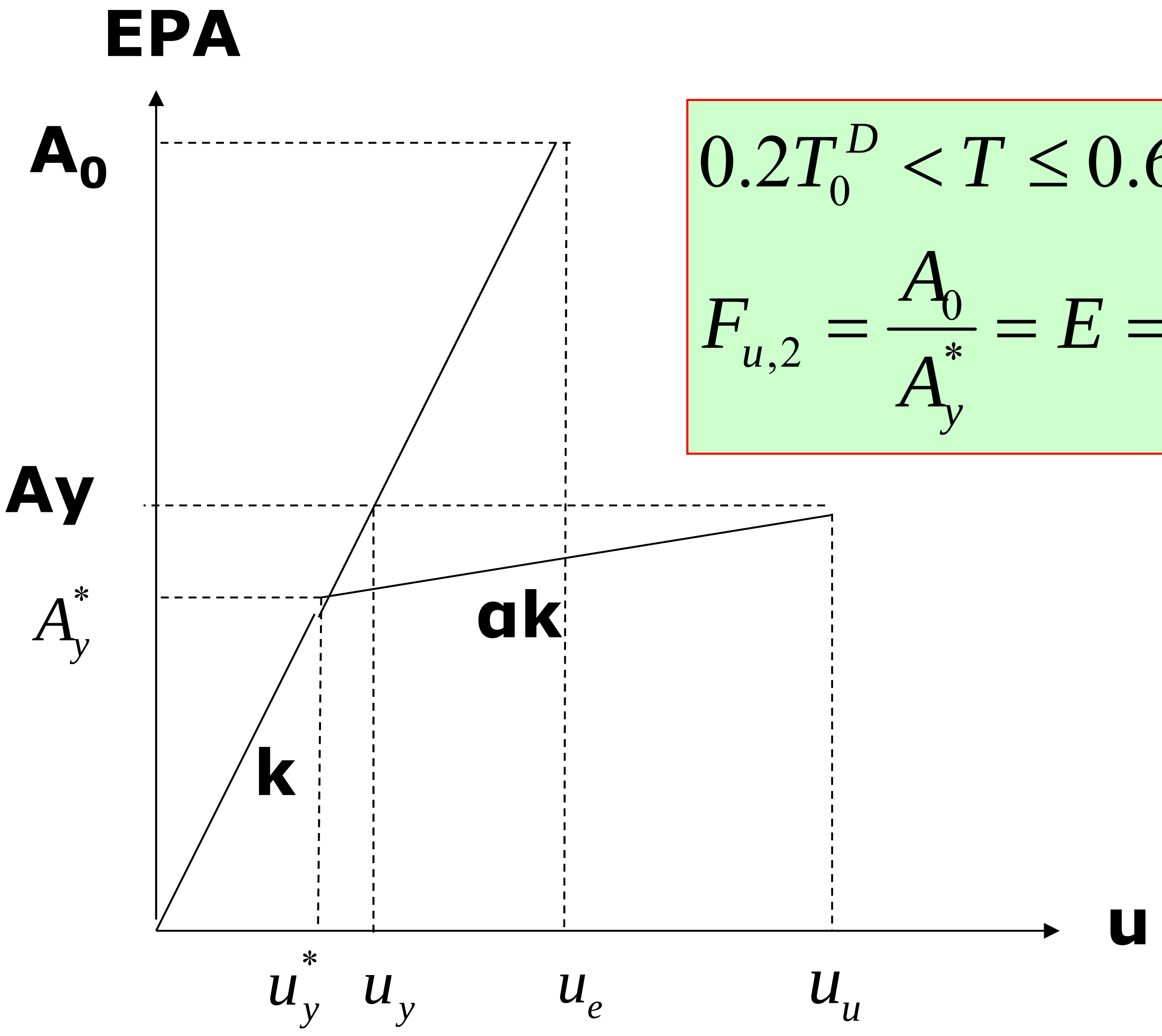
$$(S_a)_{elastic} = \frac{C_V \times EPA}{T^n}$$

方法2 由Fu-R-T法求性能點之EPA



$$T \geq T_0^D$$

$$F_{u,4} = \frac{A_0}{A_y^*} = \frac{u_u}{u_y^*} = R_i$$

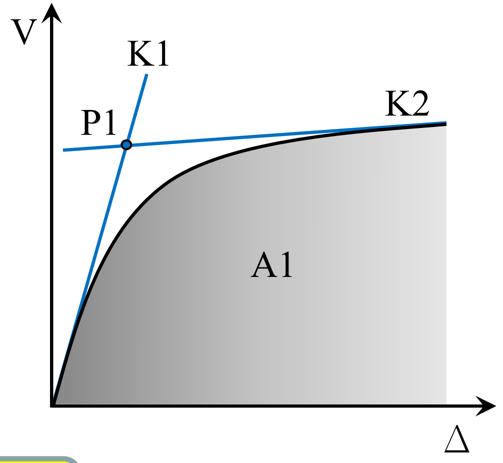


$$0.2T_0^D < T \le 0.6T_0^D$$

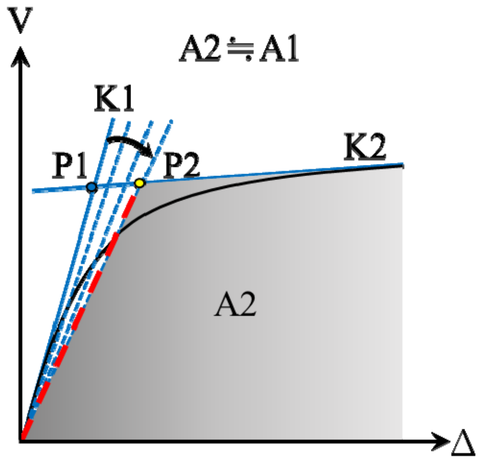
$$F_{u,2} = \frac{A_0}{A_y^*} = E = \sqrt{2R_i - 1 + \alpha(R_i - 1)}$$

容量曲線降伏點選取辦法

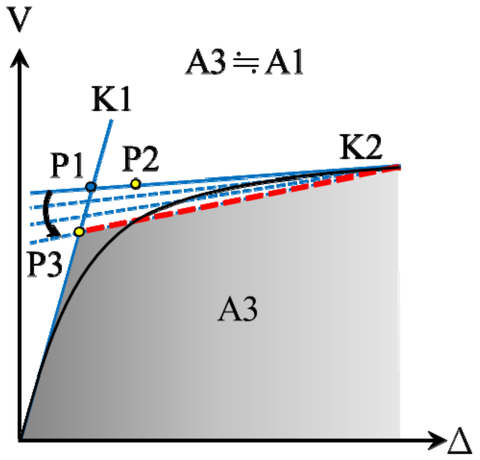
Step 1



Step 2

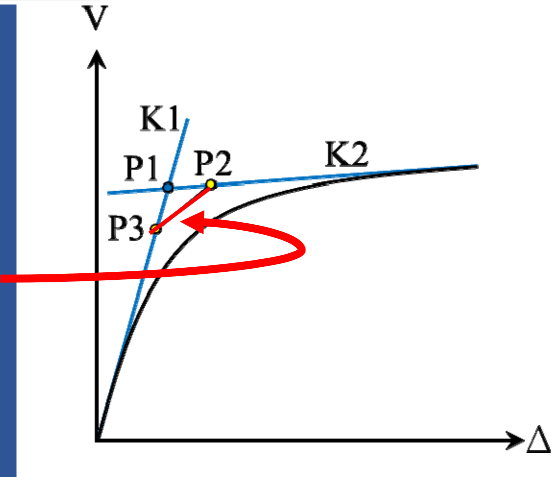


Step 3



Step 4

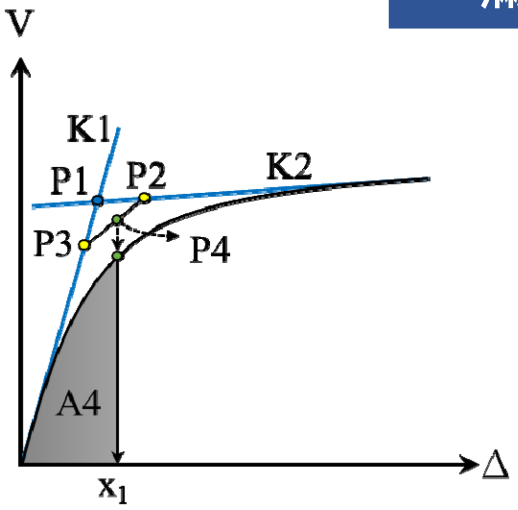
滿足等極限能量



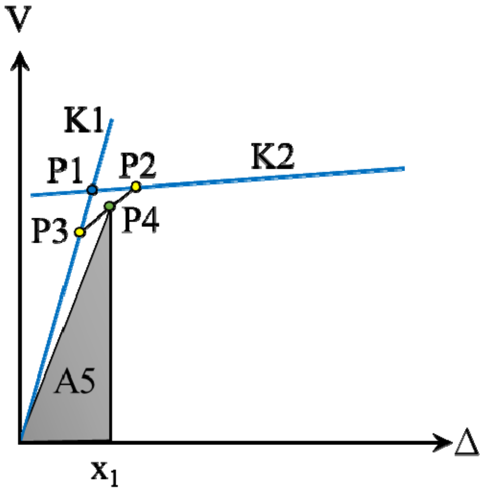
容量曲線降伏點選取辦法

Step 5

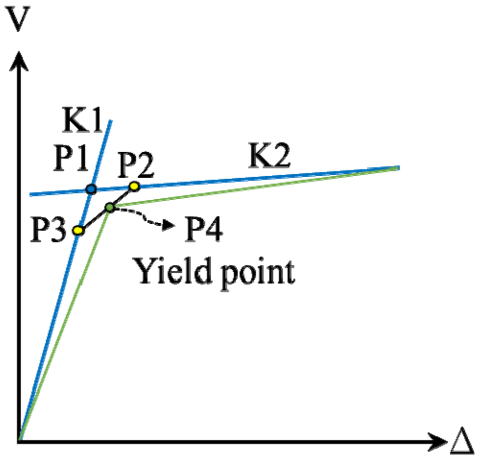
滿足等降伏能量



A5 ≒A4

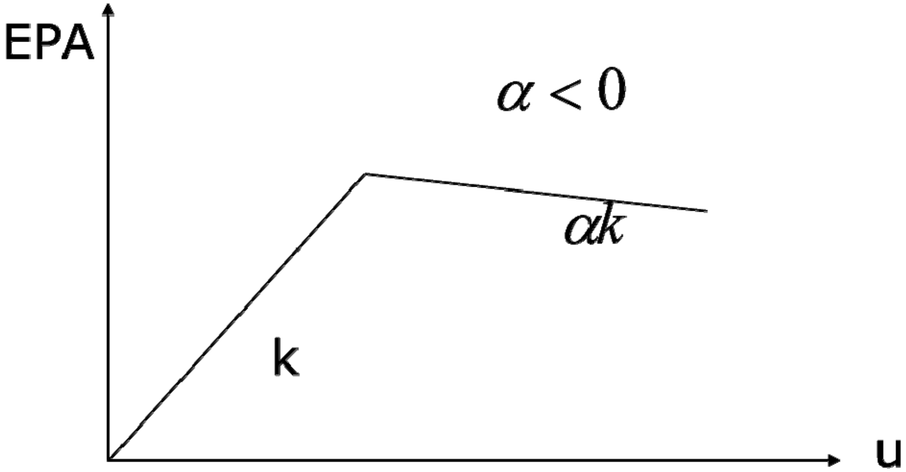


Step 6



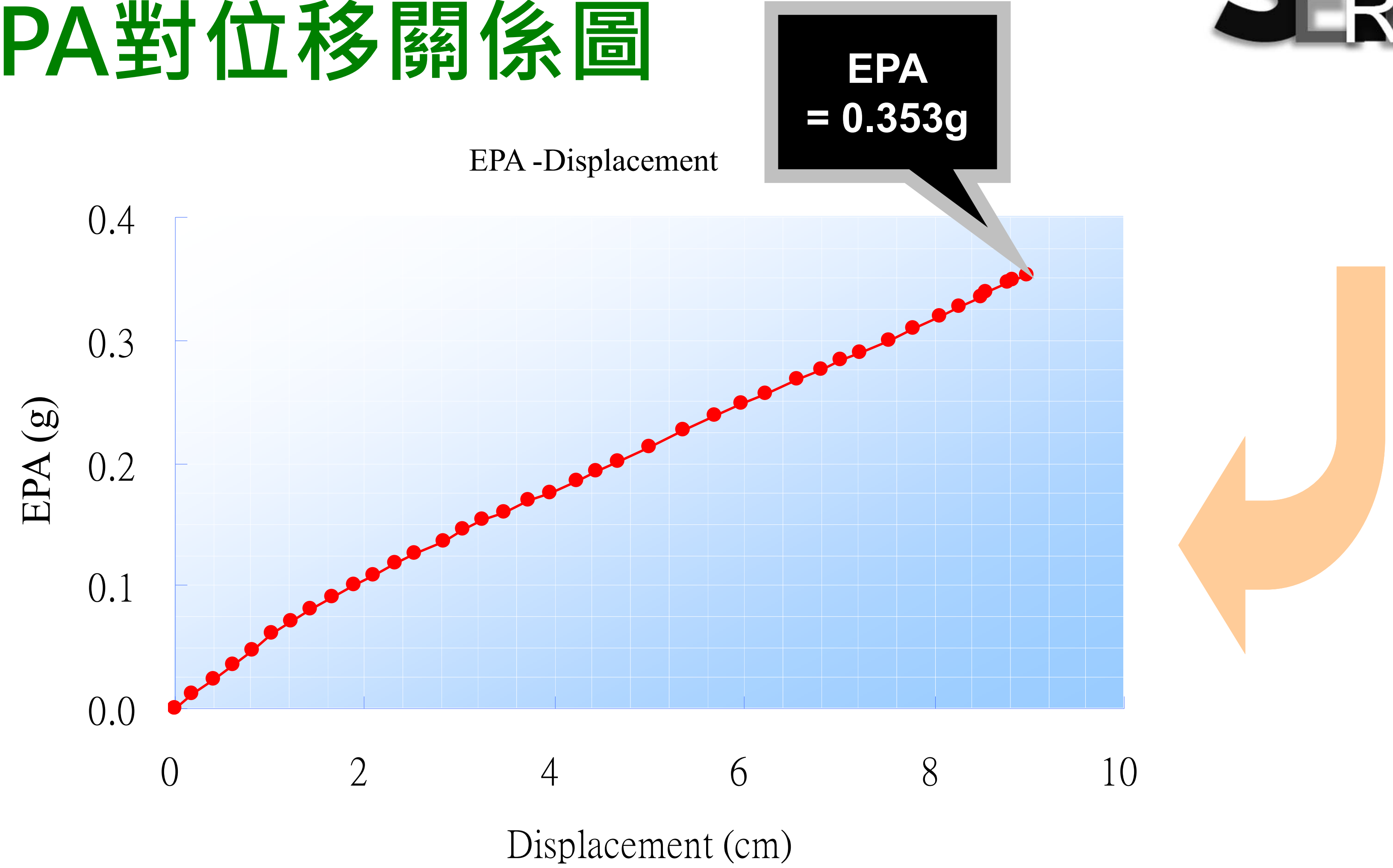
$$F_u(R_i) = \left\{ \begin{array}{lll} R_i & ; & T \geq T_0^D \\ E + (R_i - 1) \times \dfrac{T - 0.6T_0^D}{0.4T_0^D} & ; & 0.6T_0^D < T \leq T_0^D \\ E & ; & 0.2T_0^D < T \leq 0.6T_0^D \\ E + (E - 1) \times \dfrac{T - 0.2T_0^D}{0.2T_0^D} & ; & T < 0.2T_0^D \end{array} \right\}$$

$$E = \sqrt{2R_i - 1 + \alpha(R_i - 1)}$$



$$
EPA_i = \left\{ \begin{array}{lll}
\dfrac{S_{ay} \times F_{u,1}(R_i)}{\left[\dfrac{S_{aD}(T)}{0.4S_{DS}}\right]} = \dfrac{S_{ay} \times F_{u,1}(R_i)}{\left[1 + \dfrac{3T}{0.4T_0^D}\right]} & ; & T \le 0.2T_0^D \\
\dfrac{S_{ay} \times F_{u,2}(R_i)}{\dfrac{S_{aD}(T)}{0.4S_{DS}}} = \dfrac{S_{ay} \times F_{u,2}(R_i)}{2.5} & ; & 0.2T_0^D \le T \le 0.6T_0^D \\
\dfrac{S_{ay} \times F_{u,3}(R_i)}{\dfrac{S_{aD}(T)}{0.4S_{DS}}} = \dfrac{S_{ay} \times F_{u,3}(R_i)}{2.5} & ; & 0.6T_0^D \le T \le T_0^D \\
\dfrac{S_{ay} \times F_{u,4}(R_i)}{\dfrac{S_{aD}(T)}{0.4S_{DS}}} = \dfrac{S_{ay} \times F_{u,4}(R_i)}{\dfrac{S_{D1}}{0.4S_{DS}T}} & ; & T > T_0^D
\end{array} \right\}
$$

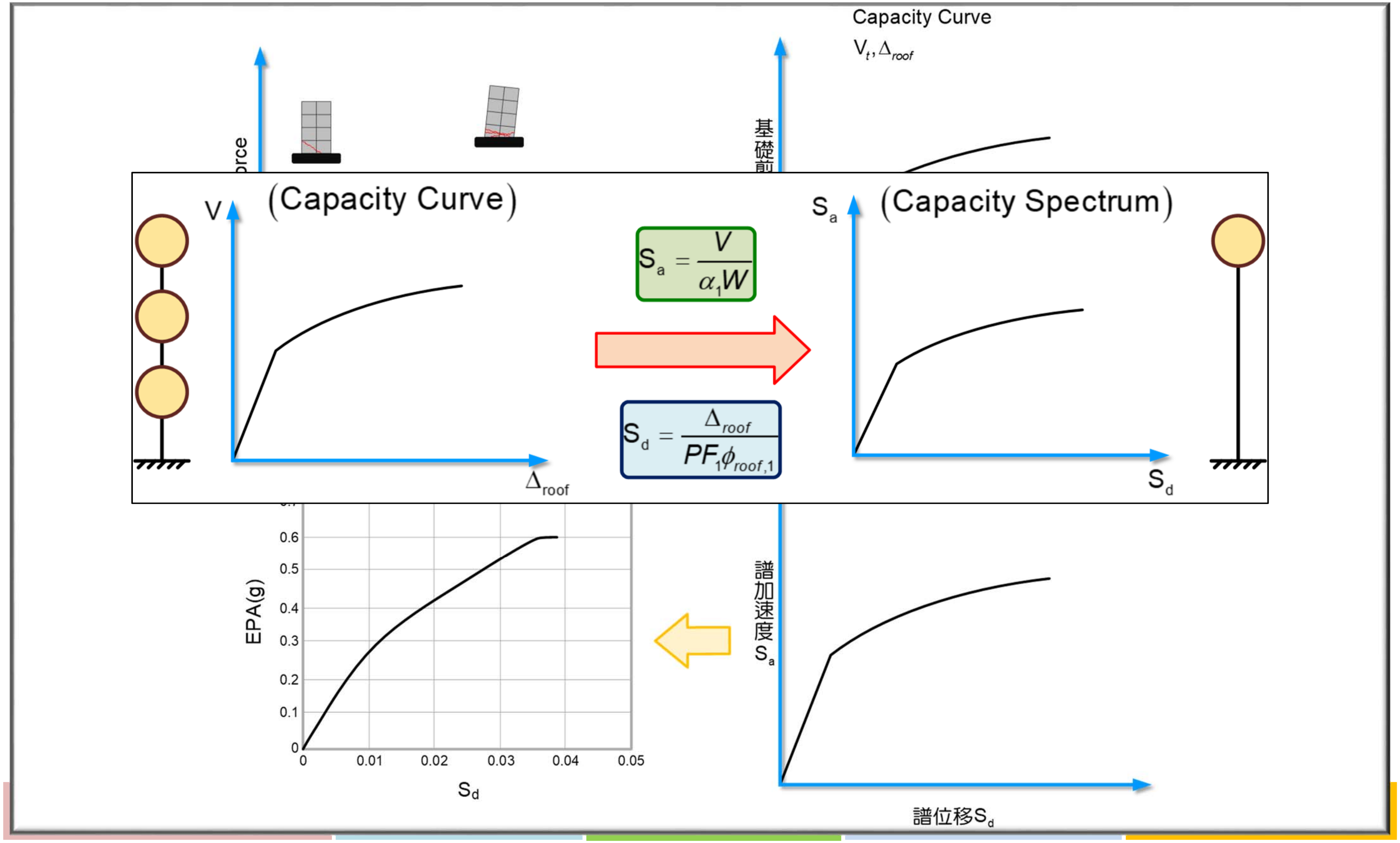
EPA對位移關係圖



Yu-Chi Sung, Chin-Kuo Su, Chuan-Wei Wu, and I-Chau Tsai, **"Performance-Based Damage Assessment of Low-Rise Reinforced Concrete Buildings"**, submitted to the Journal of China Institute of Engineer. **【SCI、EI】**

RC建築物耐震能力評估方法

建築物耐震能力評估流程



建築物之耐震性能檢核標準

性能狀態	一般工址			臺北盆地		
	I=1.0	I=1.25	I=1.5	I=1.0	I=1.25	I=1.5
PL_B	1/2	-	-	1/3	-	-
PL_C	1	-	-	2/3	-	-
PL_B*	-	5/12	-	-	7/24	-
PL_C*	-	5/6	-	-	7/12	-
PL_B**	-	-	1/3	-	-	1/4
PL_C**	-	-	2/3	-	-	2/4

註：$r=\dfrac{S_d-S_{dy}}{S_{du}-S_{dy}}$ S_d ：非彈性譜位移 S_{dy} ：降伏譜位移 S_{du} ：極限譜位移

建築物非彈性譜位移與非彈性譜位移容量之比值

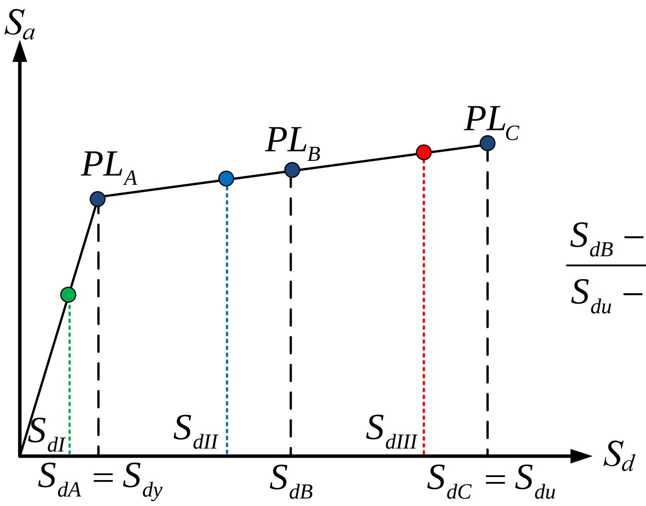
建築物之耐震性能檢核標準

一般工址之建築物耐震性能檢核

$$S_{dI} \le S_{dA}$$

$$S_{dII} \le S_{dB}$$

$$S_{dIII} \le S_{dC}$$



(a) I=1.0

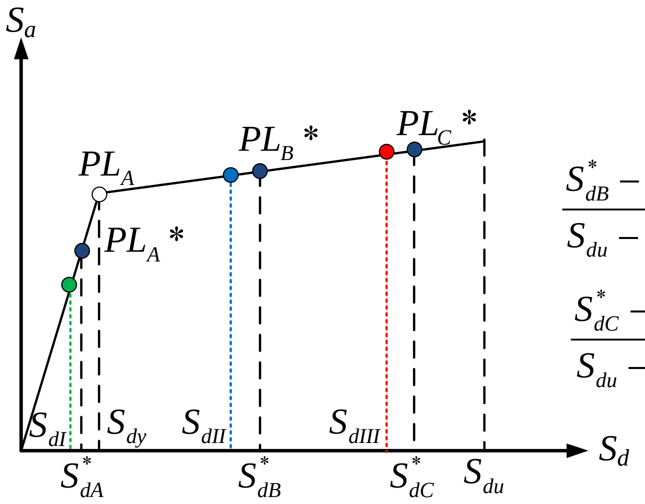
$$\frac{S_{dB}-S_{dy}}{S_{du}-S_{dy}} = \frac{1}{2}$$

$$S_{dI} \le S^*_{dA}$$

$$S_{dII} \le S^*_{dB}$$

$$S_{dIII} \le S^*_{dC}$$

註：$S^*_{dA} = \dfrac{S_{dy}}{1.1}$



(b) I=1.25

$$\frac{S^*_{dB}-S_{dy}}{S_{du}-S_{dy}} = \frac{5}{12}$$

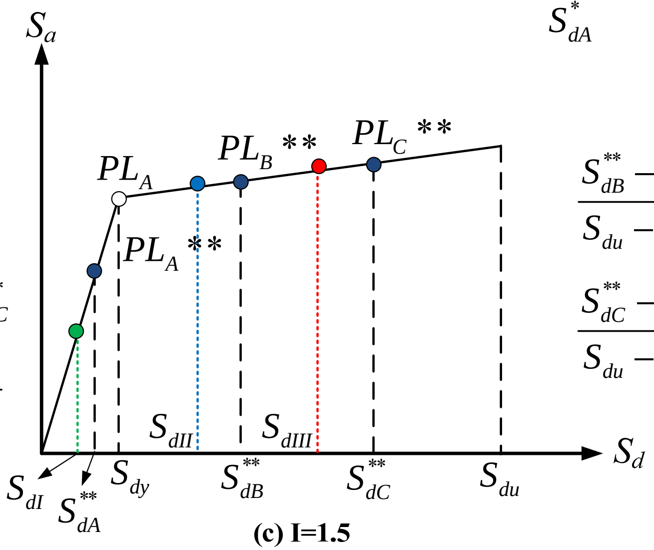
$$\frac{S^*_{dC}-S_{dy}}{S_{du}-S_{dy}} = \frac{5}{6}$$

$$S_{dI} \le S^{**}_{dA}$$

$$S_{dII} \le S^{**}_{dB}$$

$$S_{dIII} \le S^{**}_{dC}$$

註：$S^{**}_{dA} = \dfrac{S_{dy}}{1.2}$



(c) I=1.5

$$\frac{S^{**}_{dB}-S_{dy}}{S_{du}-S_{dy}} = \frac{1}{3}$$

$$\frac{S^{**}_{dC}-S_{dy}}{S_{du}-S_{dy}} = \frac{2}{3}$$

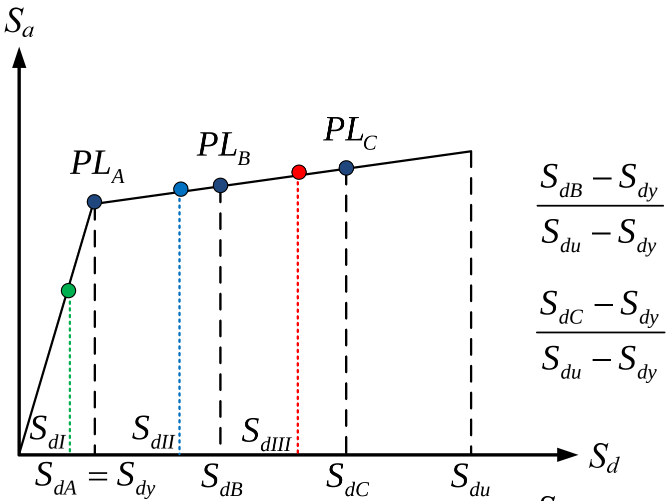
建築物之耐震性能檢核標準

臺北盆地之建築物耐震性能檢核

$S_{dI} \le S_{dA}$

$S_{dII} \le S_{dB}$

$S_{dIII} \le S_{dC}$



(a) I=1.0

$$\frac{S_{dB}-S_{dy}}{S_{du}-S_{dy}}=\frac{1}{3}$$

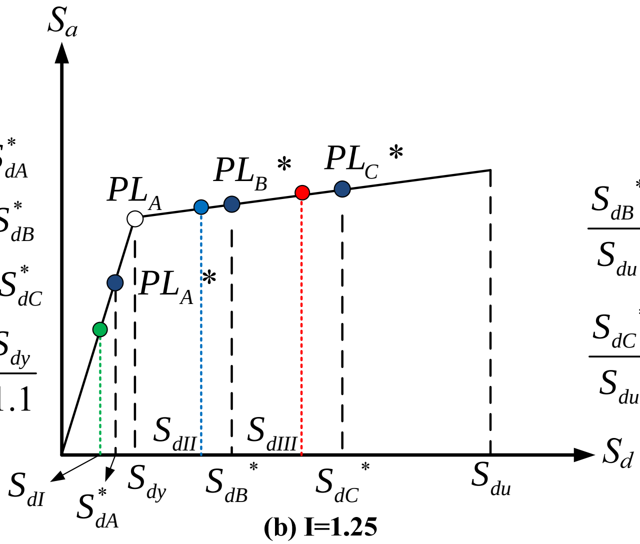
$$\frac{S_{dC}-S_{dy}}{S_{du}-S_{dy}}=\frac{2}{3}$$

$S_{dI} \le S^{*}_{dA}$

$S_{dII} \le S^{*}_{dB}$

$S_{dIII} \le S^{*}_{dC}$

註：$S^{*}_{dA}=\dfrac{S_{dy}}{1.1}$



(b) I=1.25

$$\frac{S_{dB}{}^{*}-S_{dy}}{S_{du}-S_{dy}}=\frac{7}{24}$$

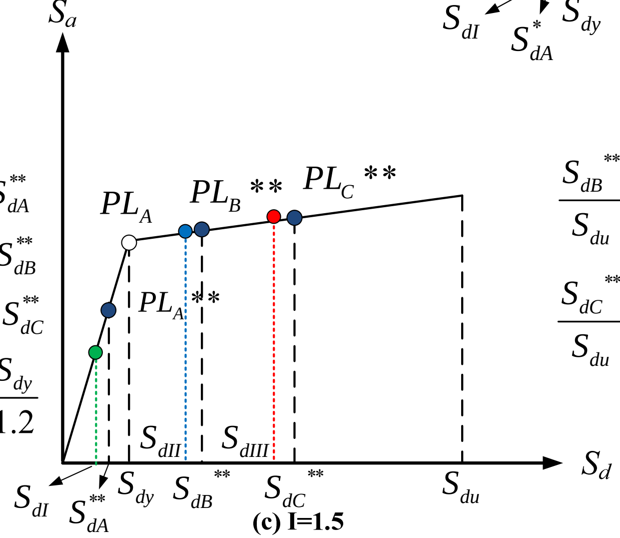
$$\frac{S_{dC}{}^{*}-S_{dy}}{S_{du}-S_{dy}}=\frac{7}{12}$$

$S_{dI} \le S^{**}_{dA}$

$S_{dII} \le S^{**}_{dB}$

$S_{dIII} \le S^{**}_{dC}$

註：$S^{**}_{dA}=\dfrac{S_{dy}}{1.2}$



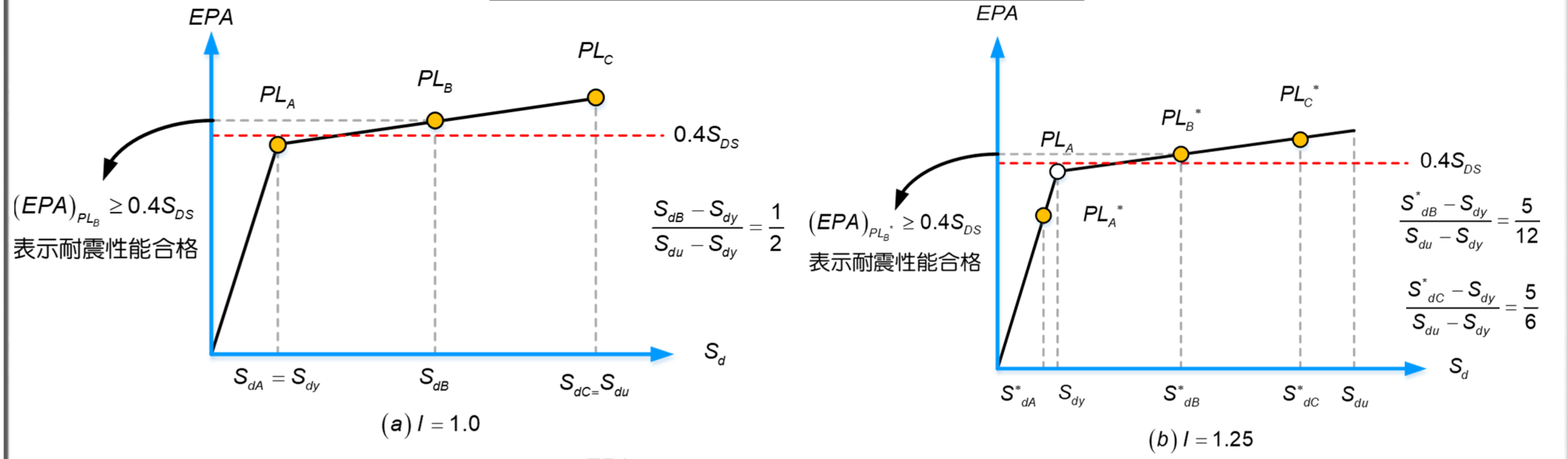
(c) I=1.5

$$\frac{S_{dB}{}^{**}-S_{dy}}{S_{du}-S_{dy}}=\frac{1}{4}$$

$$\frac{S_{dC}{}^{**}-S_{dy}}{S_{du}-S_{dy}}=\frac{1}{2}$$

建築物之耐震性能檢核標準

一般工址之建築物耐震性能狀態



$(EPA)_{PL_B} \geq 0.4S_{DS}$

表示耐震性能合格

$$\frac{S_{dB}-S_{dy}}{S_{du}-S_{dy}}=\frac{1}{2}$$

$(a)\ I=1.0$

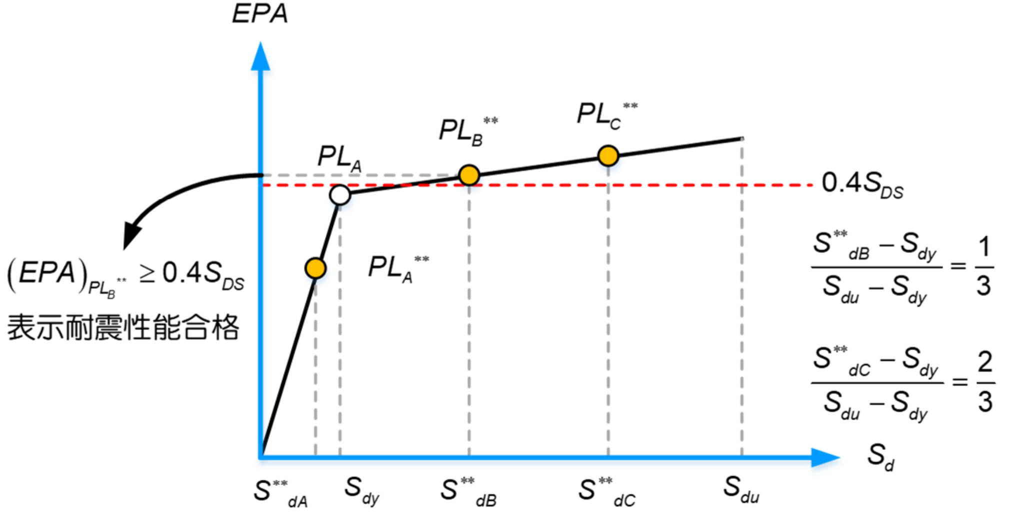
$(EPA)_{PL_B^*} \geq 0.4S_{DS}$

表示耐震性能合格

$$\frac{S^*_{dB}-S_{dy}}{S_{du}-S_{dy}}=\frac{5}{12}$$

$$\frac{S^*_{dC}-S_{dy}}{S_{du}-S_{dy}}=\frac{5}{6}$$

$(b)\ I=1.25$



$(EPA)_{PL_B^{**}} \geq 0.4S_{DS}$

表示耐震性能合格

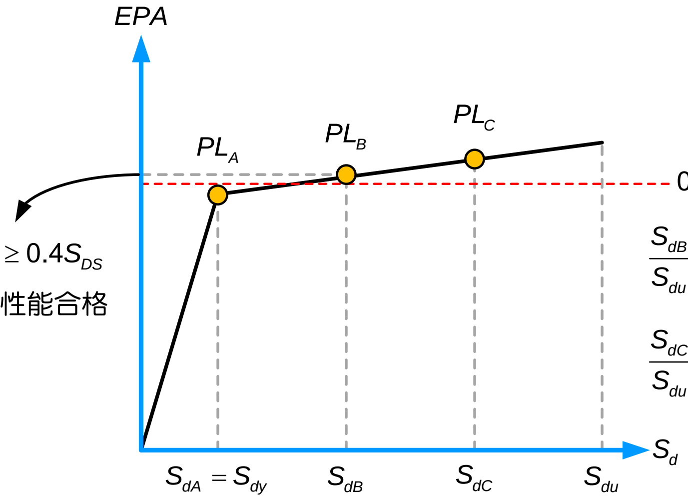
$$\frac{S^{**}_{dB}-S_{dy}}{S_{du}-S_{dy}}=\frac{1}{3}$$

$$\frac{S^{**}_{dC}-S_{dy}}{S_{du}-S_{dy}}=\frac{2}{3}$$

$(c)\ I=1.5$

建築物之耐震性能檢核標準

臺北盆地之建築物耐震性能檢核



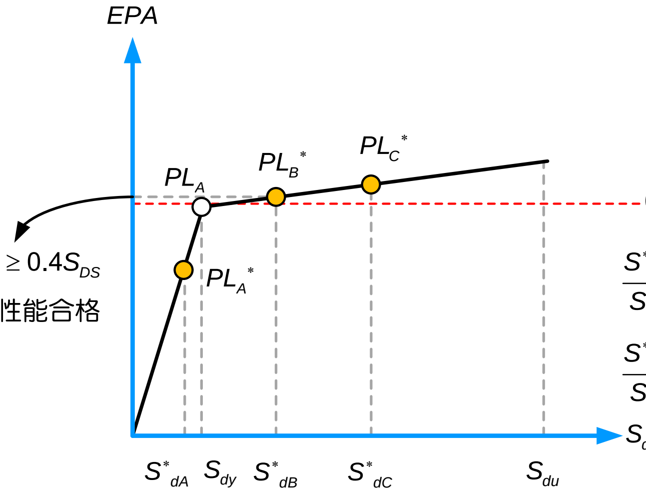
$(EPA)_{PL_B} \geq 0.4S_{DS}$

表示耐震性能合格

$$\frac{S_{dB} - S_{dy}}{S_{du} - S_{dy}} = \frac{1}{3}$$

$$\frac{S_{dC} - S_{dy}}{S_{du} - S_{dy}} = \frac{2}{3}$$

$(a)\ I = 1.0$



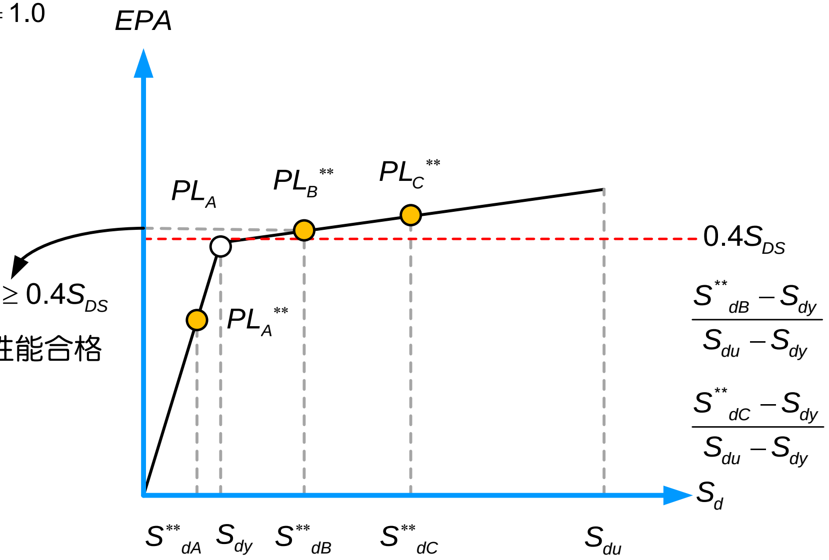
$(EPA)_{PL_B^*} \geq 0.4S_{DS}$

表示耐震性能合格

$$\frac{S^*{}_{dB} - S_{dy}}{S_{du} - S_{dy}} = \frac{7}{24}$$

$$\frac{S^*{}_{dC} - S_{dy}}{S_{du} - S_{dy}} = \frac{7}{12}$$

$(b)\ I = 1.25$



$(EPA)_{PL_B^{**}} \geq 0.4S_{DS}$

表示耐震性能合格

$$\frac{S^{**}{}_{dB} - S_{dy}}{S_{du} - S_{dy}} = \frac{1}{4}$$

$$\frac{S^{**}{}_{dC} - S_{dy}}{S_{du} - S_{dy}} = \frac{1}{2}$$

$(c)\ I = 1.5$

建築物之耐震性能檢核標準

RC建築物層間相對位移角檢核標準

【用途係數I=1.0】 性能狀態PLB對應之任一樓層相對位移角，應小於或等於3%。若結構物達到性能點前，任一樓層之層間相對位移角已達到3%，則定義該樓層最大層間位移角為3%對應之EPA為性能狀態PLB，並且應大於或等於475年回歸期地震之EPA $0.4S_{DS}$ (=　　　)。

【用途係數I=1.25】 性能狀態PLB*對應之任一樓層相對位移角，應小於或等於2. 4%。若結構物達到性能點前，任一樓層之層間相對位移角已達到2. 4%，則定義該樓層最大層間位移角為2. 4%對應之EPA為性能狀態PLB*，並且應大於或等於475年回歸期地震之EPA $0.4S_{DS}$ (=　　　)。

【用途係數I=1.5】 性能狀態PLB**對應之任一樓層相對位移角，應小於或等於2%。若結構物達到性能點前，任一樓層之層間相對位移角已達到2%，則定義該樓層最大層間位移角為2%對應之EPA為性能狀態PLB**，並且應大於或等於475年回歸期地震之EPA $0.4S_{DS}$ (=　　　)。

地震等級	用途係數		
	I = 1.0	I = 1.25	I = 1.5
475年回歸期地震	3 %	2.4 %	2 %

鋼筋混凝土構件補強
理論探討與分析驗證

耐震評估

補強分析

塑鉸設定

視窗化

專案管理

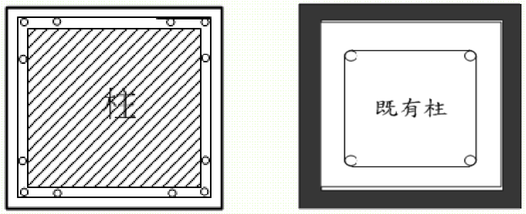
SERCB

構件補強

系統補強

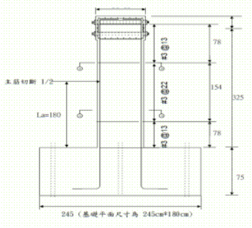
鋼筋混凝土構件補強
理論探討與分析驗證

RC柱包覆補
非線性行為探



強度、圍束補強

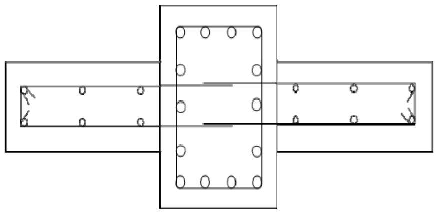
RC柱包覆補強
分析與驗證



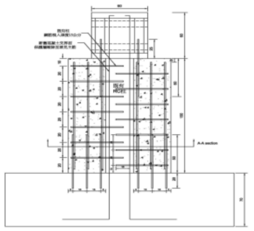
單柱

構架

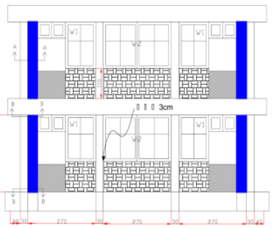
翼牆補強
分析與驗證



雙側、單側、偏心



單柱



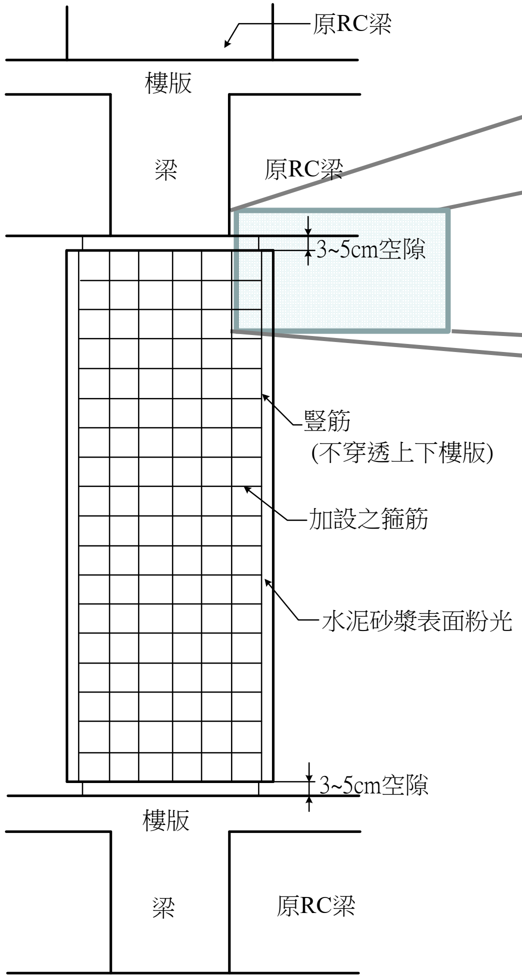
構架

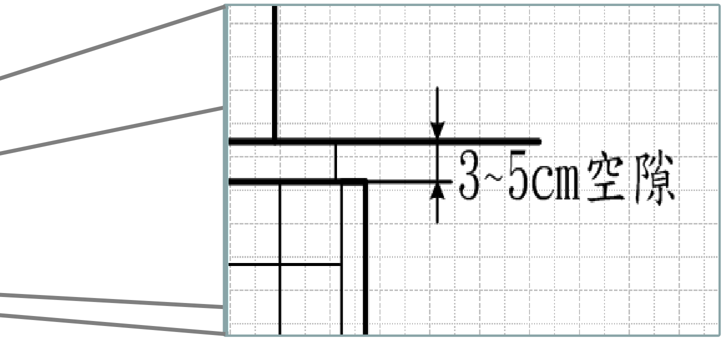
分析方法

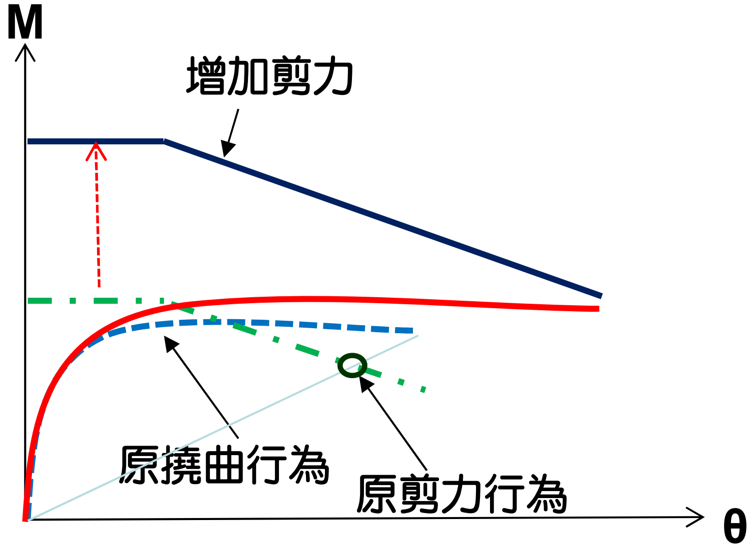
- ▪切片法
- ▪撓曲分析
- ▪剪力分析
- ▪破壞模式

構件補強非線性行為分析方法

RC包覆圍束補強工法

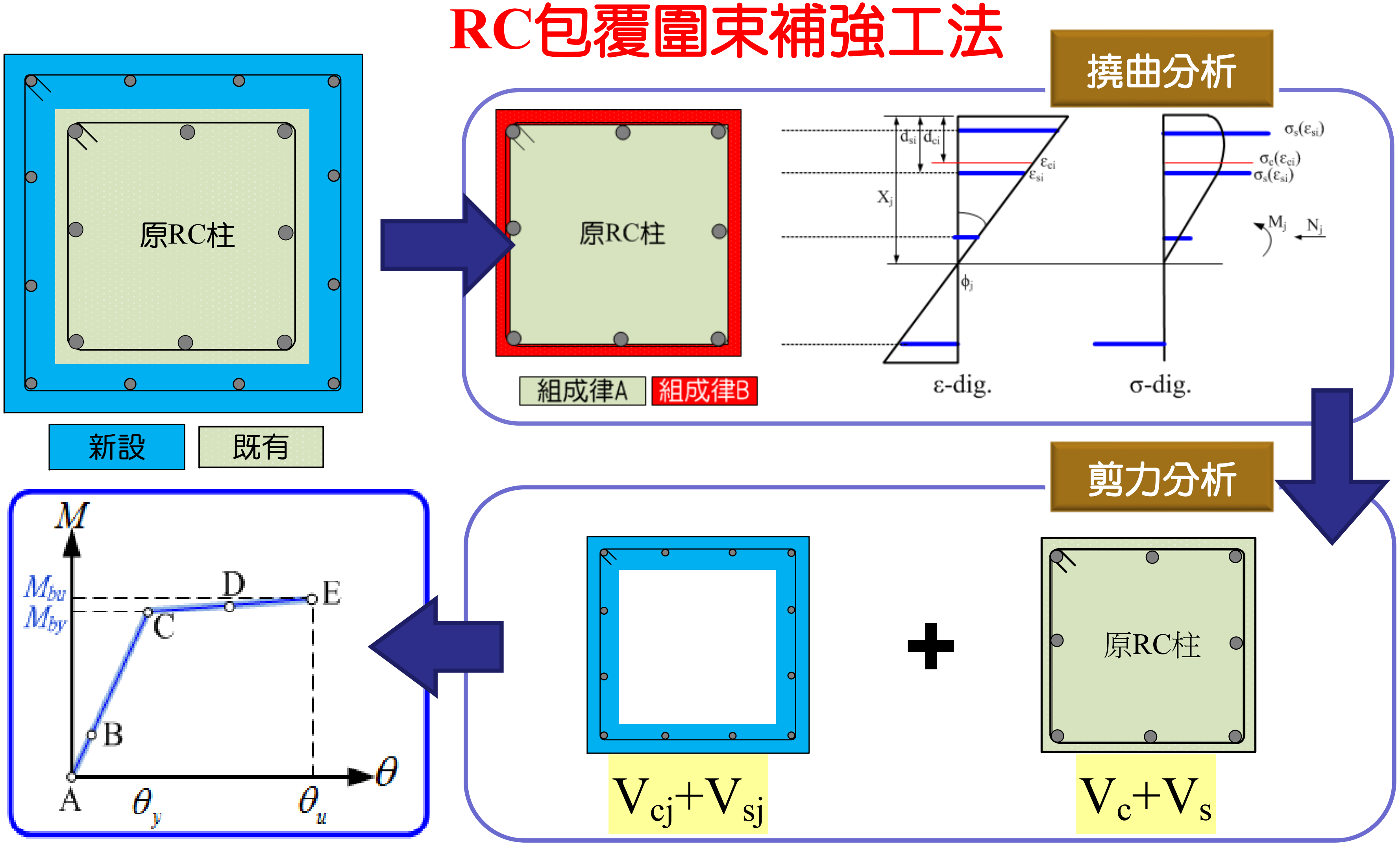






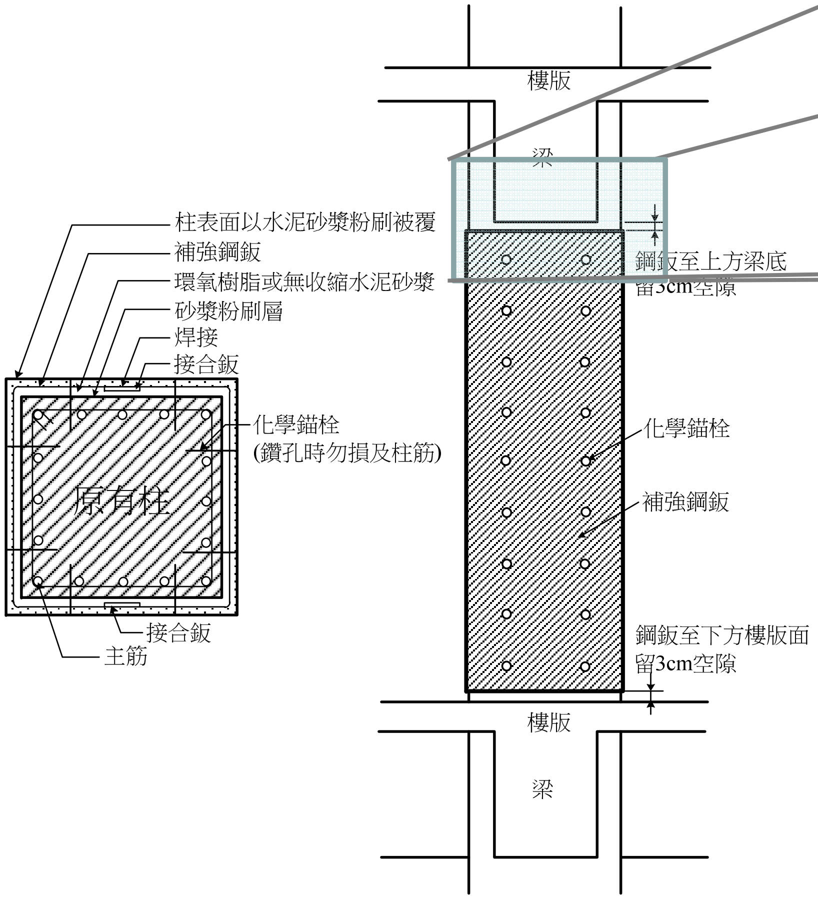
構件補強非線性行為分析方法

RC包覆圍束補強工法

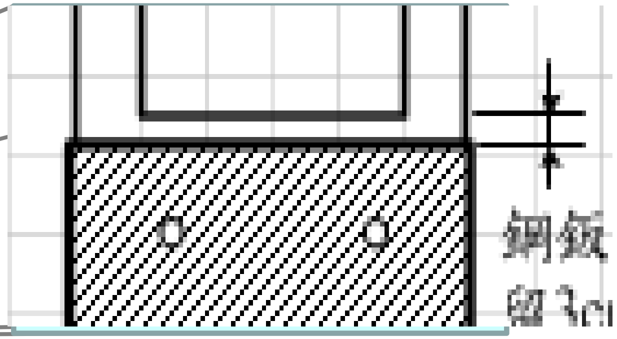


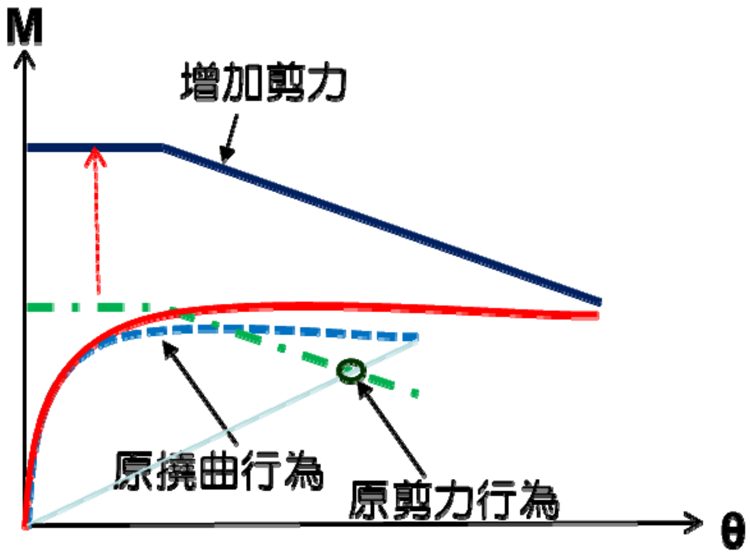
構件補強非線性行為分析方法

鋼板包覆圍束補強工法



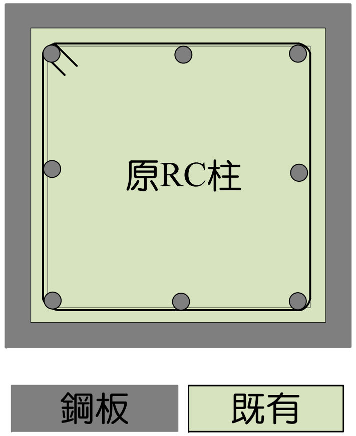
立面圖



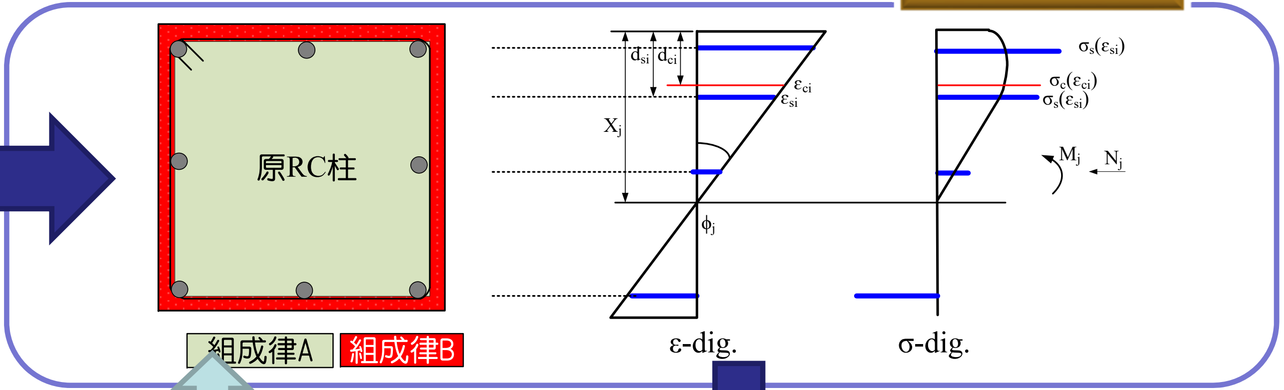


構件補強非線性行為分析方法

鋼板包覆圍束補強工法



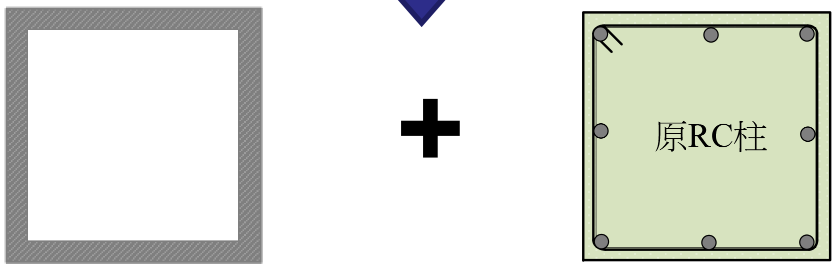
撓曲分析



$\rho_{steel} = 2t/b_2$ 矩型

$\rho_{steel} = 4t/D$ 圓型

剪力分析

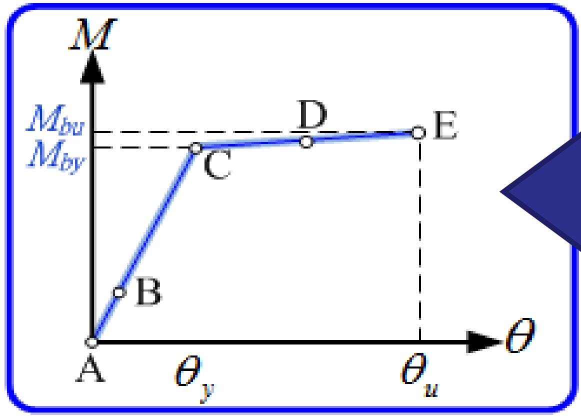


V_{sj}

$$V_{sj} = \frac{\pi}{2} t_j f_{yj} D Cot\theta \quad 圓型$$

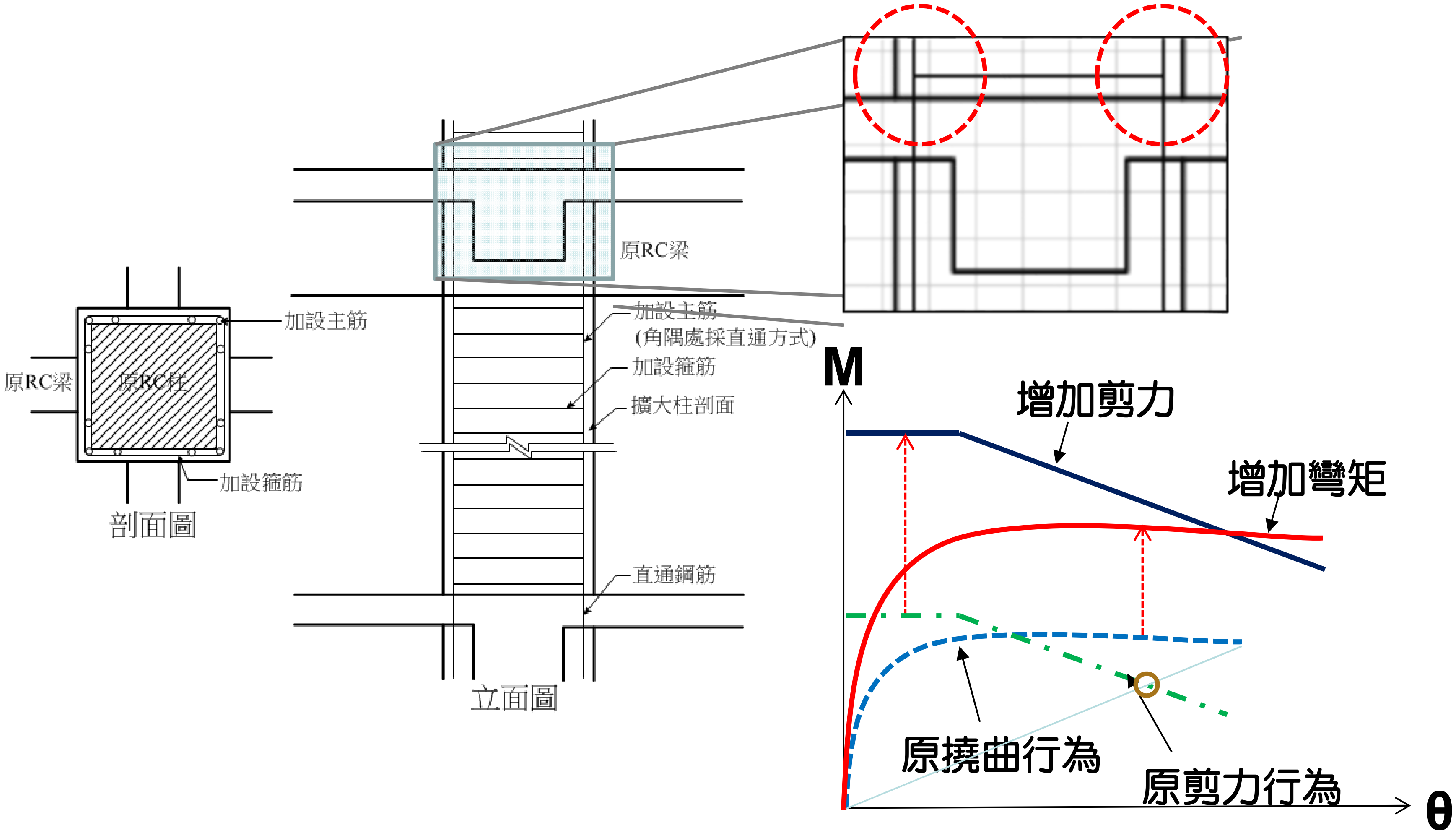
$$V_{sj} = t_j f_{yj} D Cot\theta \quad 矩型$$

$V_c + V_s$



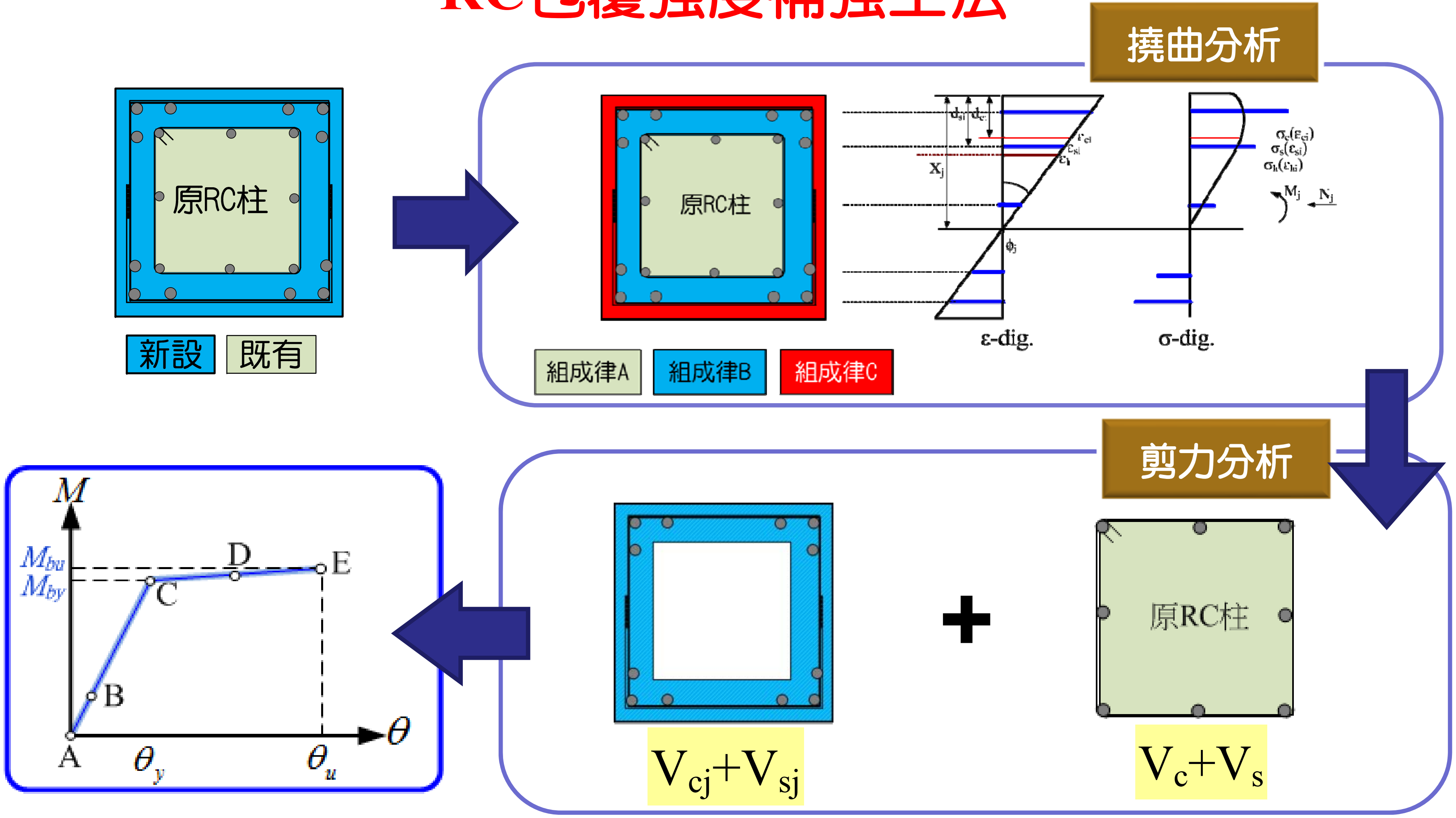
構件補強非線性行為分析方法

RC包覆強度補強工法



構件補強非線性行為分析方法

RC包覆強度補強工法



新設　既有

撓曲分析

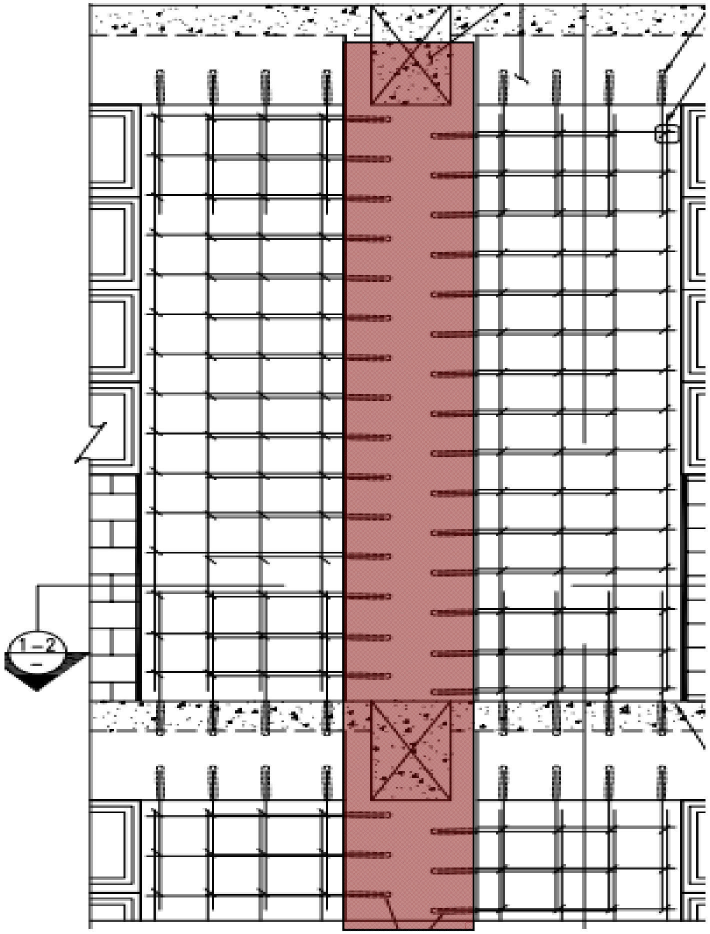
組成律A　組成律B　組成律C

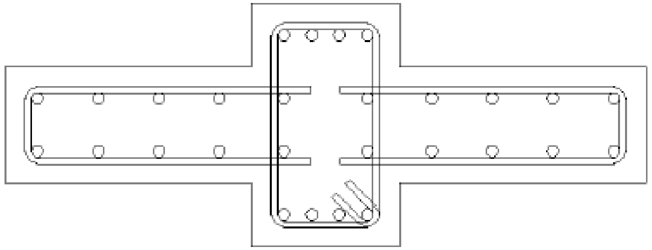
剪力分析

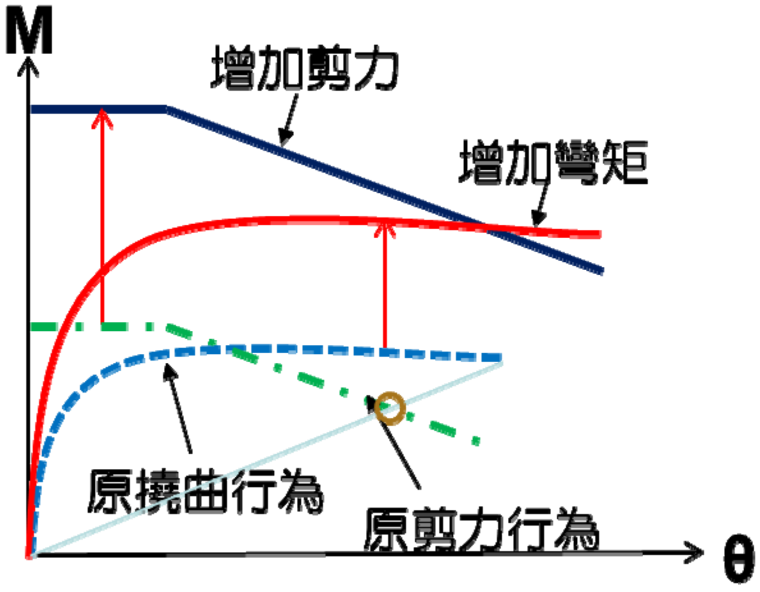
$V_{cj}+V_{sj}$ ＋ V_c+V_s

構件補強非線性行為分析方法

翼牆補強工法

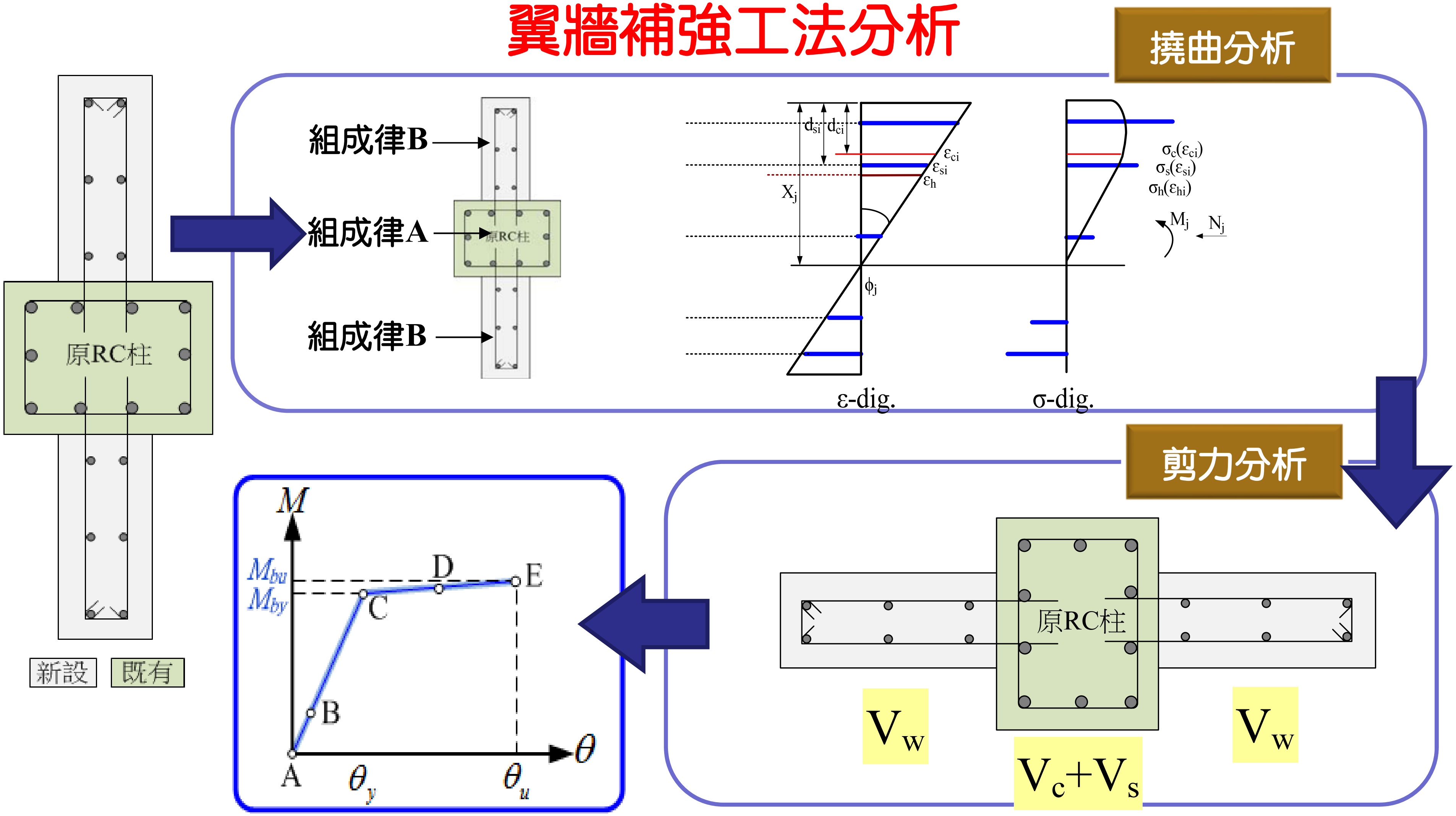






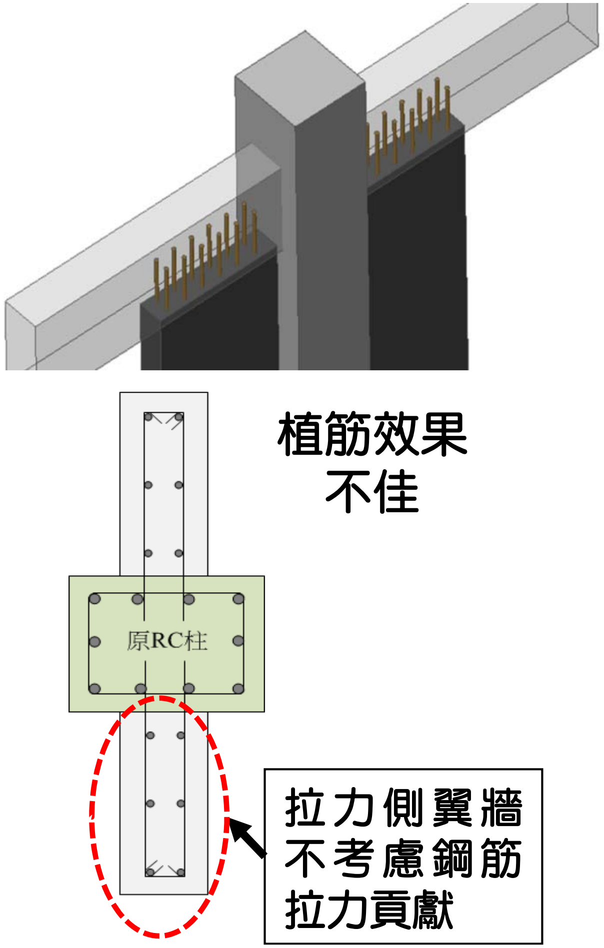
構件補強非線性行為分析方法

翼牆補強工法分析



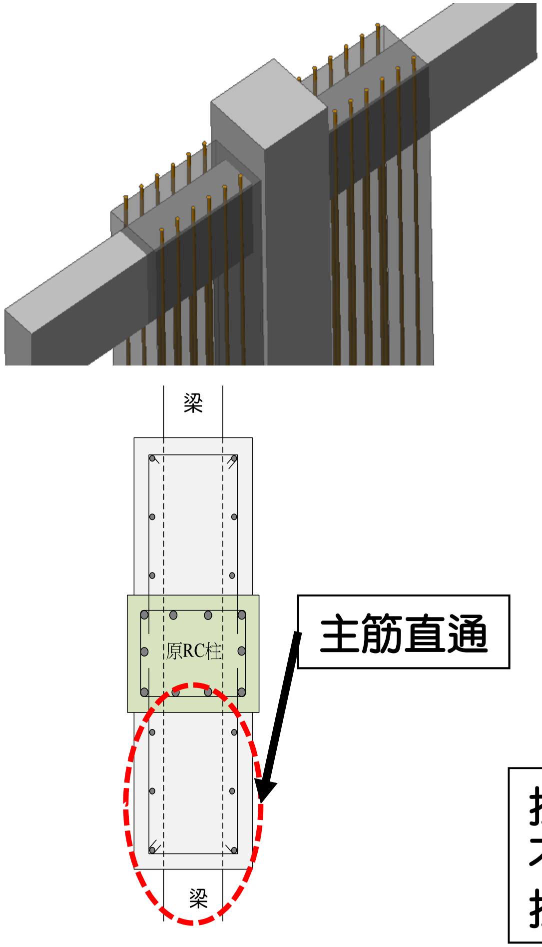
構件補強非線性行為分析方法

翼牆補強工法分析種類

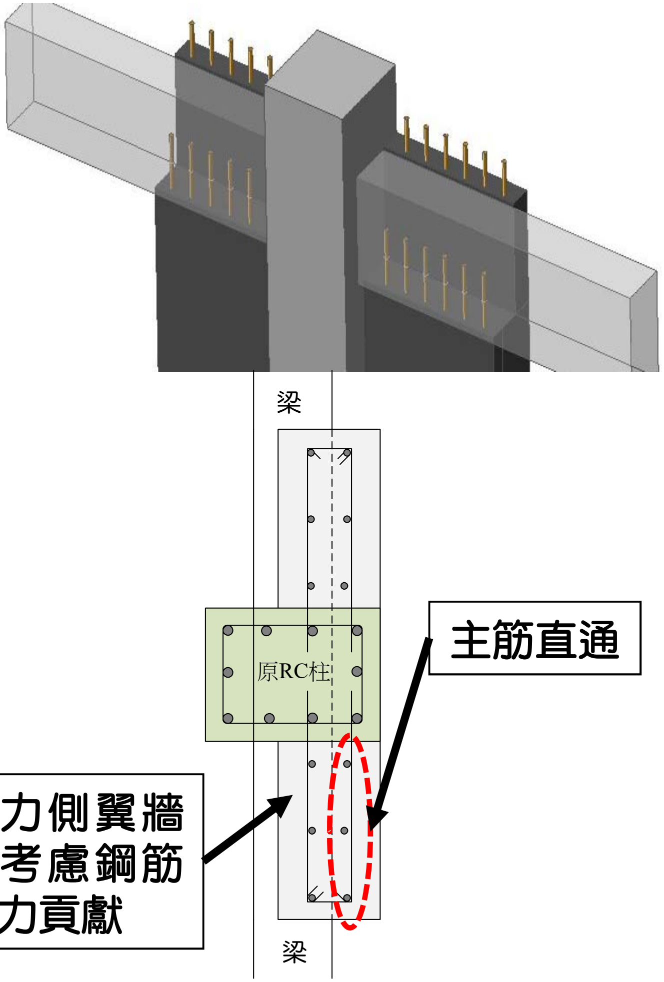


植筋效果不佳

拉力側翼牆不考慮鋼筋拉力貢獻



主筋直通

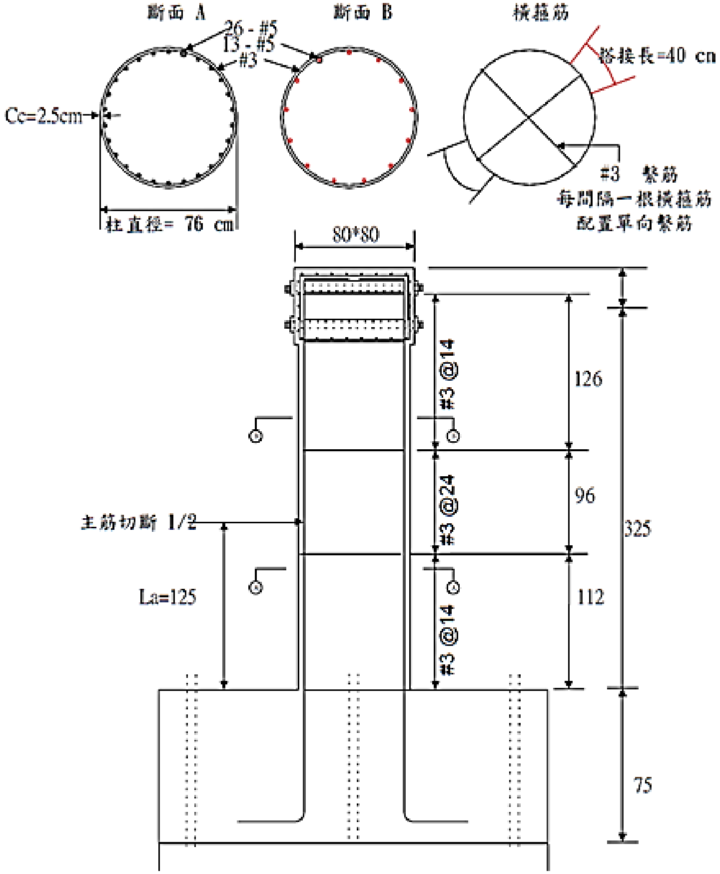


主筋直通

拉力側翼牆不考慮鋼筋拉力貢獻

構件補強非線性行為分析驗證

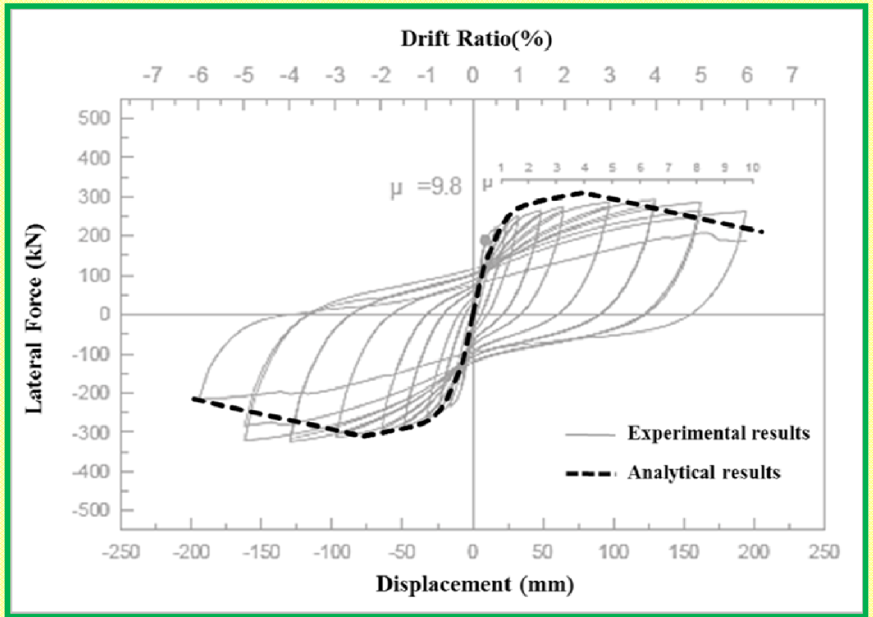
鋼板包覆圍束補強



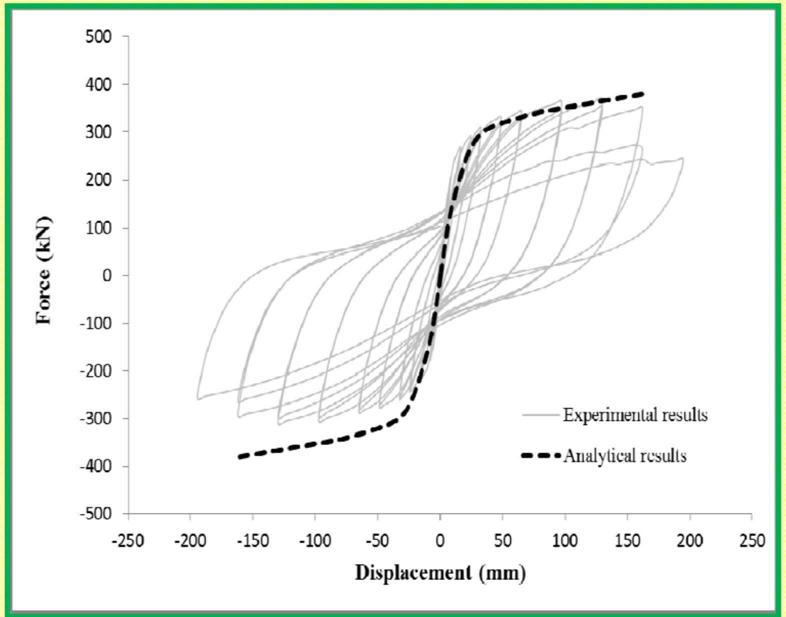
試體SC2(SC1) 配筋示意圖(黃震興，1999)

試體編號		SC2	SC3
斷面形狀		圓形	圓形
混凝土抗壓強度	kgf/cm^2	260	260
混凝土保護層	cm	2.5	2.5
斷面尺寸(深×寬)	cm	76 × 76	76 × 76
柱高	cm	325	325
主筋降伏強度	kgf/cm^2	3500	3500
主筋配置	–	26–#5	30–#5
箍筋降伏強度	kgf/cm^2	5000	5000
塑鉸區箍筋間距	cm	#3@13	#3@13
補強鋼板厚度	cm	0.3	0.3
柱軸力	tonf	143	143

構件補強非線性行為分析驗證



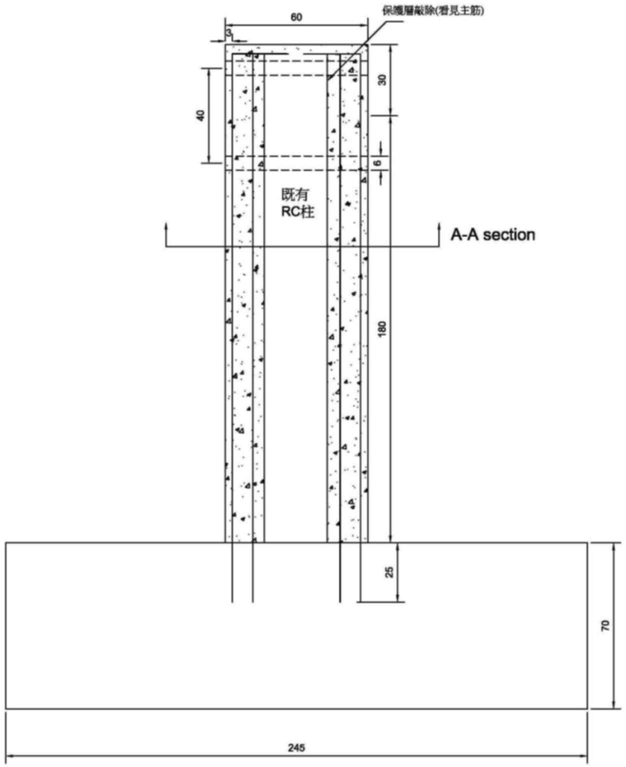
試體編號：SC2



試體編號：SC3

構件補強非線性行為分析驗證

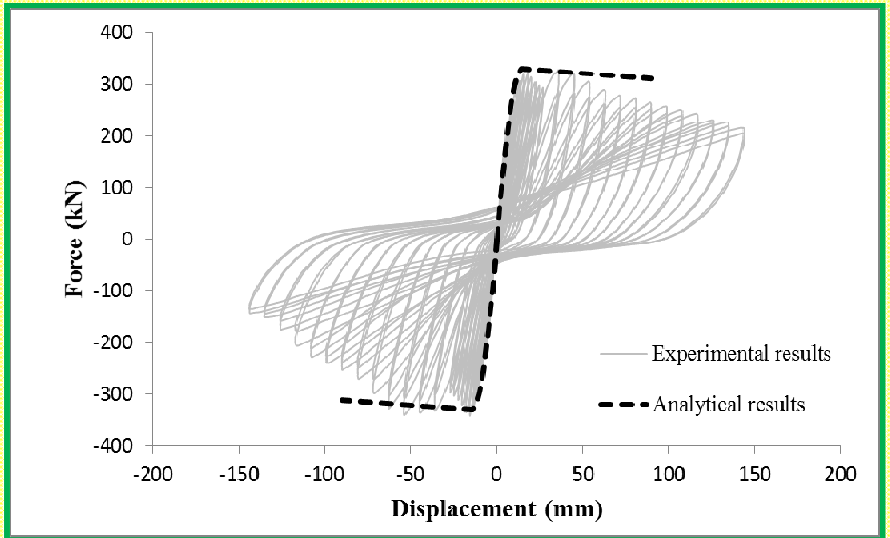
RC包覆強度補強



擴柱補強柱S2之立面圖(張順益，2010)

試體編號		S2
斷面形狀		矩形
混凝土抗壓強度	kgf/cm^2	175
混凝土保護層	cm	4
斷面尺寸(深×寬)	cm	40 × 30
柱高	cm	180
主筋降伏強度	kgf/cm^2	3547
主筋配置	–	14–#5
箍筋降伏強度	kgf/cm^2	4257
塑鉸區箍筋間距	cm	#3@25

構件補強非線性行為分析驗證

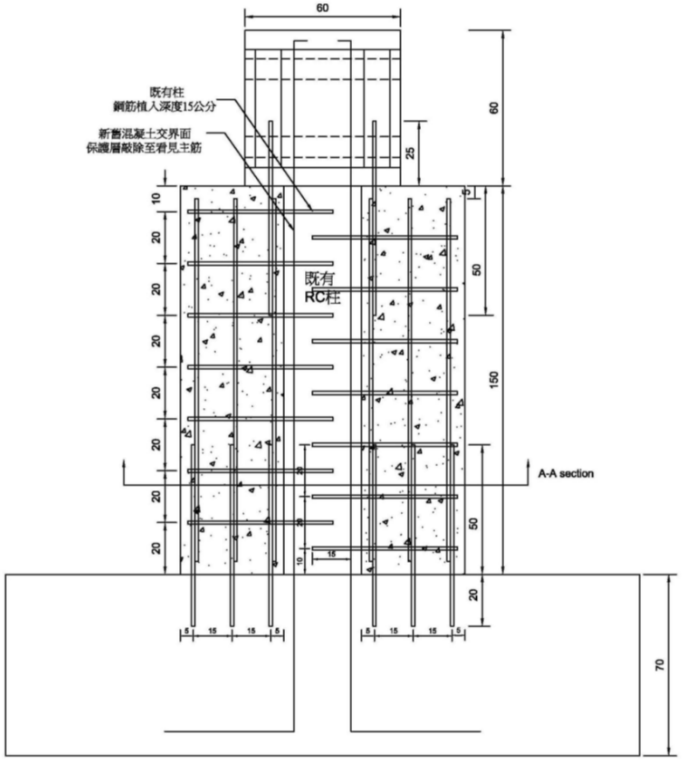


試體編號：S2

試體編號(補強資料)		S2
斷面形狀		矩形
補強混凝土厚度	cm	15
補強斷面(深×寬)	cm	70 × 60
補強區保護層	cm	3
補強混凝土強度	kgf/cm^2	245
補強主筋配置	–	12-#6
補強箍筋配置	cm	#3@10
補強主筋降伏強度	kgf/cm^2	4995
補強箍筋降伏強度	kgf/cm^2	4257
RC包覆與基礎間隙	cm	–
柱軸力	Tonf	28

構件補強非線性行為分析驗證

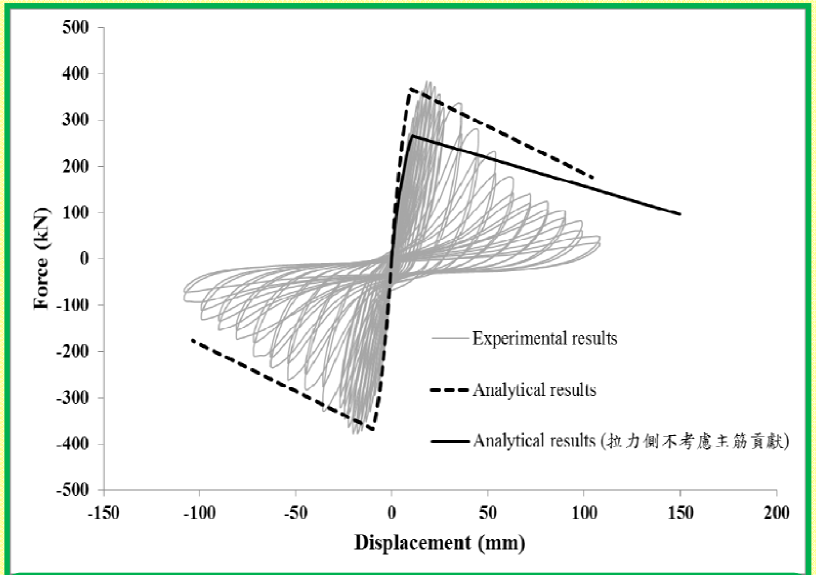
翼牆補強



翼牆補強柱S5之立面圖(張順益，2010)

試體編號		S5
斷面形狀		矩形
混凝土抗壓強度	kgf/cm^2	175
混凝土保護層	cm	4
斷面尺寸(深×寬)	cm	40 × 30
柱高	cm	150
主筋降伏強度	kgf/cm^2	3547
主筋配置	–	14–#5
箍筋降伏強度	kgf/cm^2	4257
塑鉸區箍筋間距	cm	#3@25

構件補強非線性行為分析驗證

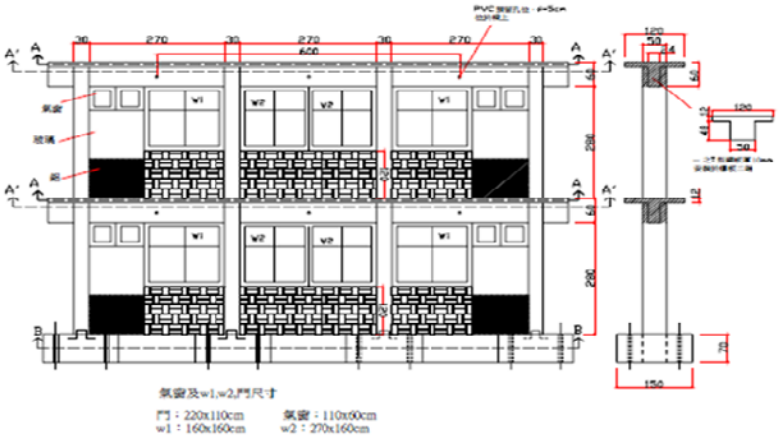


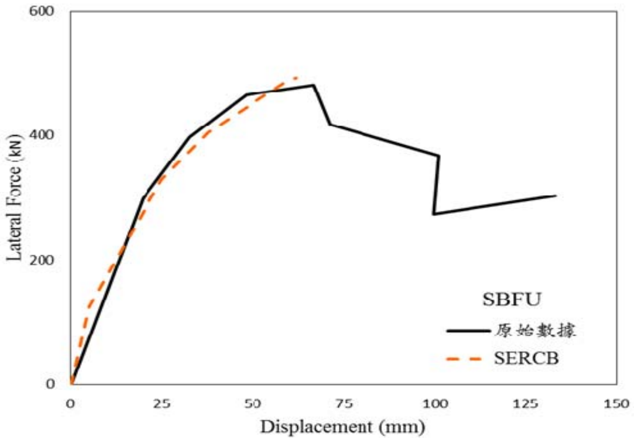
試體編號：S5

試體編號(補強資料)		S5
斷面形狀		矩形
翼牆斷面(深×寬)	cm	18× 40
補強區保護層	cm	2.7
補強混凝土強度	kgf/cm^2	245
補強主筋配置	–	6-#4
補強箍筋配置	cm	#4@20
補強主筋降伏強度	kgf/cm^2	4159
補強箍筋降伏強度	kgf/cm^2	4159
基礎間隙	cm	–
柱軸力	Tonf	28

構件補強非線性行為分析驗證

鋼筋混凝土構架

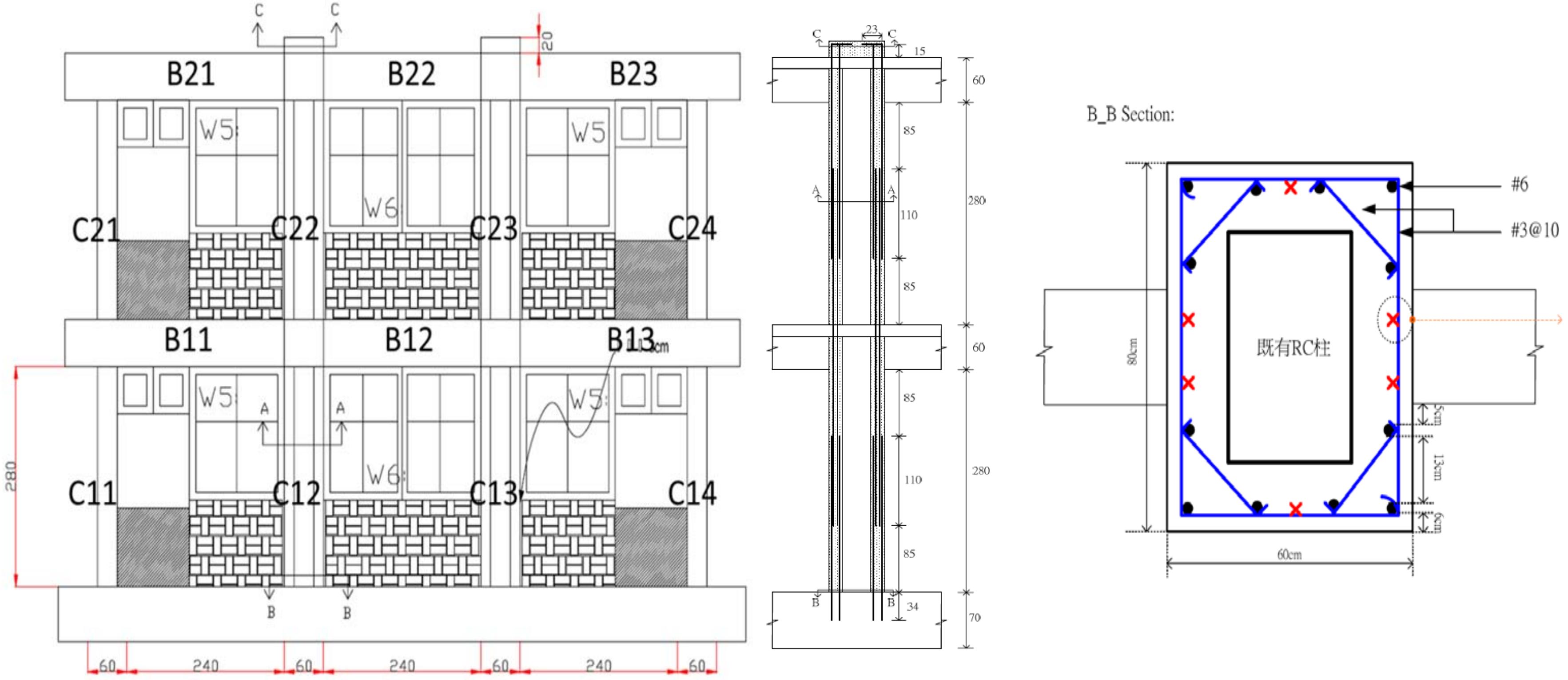




(典型校舍耐震補強設計與驗證，邱聰智、邱建國、葉勇凱、簡文郁、鐘立來、周德光，2008)

構件補強非線性行為分析驗證

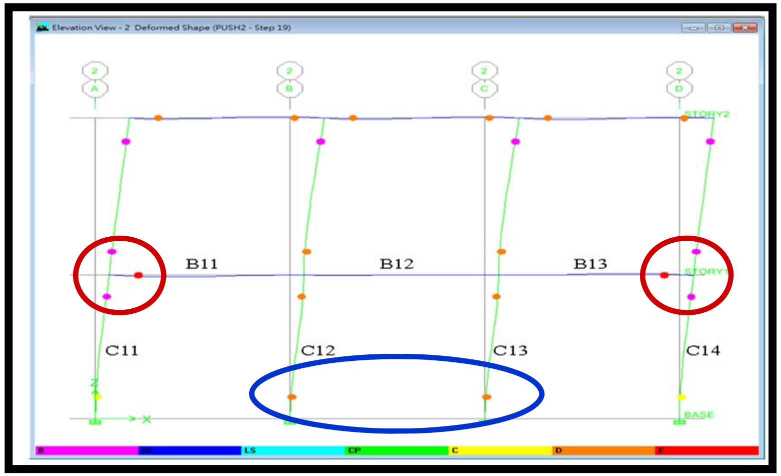
鋼筋混凝土構架- RC包覆強度補強(SBFU-C)



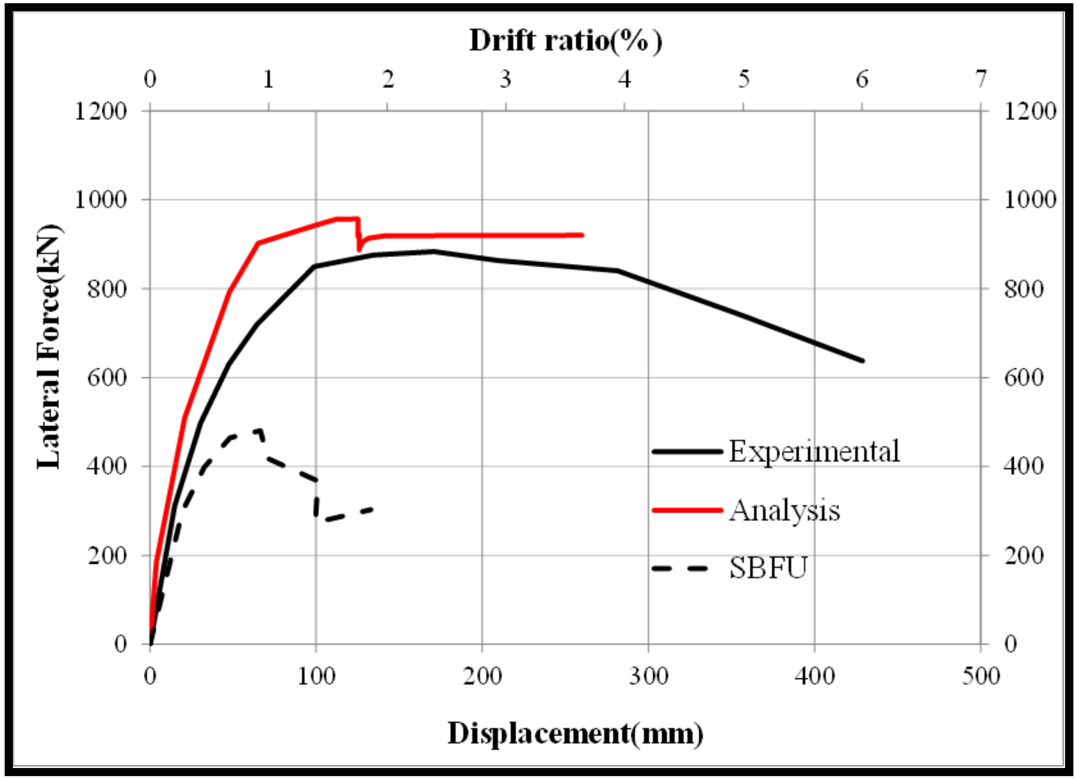
國家地震中心SBFU-C試體

構件補強非線性行為分析驗證

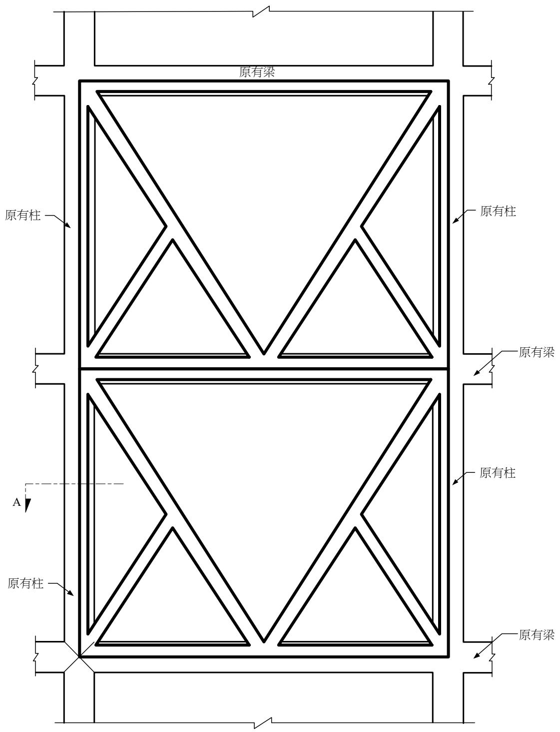
鋼筋混凝土構架- RC包覆強度補強(SBFU-C)



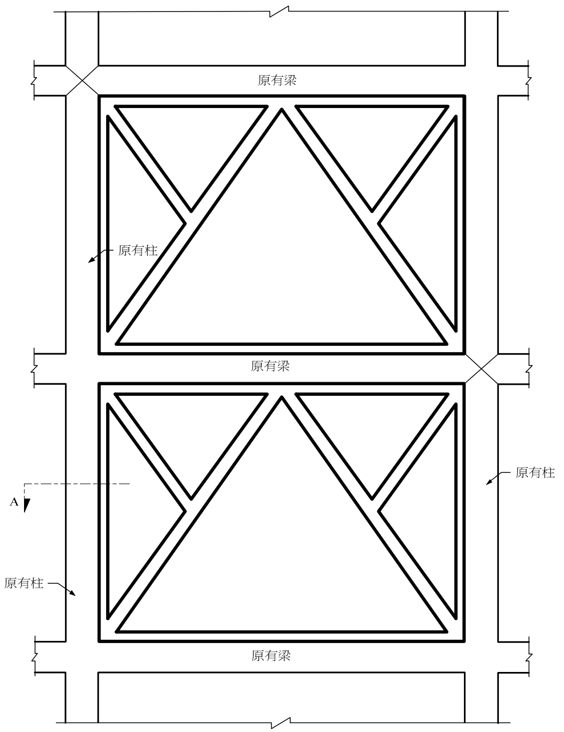
Elevation View - 2 Deformed Shape (PUSH2 - Step 19)



鋼斜撐框架補強理論探討與分析驗證



外部鋼框架補強示意圖



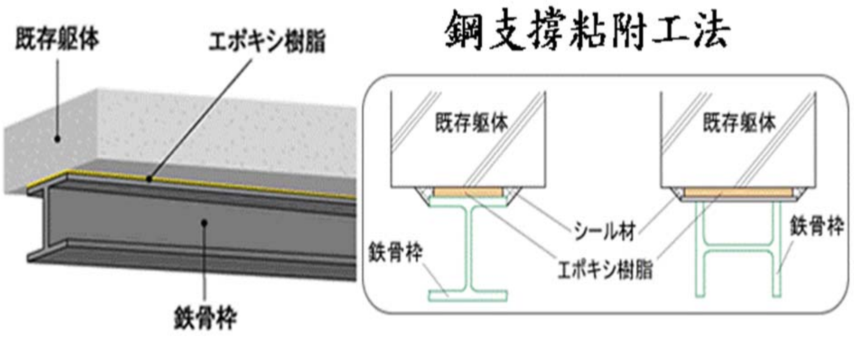
增設鋼框架補強示意圖

外附式鋼斜撐框架補強

內嵌式鋼斜撐框架補強

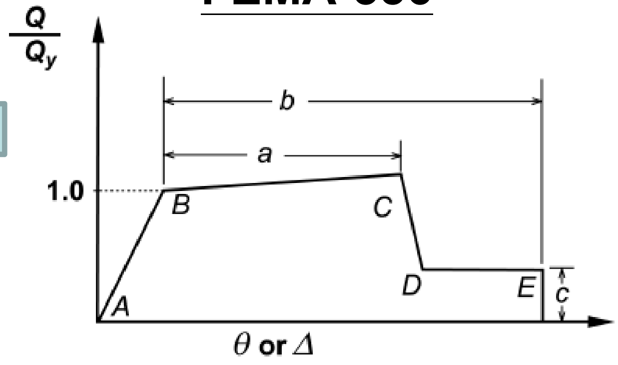
鋼斜撐框架補強理論探討與分析驗證

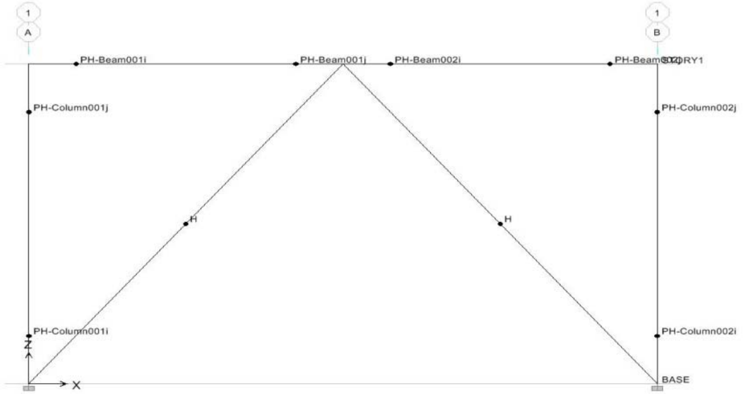
內嵌式-環氧樹脂接合



忽略鋼框架
僅考慮鋼斜撐

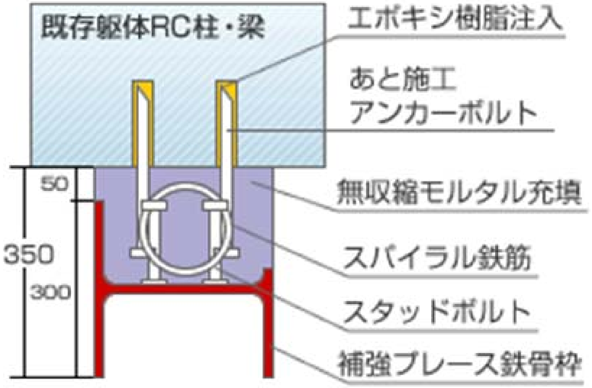
FEMA-356





鋼斜撐框架補強理論探討與分析驗證

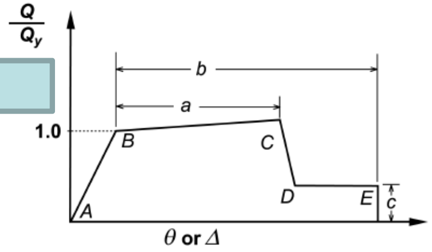
內嵌式-錨碇接合



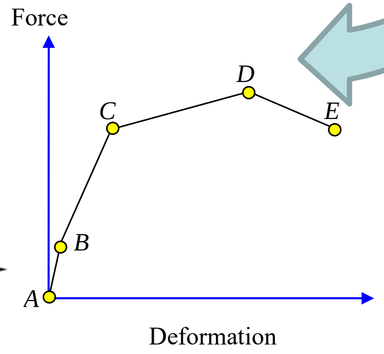
**梁、柱以合成斷面
進行切面法分析**

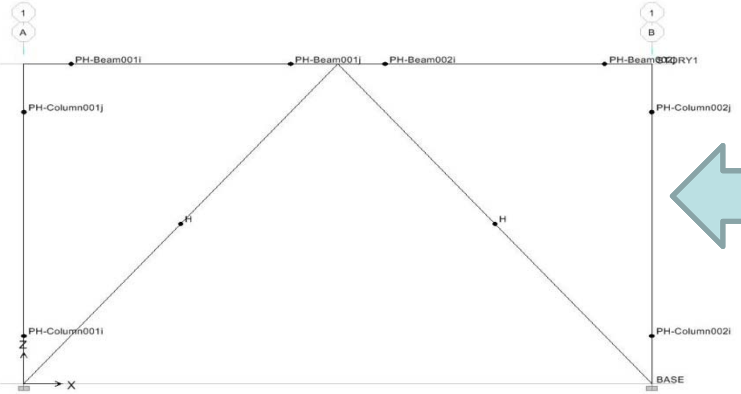
鋼斜撐-FEMA-

FEMA-356



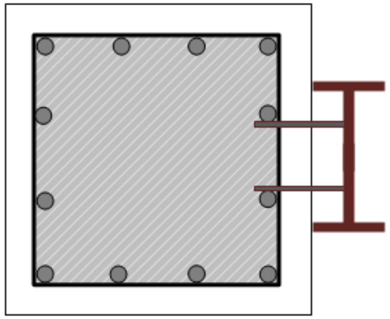
SERCB-合成斷面



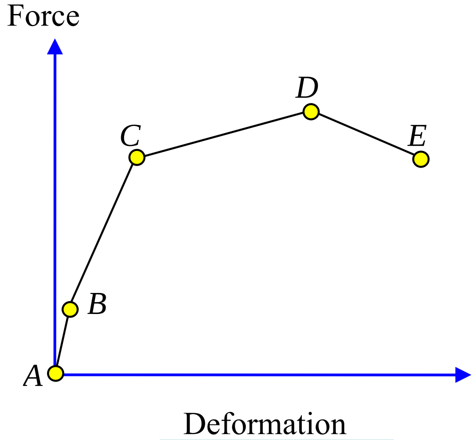


鋼斜撐框架補強理論探討與分析驗證

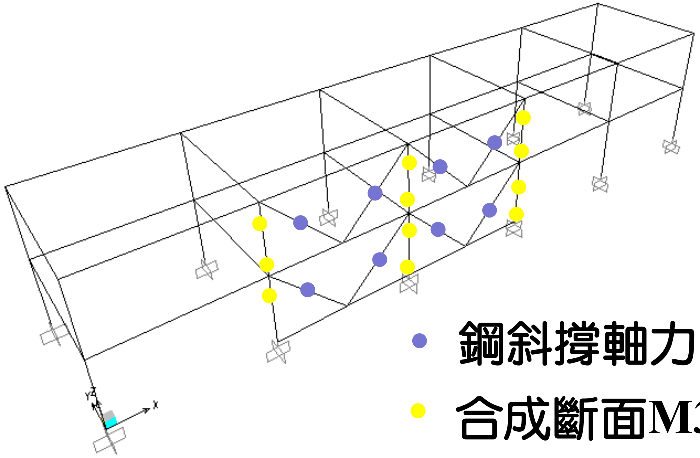
外附式鋼斜撐框架



切片法



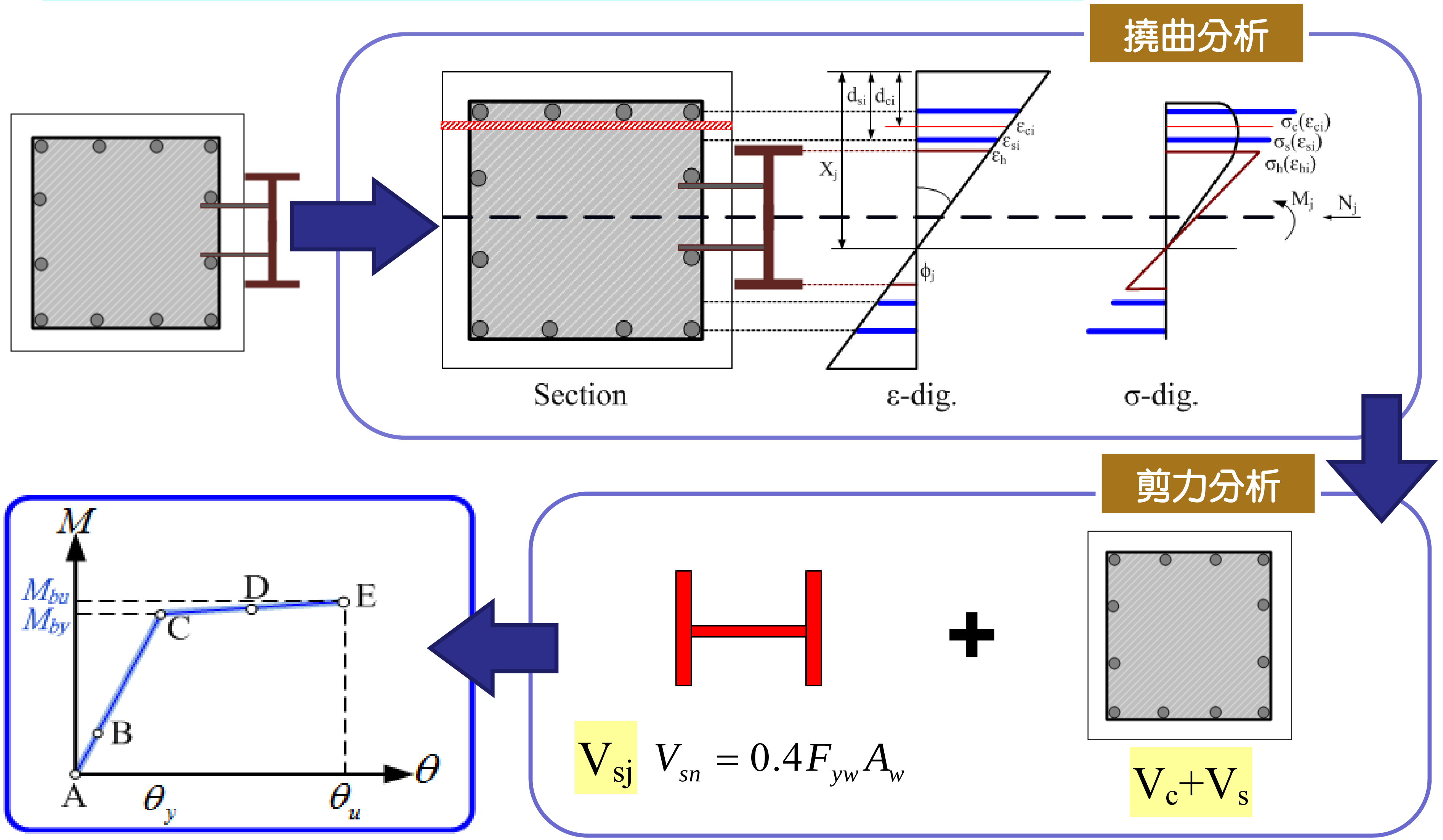
塑性鉸



● 鋼斜撐軸力塑性鉸

● 合成斷面M3塑性鉸

鋼斜撐框架補強理論探討與分析驗證



V_{sj} $V_{sn}=0.4F_{yw}A_w$

V_c+V_s

鋼斜撐框架補強理論探討與分析驗證

拉力構件

$$Q_{CE} = T_n = A_g F_y$$

A_g斜撐斷面積

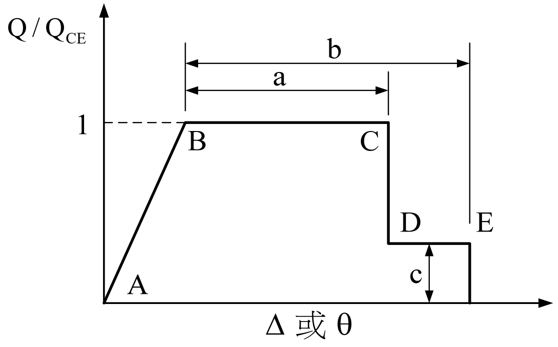
壓力構件

$$Q_{CE} = P_n = A_g F_{cr}$$

$$\lambda_c \le 1.5$$
$$F_{cr} = 0.658^{\lambda_c^2} F_y$$

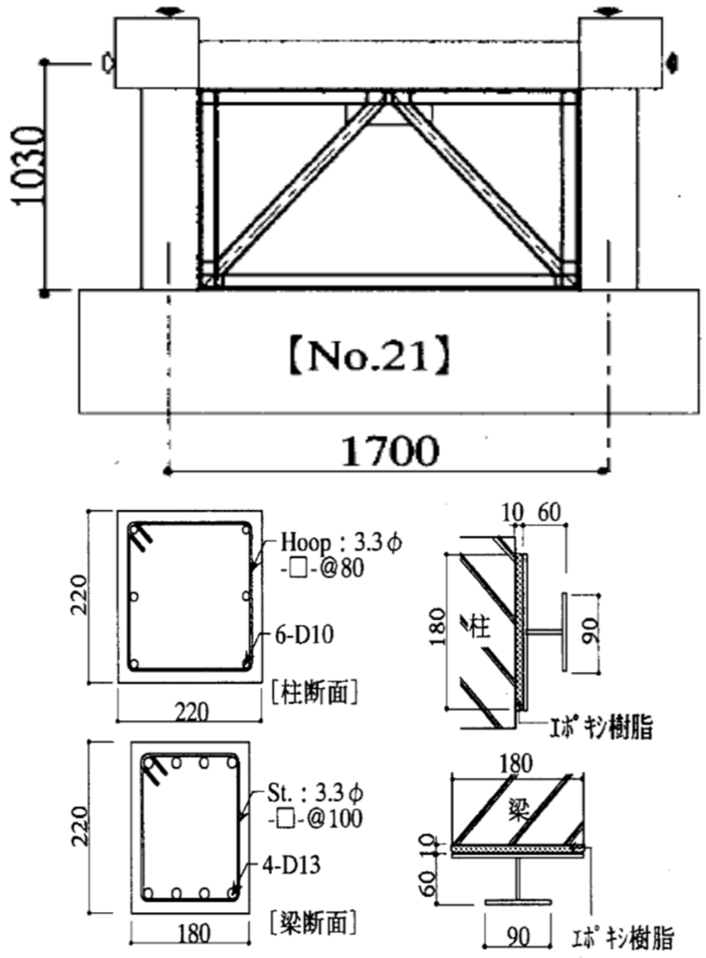
$$\lambda_c > 1.5$$
$$F_{cr} = 0.877 F_y / \lambda_c^2$$

	模型參數			可接受標準				
	塑性變形		殘餘強度比	塑性變形				
				IO	主要桿件		次要桿件	
桿件	a	b	c		LS	CP	LS	CP
受壓斜撐(EBF斜撐除外)								
a.雙角鋼面內挫屈	$0.5\Delta_c$	$9\Delta_c$	0.2	$0.25\Delta_c$	$5\Delta_c$	$7\Delta_c$	$7\Delta_c$	$8\Delta_c$
b.雙角鋼面外挫屈	$0.5\Delta_c$	$8\Delta_c$	0.2	$0.25\Delta_c$	$4\Delta_c$	$6\Delta_c$	$6\Delta_c$	$7\Delta_c$
c.W或I型	$0.5\Delta_c$	$8\Delta_c$	0.2	$0.25\Delta_c$	$5\Delta_c$	$7\Delta_c$	$7\Delta_c$	$8\Delta_c$
d.雙槽型鋼面內挫屈	$0.5\Delta_c$	$9\Delta_c$	0.2	$0.25\Delta_c$	$5\Delta_c$	$7\Delta_c$	$7\Delta_c$	$8\Delta_c$
e.雙槽型鋼面外挫屈	$0.5\Delta_c$	$8\Delta_c$	0.2	$0.25\Delta_c$	$4\Delta_c$	$6\Delta_c$	$6\Delta_c$	$7\Delta_c$
f.鋼管混凝土	$0.5\Delta_c$	$7\Delta_c$	0.2	$0.25\Delta_c$	$4\Delta_c$	$6\Delta_c$	$6\Delta_c$	$7\Delta_c$
受接斜撐(EBF斜撐除外)	$11\Delta_T$	$14\Delta_T$	0.8	$0.25\Delta_T$	$7\Delta_T$	$9\Delta_T$	$11\Delta_T$	$13\Delta_T$



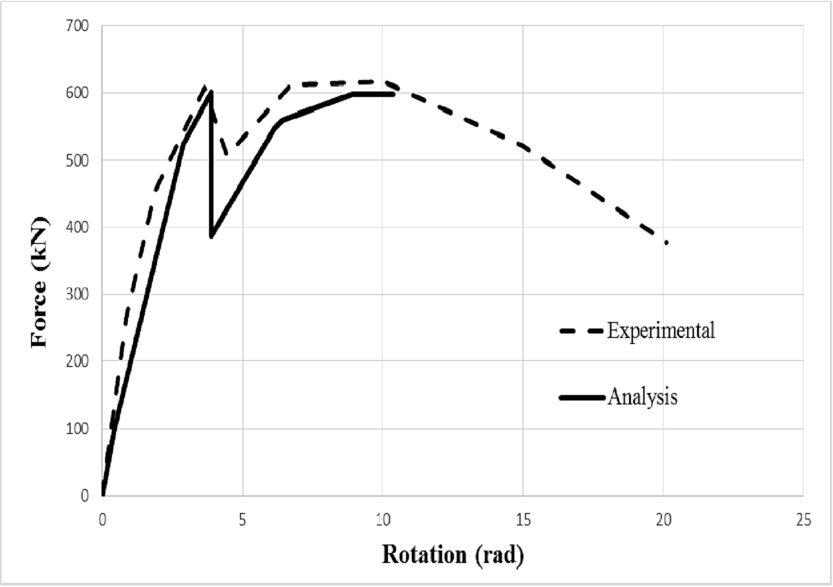
FEMA-356軸力位移關係圖

內嵌式鋼斜撐框架補強試體分析比對

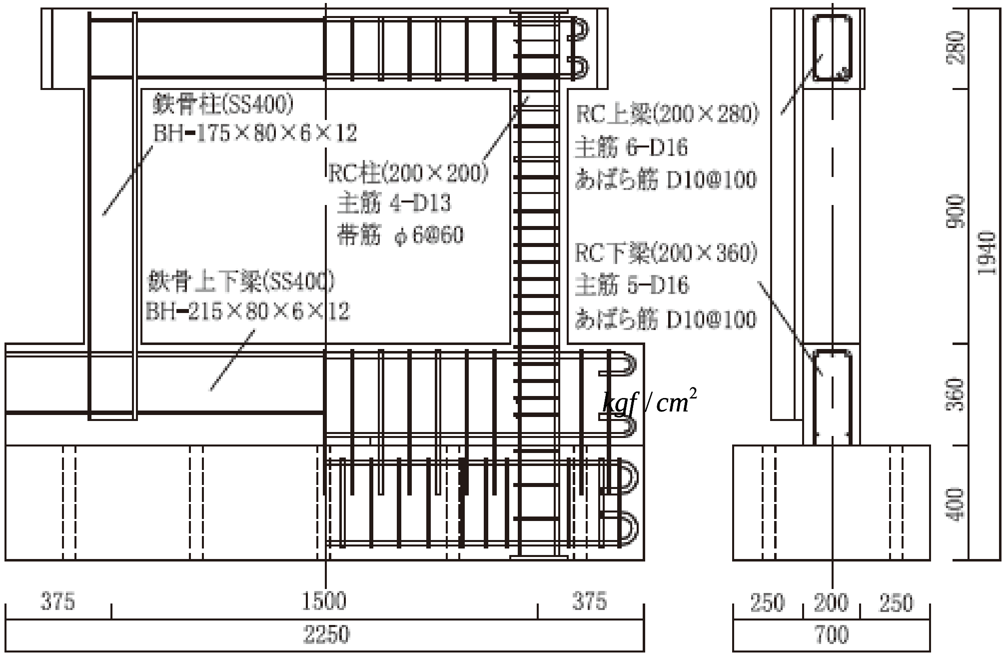


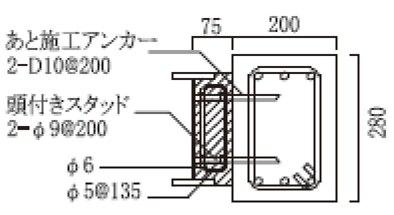
大塚真祐，毛井崇博(1999)

試體編號		No.21
混凝土抗壓強度	kgf/cm^2	330
型鋼降伏應力	kgf/cm^2	3180
D10 主筋降伏應力	kgf/cm^2	3720
D13 主筋降伏應力	kgf/cm^2	3710
箍筋降伏應力	kgf/cm^2	4821

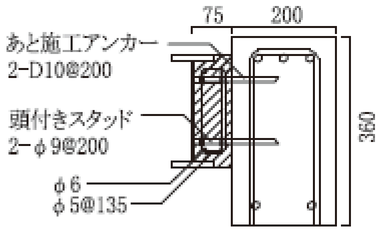


外附式鋼框架補強試體分析比對

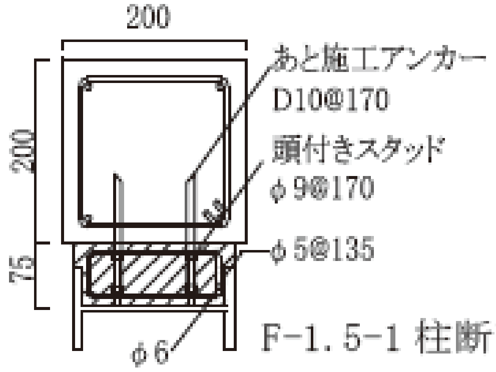




Fシリーズ上梁断面



Fシリーズ下梁断面



F-1.5-1 柱断面

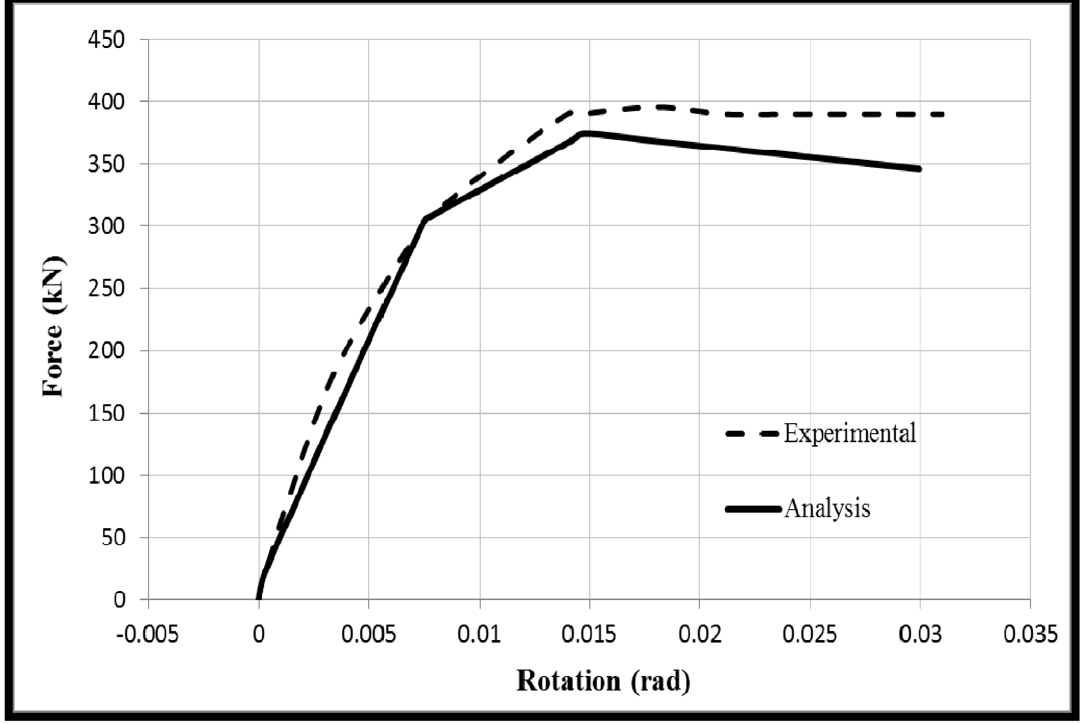
試體編號		F-1.5-1
混凝土抗壓強度	kgf/cm^2	277
型鋼降伏應力	kgf/cm^2	3506
D10 主筋降伏應力	kgf/cm^2	3792
D13 主筋降伏應力	kgf/cm^2	3578

(Rieko UEKI, Katsuhiko IMAI)

外附式鋼框架補強試體分析比對

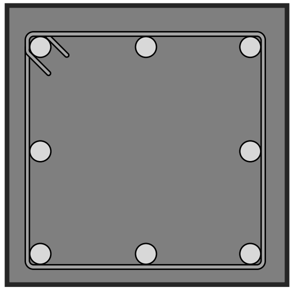
外加鋼框架試體F-1.5-1

F-1.5-1

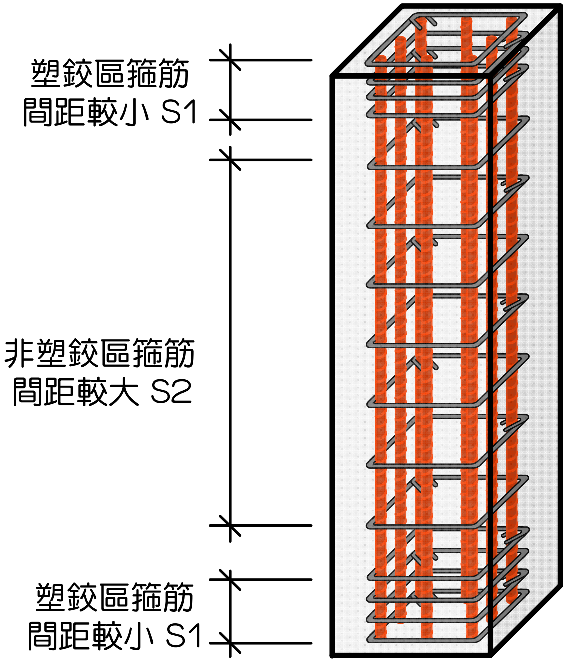


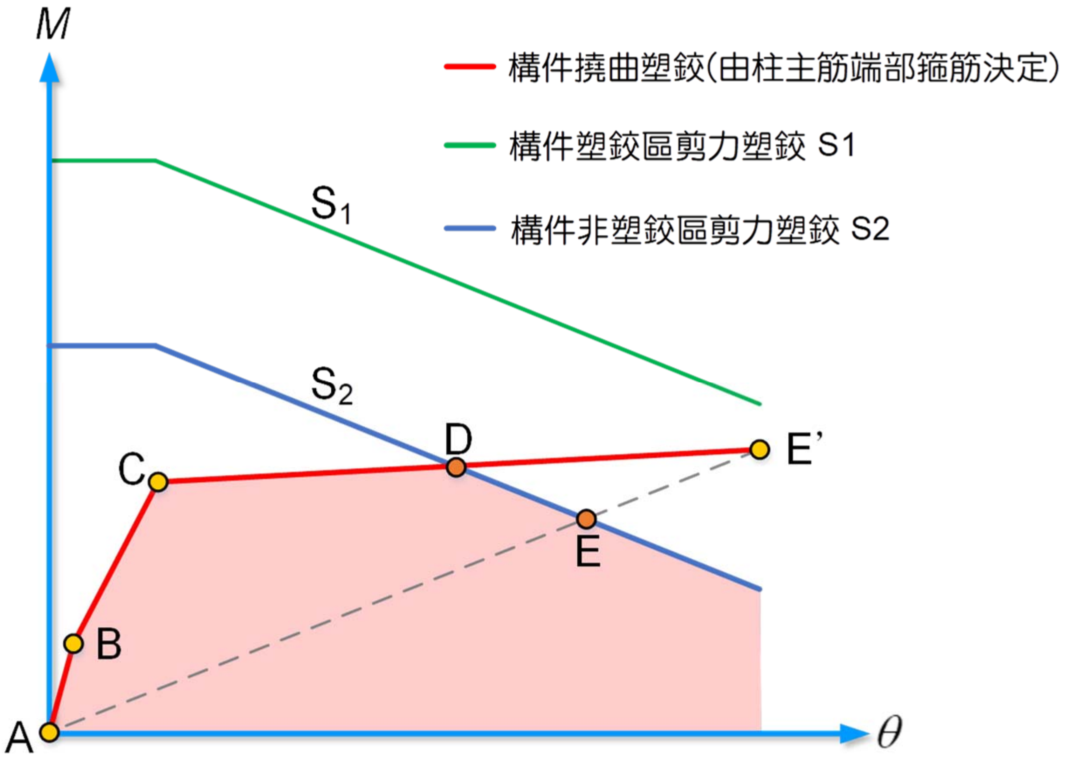
鋼筋混凝土建築結構
耐震能力評估

考慮非塑鉸區箍筋剪力之鋼筋混凝土構件塑性鉸之設定

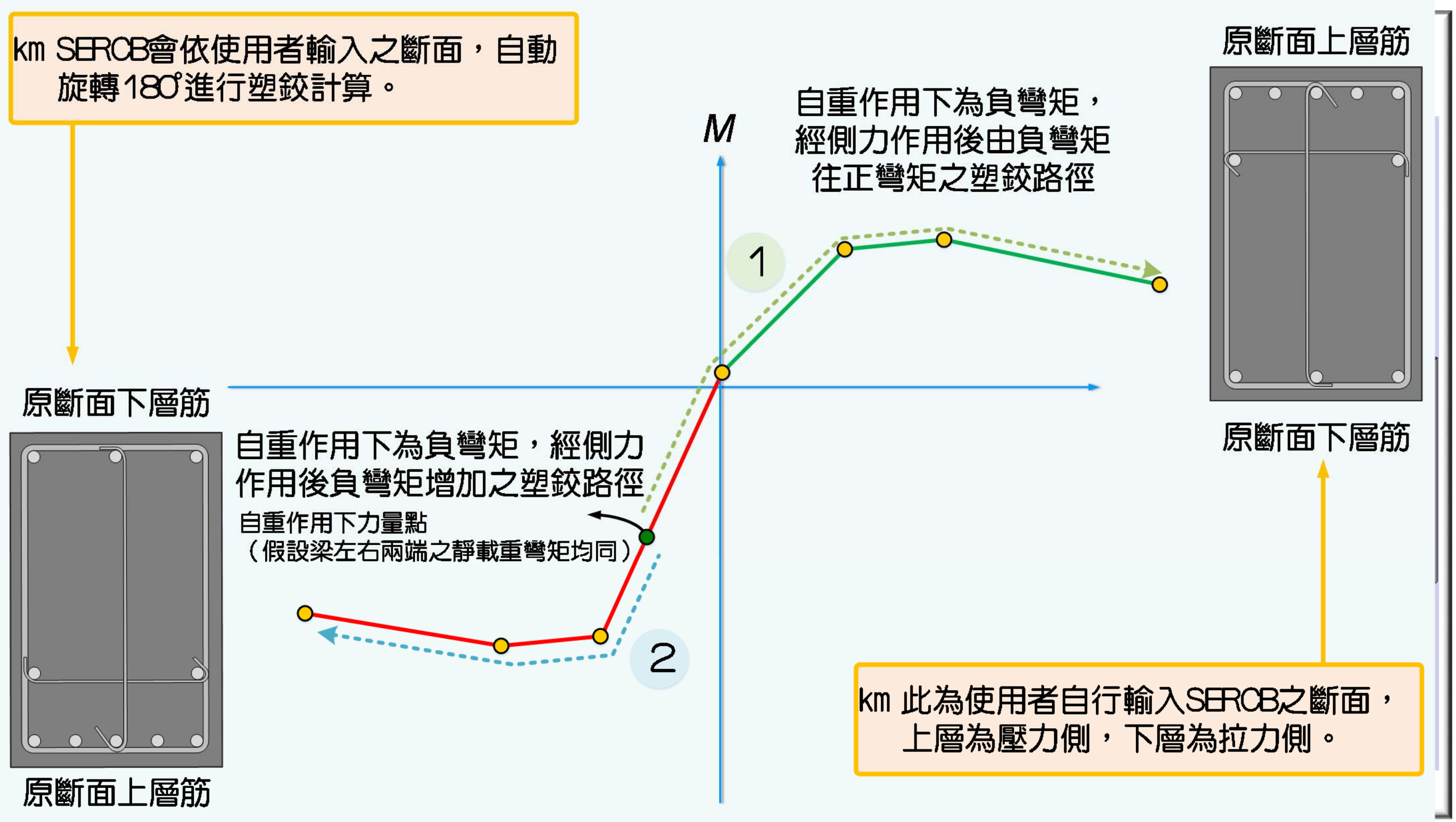


構件斷面示意圖





鋼筋混凝土構架梁端部塑性鉸之設定



km SERCB會依使用者輸入之斷面，自動旋轉180°進行塑鉸計算。

自重作用下為負彎矩，經側力作用後由負彎矩往正彎矩之塑鉸路徑

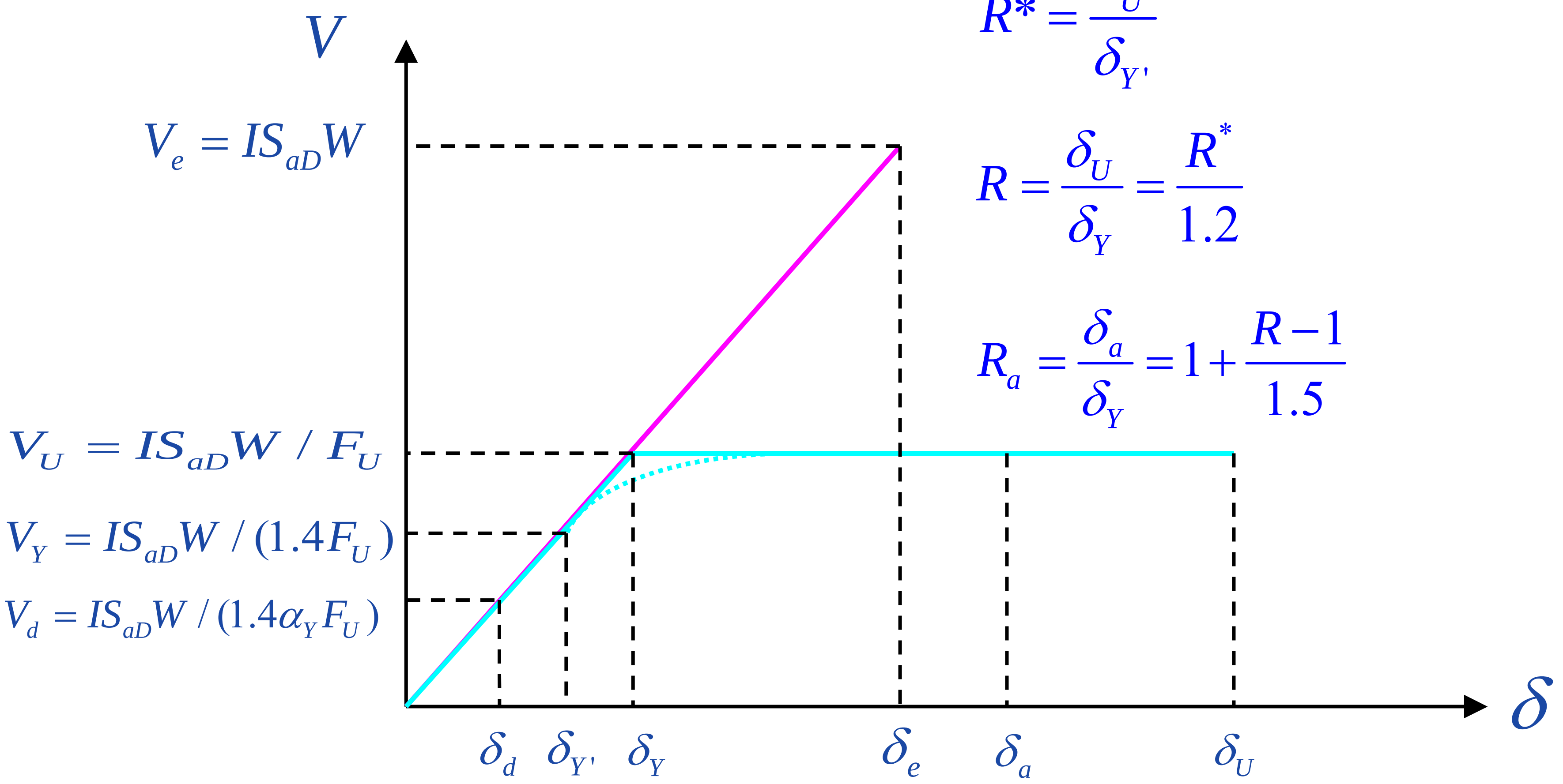
自重作用下為負彎矩，經側力作用後負彎矩增加之塑鉸路徑

自重作用下力量點
（假設梁左右兩端之靜載重彎矩均同）

km 此為使用者自行輸入SERCB之斷面，上層為壓力側，下層為拉力側。

耐震設計規範中各種地震力之定義

彈性系統與非彈性系統



$$R^* = \frac{\delta_U}{\delta_{Y'}}$$

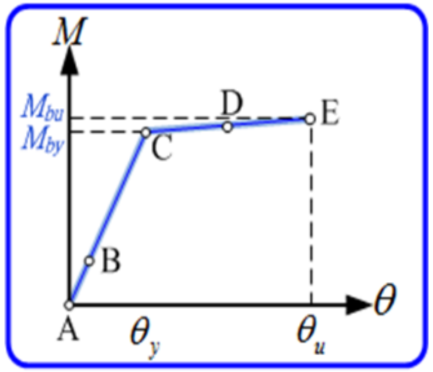
$$R = \frac{\delta_U}{\delta_Y} = \frac{R^*}{1.2}$$

$$R_a = \frac{\delta_a}{\delta_Y} = 1 + \frac{R-1}{1.5}$$

結論

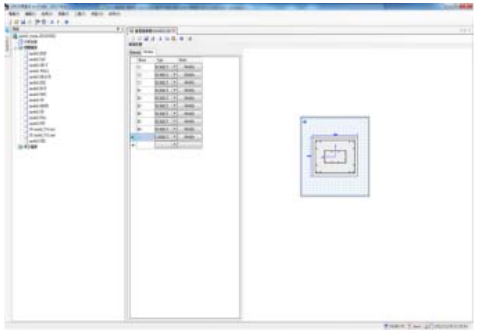
- 內政部營建署97年3月審核通過。
- 修正ATC-40所提出之容量震譜法(或稱PushOver Method)觀念，提出改良式建築物耐震能力評估方法。
- 考量材料非線性組成率，由考量彎矩-剪力互制行為建立塑鉸，理論背景完整，並已與實驗比對驗證。
- 可提供ETABS(V8.4.8與V9.0以上之版本)與MIDAS GEN(V761與V820)執行PUSHOVER分析。(持續銜接最新版)
- 考量地震力作用下，軸力變化對構件性能之影響。
- 應用上無樓層數限制。
- 操作全視窗化，易操作，易檢核分析後結果。
- 開闢專屬網頁供使用者免費下載軟體，並設立Q & A專欄作為各界意見溝通之平台。定期(每年元旦)與不定期通知使用者最新資訊。專屬網站：http://sercb.dyndns.org/SERCBWeb/Default.aspx

結論

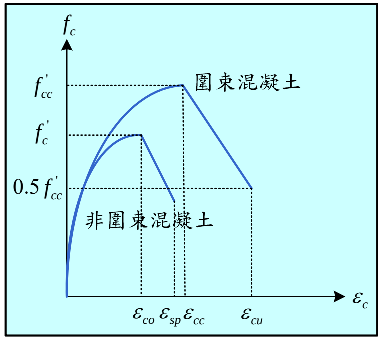


本研究建立補強後構件非線性行為的分析方法，並擴充於SERCBWin2013中，以作為工程師補強分析的工具，提升國內補強分析之效率及精確性。

目前針對各補強工法所完成之**分析斷面型式共計10餘種**，且提供使用者於補強分析時有更**簡便的輸入視窗介面**。



結論



提供Kawashima及Mander兩種混凝土組成律；在鋼筋部份，提供完全彈塑性之組成律，但在同一斷面如有不同強度之鋼筋，則可定義不同組成律參數。

現場進行翼牆補強施工時，植筋效果不容易掌控，因此建議工程師在分析時可保守的忽略拉力側翼牆主筋貢獻，而目前程式也提供此選項的分析功能。

Thanks for your attention