公路橋梁耐震能力評估與補強講習會
—橋梁耐震能力詳細評估方法介紹—

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報告內容大綱

① 前言
② 改良式耐震能力評估法
③ RC橋柱塑性鉸之分析與設定
④ 側推分析與橋柱反覆載重實驗之比對
⑤ 非線性動力歷時分析與橋柱擬動態實驗之比對
⑥ 結構分析模式建立之注意要項
⑦ 耐震性能評估流程
⑧ 耐震性能是否合格之判別
⑨ 結論

一、前言

結構非線性分析方法

靜態側推分析
（Pushover Analysis）

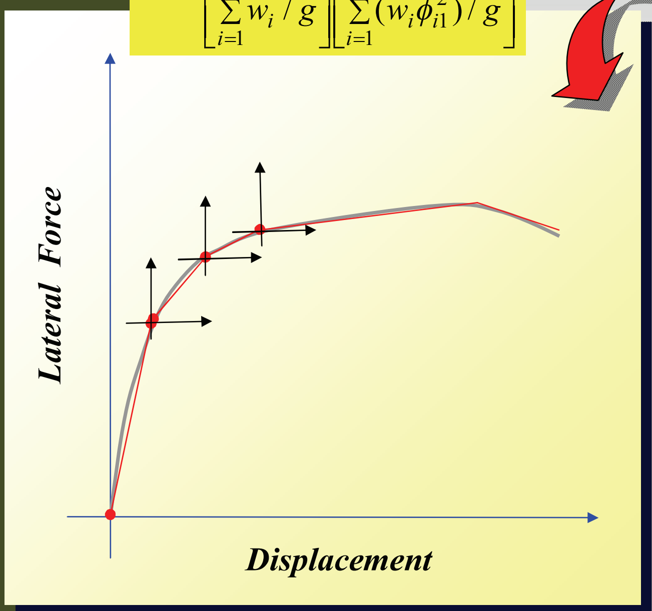
能清楚地提供結構物在各性能階段的受力與變形行為

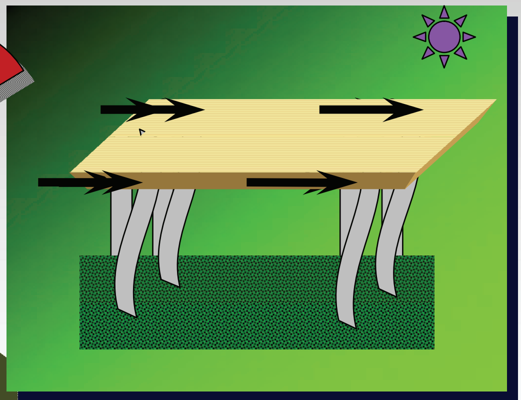
動態非線性歷時分析
（Nonlinear Time History Analysis）

顯現結構在振動過程中所有的動態反應，但因分析過程耗時較久，較適用於特殊結構之檢核。

二、改良式耐震能力評估法

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





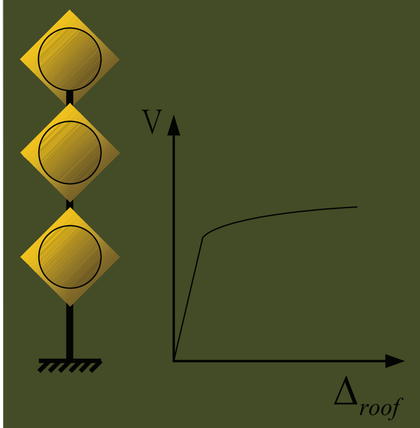
$$PF_1 = \left[\frac{\sum_{i=1}^{N}(w_i\phi_{i1})/g}{\sum_{i=1}^{N}(w_i\phi_{i1}^2)/g}\right]$$

容量震譜法

容量震譜法

1. Transform Capacity Curve to Capacity Spectrum

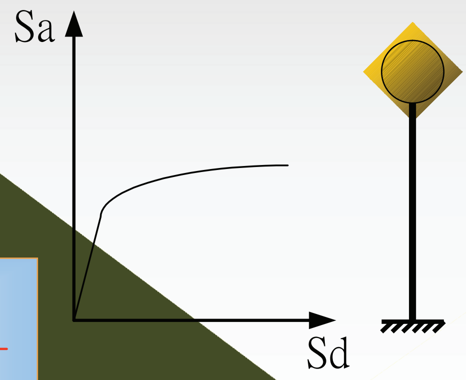
(Capacity Curve)



$$S_a = \frac{V}{\alpha_1 W}$$

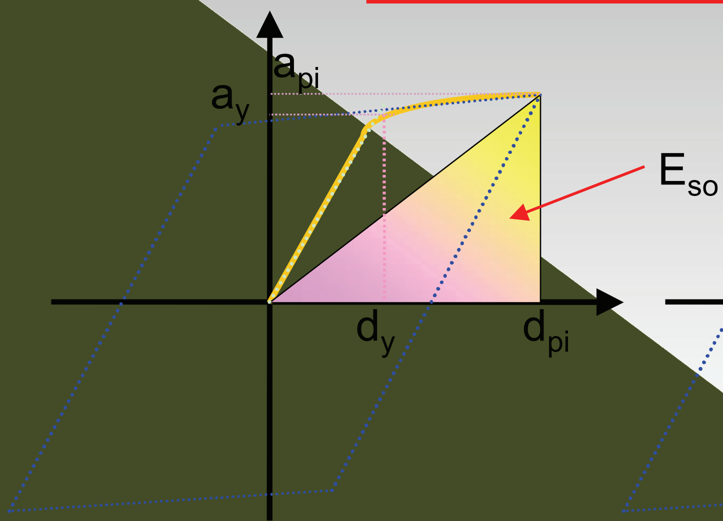
$$S_d = \frac{\Delta_{roof}}{PF_1 \phi_{roof,1}}$$

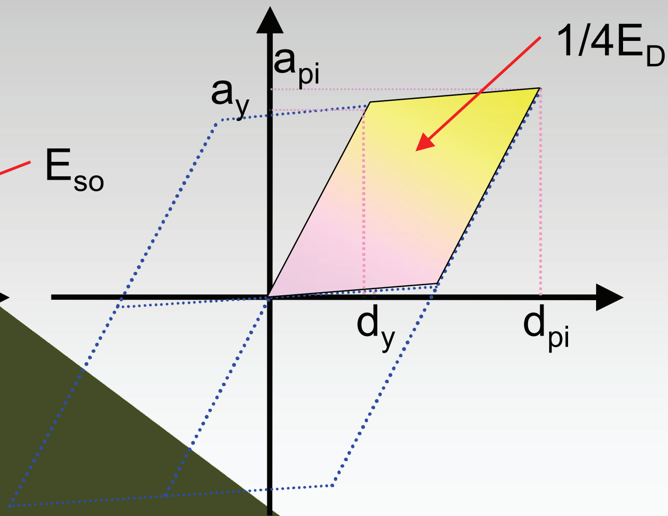
(Capacity Spectrum)



容量震譜法

2. Damping calculation





$$\beta_0 = \frac{1}{4\pi} \cdot \frac{E_D}{E_{SO}} = \frac{0.637\left(a_y d_{pi} - d_y a_{pi}\right)}{a_{pi} d_{pi}}$$

$$\beta_{eff} = \kappa \beta_0 + \beta_{basic}$$

ATC-40結構性能耐震能力評估法簡介

Spectral amplification factor

$$AF_A(\beta_{eff}) = \frac{PSA}{PGA} = 3.21 - 0.68 \cdot \ln(\beta_{eff})$$

$$AF_V(\beta_{eff}) = \frac{PSV}{PGV} = 2.31 - 0.41 \cdot \ln(\beta_{eff})$$

$$AF_D(\beta_{eff}) = \frac{PSD}{PGD} = 1.82 - 0.27 \cdot \ln(\beta_{eff})$$

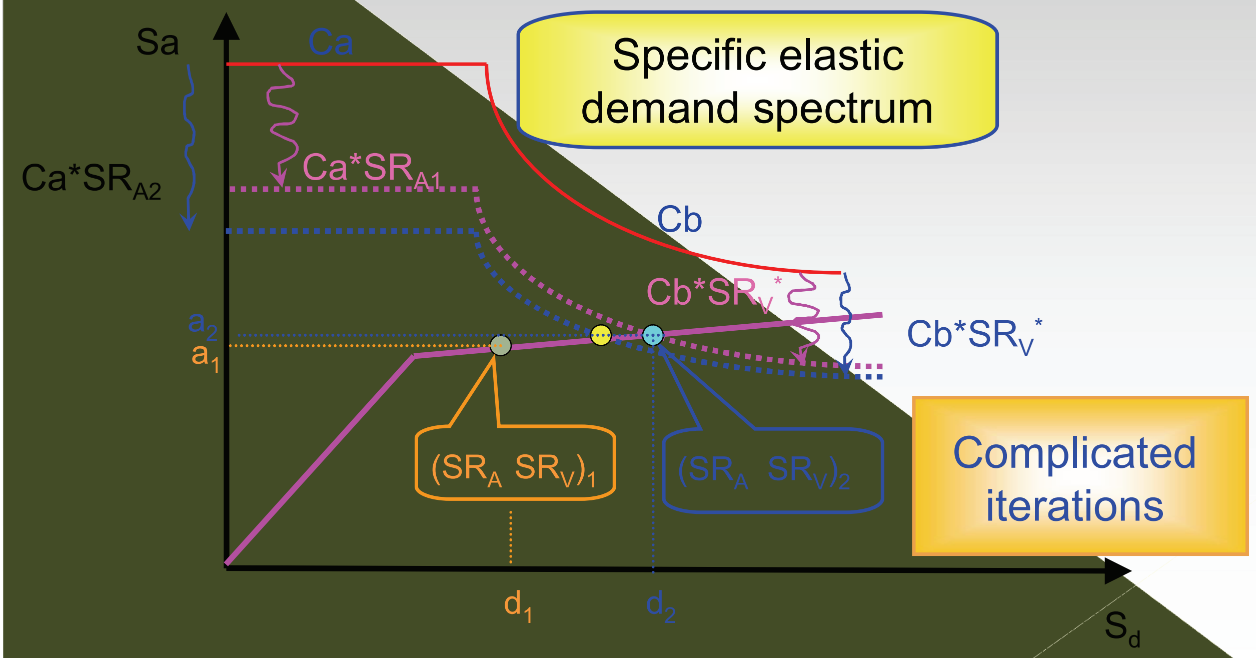
Spectral reduction factor

$$SR_A = \frac{AF_A(\beta_{eff})}{AF_A(\beta_{code})} = \frac{3.21 - 0.68 \cdot \ln(\beta_{eff})}{3.21 - 0.68 \cdot \ln(\beta_{code})}$$

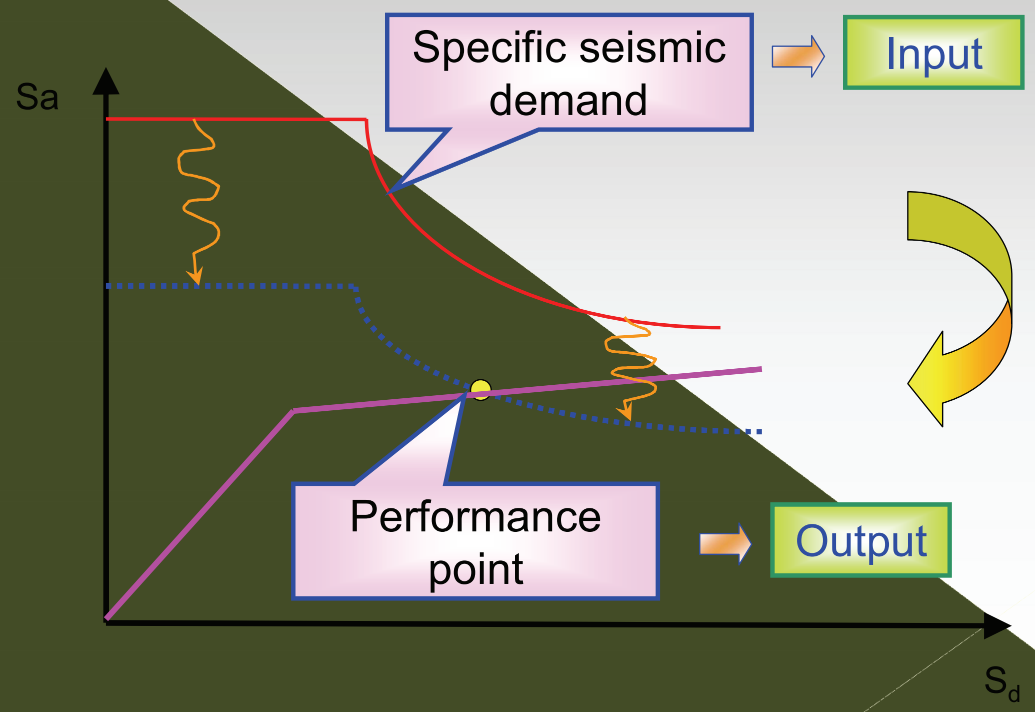
$$SR_V = \frac{AF_V(\beta_{eff})}{AF_V(\beta_{code})} = \frac{2.31 - 0.41 \cdot \ln(\beta_{eff})}{2.31 - 0.41 \cdot \ln(\beta_{code})}$$

$$SR_D = \frac{AF_D(\beta_{eff})}{AF_D(\beta_{code})} = \frac{1.82 - 0.27 \cdot \ln(\beta_{eff})}{1.82 - 0.27 \cdot \ln(\beta_{code})}$$

■Performance point finding

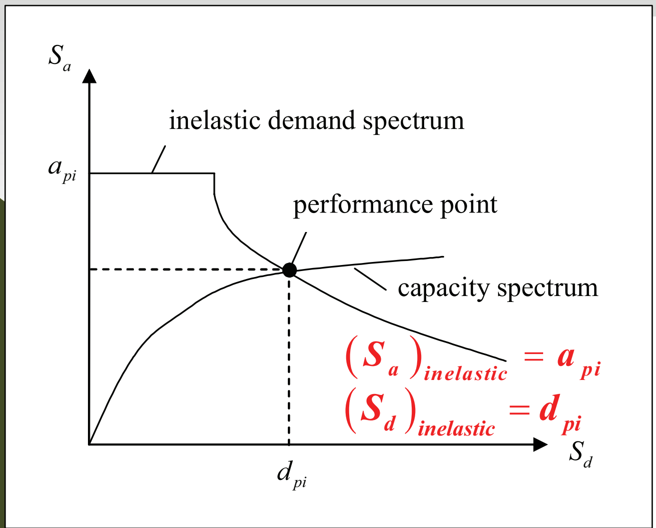


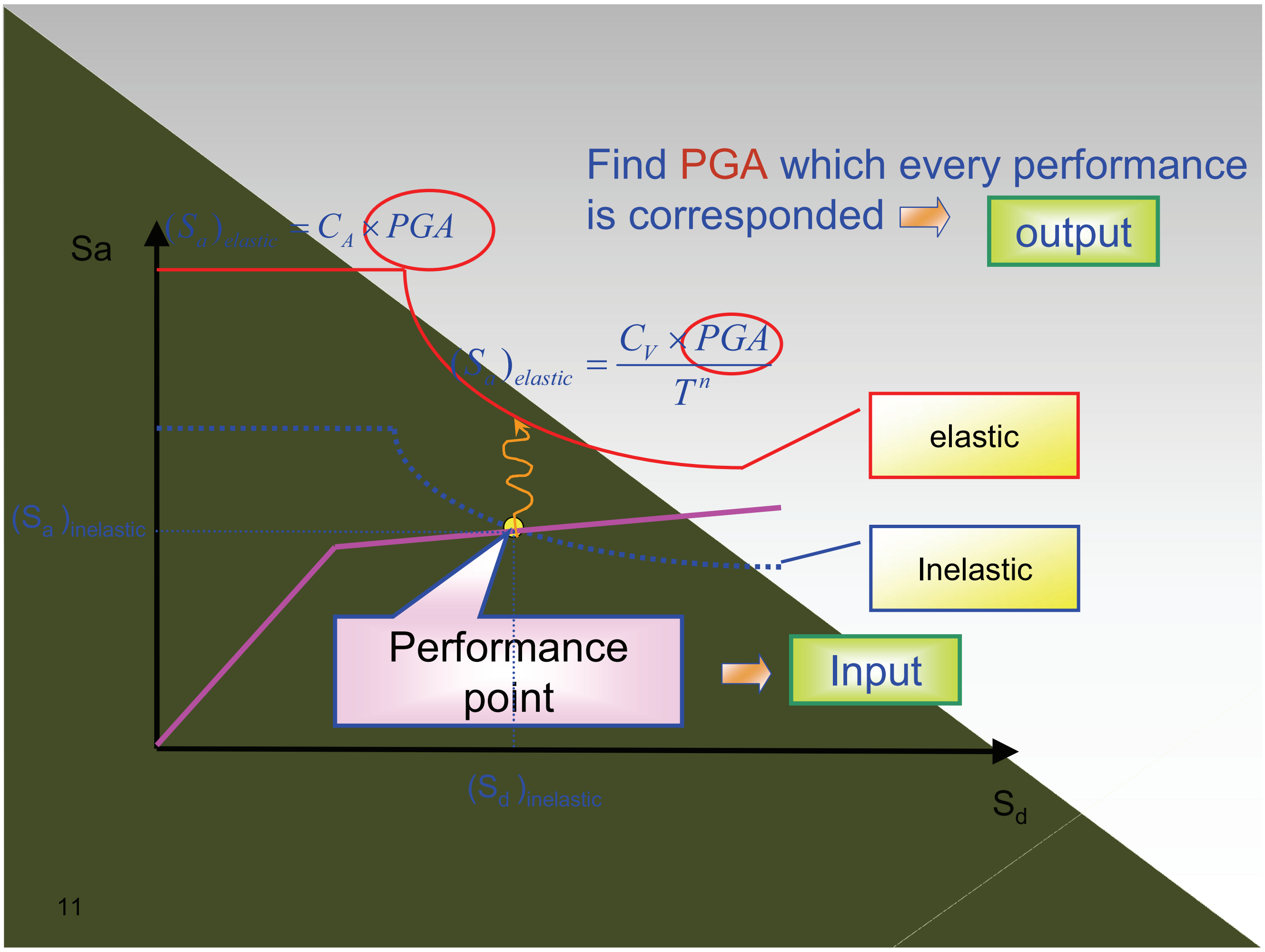
Philosophy of ATC-40

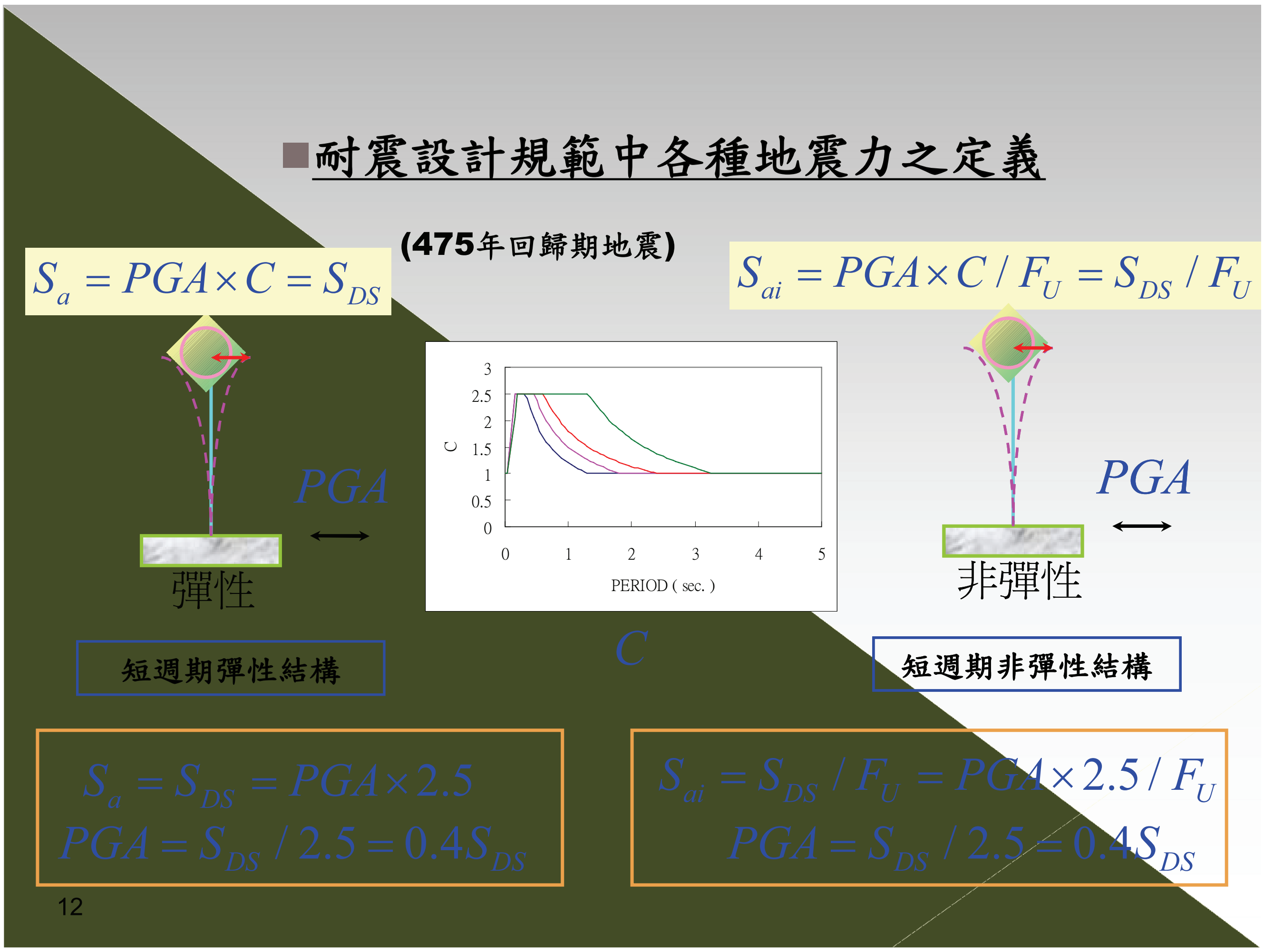


改良式結構性能耐震能力評估方法

性能點(performance point)位於容量譜與非彈性需求譜之交點上，性能點需同時滿足該二者之特性，故容量譜上之譜加速度a_{pi}與譜位移d_{pi}等於非彈性需求譜上的譜加速度$(S_a)_{inelastic}$與譜位移$(S_d)_{inelastic}$。




Find PGA which every performance is corresponded
output
$(S_a)_{elastic} = C_A \times PGA$
Sa
$(S_a)_{elastic} = \frac{C_V \times PGA}{T^n}$
elastic
$(S_a)_{inelastic}$
Inelastic
Performance point
Input
$(S_d)_{inelastic}$
S_d
11


耐震設計規範中各種地震力之定義
(475年回歸期地震)
$S_a = PGA \times C = S_{DS}$
$S_{ai} = PGA \times C / F_U = S_{DS} / F_U$
PGA
彈性
3
2.5
2
1.5
1
0.5
0
0 1 2 3 4 5
C
PERIOD (sec.)
PGA
非彈性
短週期彈性結構
C
短週期非彈性結構
$S_a = S_{DS} = PGA \times 2.5$
$PGA = S_{DS} / 2.5 = 0.4 S_{DS}$
$S_{ai} = S_{DS} / F_U = PGA \times 2.5 / F_U$
$PGA = S_{DS} / 2.5 = 0.4 S_{DS}$
12

■最大地表加速度(PGA)

非彈性譜加速度與彈性譜加速度之比

$$B_1 \text{ or } B_s = \frac{(S_a)_{elastic}}{(S_a)_{inelastic}}$$

$$(S_a)_{inelastic} = a_{pi} \qquad (S_d)_{inelastic} = d_{pi}$$

側推分析所得容量震譜中任何一處譜加速度 a_{pi}

對應的最大地表加速度(PGA)與所在工址之地震需求特性(如 S_{aD} & S_{DS} 等)，橋梁結構物等效振動週期 T、等值黏滯性阻尼 (β_{eff}) 與阻尼比異於5%加速度修正係數 B_1 或 B_s

等有關，可表示如下：

■最大地表加速度(PGA)之計算

一般工址

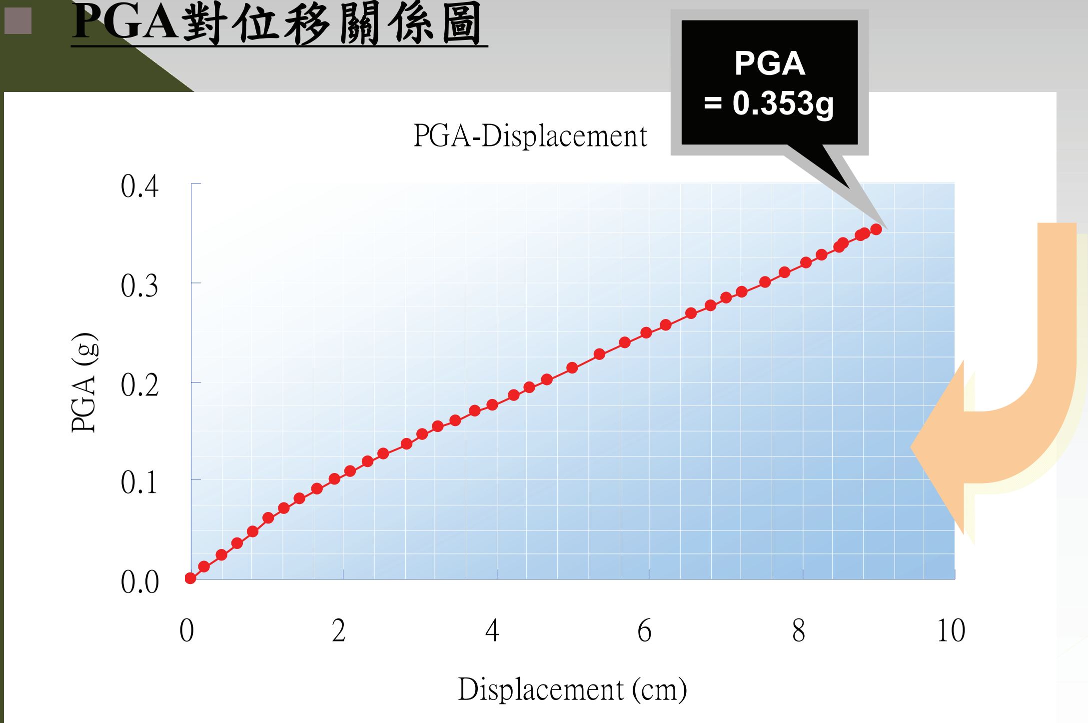
較短及短週期	中週期	長週期
$T \le T_0^D \dfrac{B_s(\beta_{eff})}{B_1(\beta_{eff})}$	$T_0^D \dfrac{B_s(\beta_{eff})}{B_1(\beta_{eff})} < T \le 2.5T_0^D \dfrac{B_s(\beta_{eff})}{B_1(\beta_{eff})}$	$2.5T_0^D \dfrac{B_s(\beta_{eff})}{B_1(\beta_{eff})} < T$
$PGA = \dfrac{a_{pi} \times B_s(\beta_{eff})}{(\dfrac{2.5S_{aD}(T)}{S_{DS}})}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

■最大地表加速度(PGA)之計算

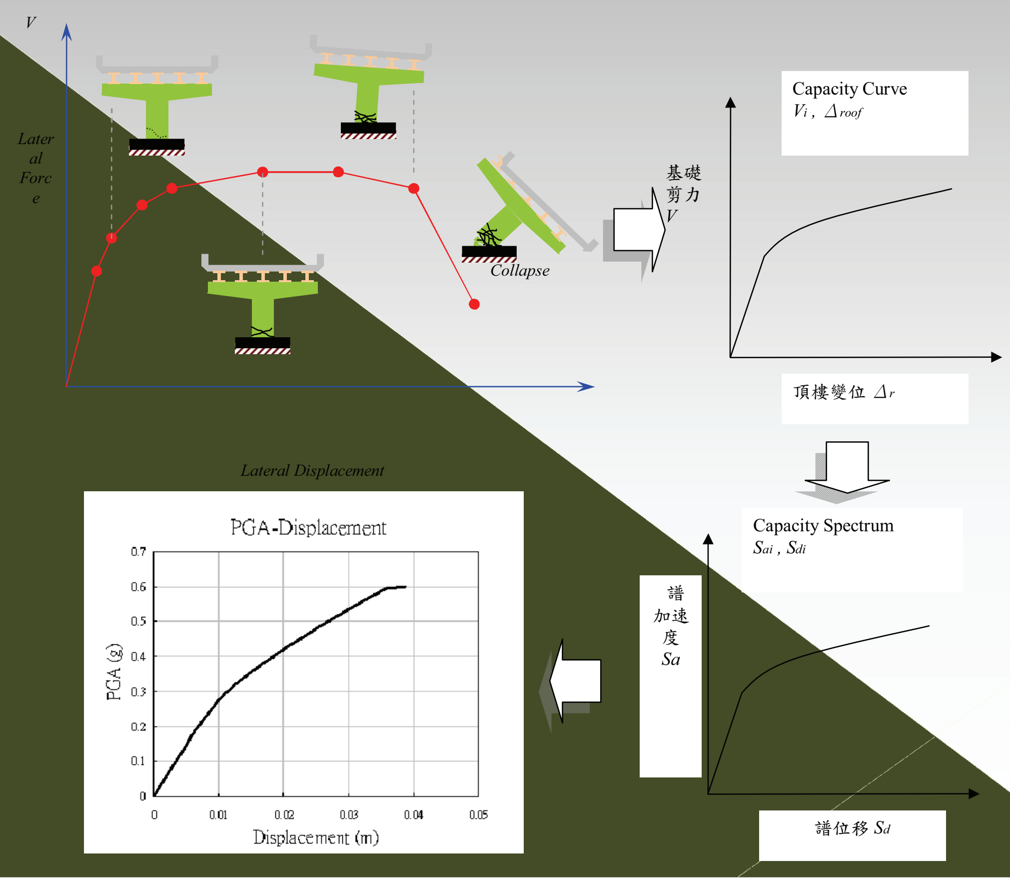
臺北盆地

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

■ PGA對位移關係圖

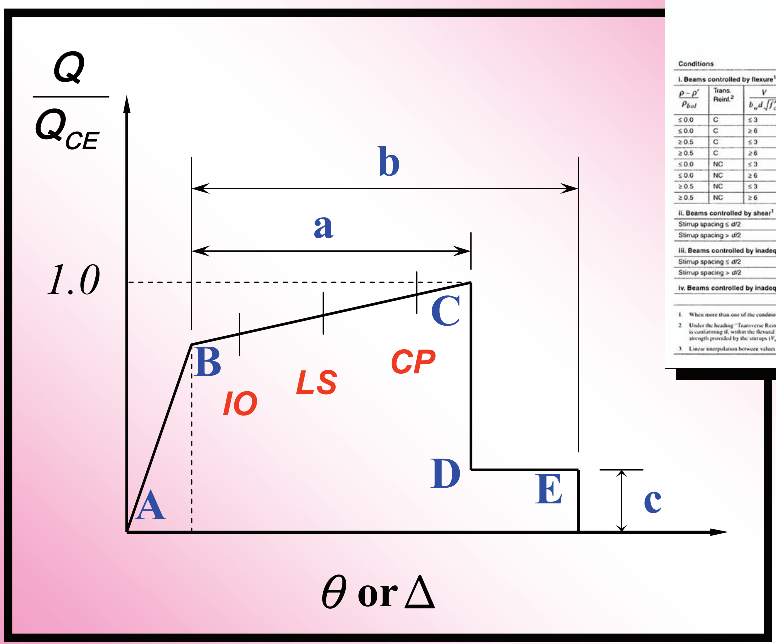


■結構性能耐震評估



三、RC橋柱塑性鉸之分析與設定

■FEMA-273 有關塑性鉸之設定



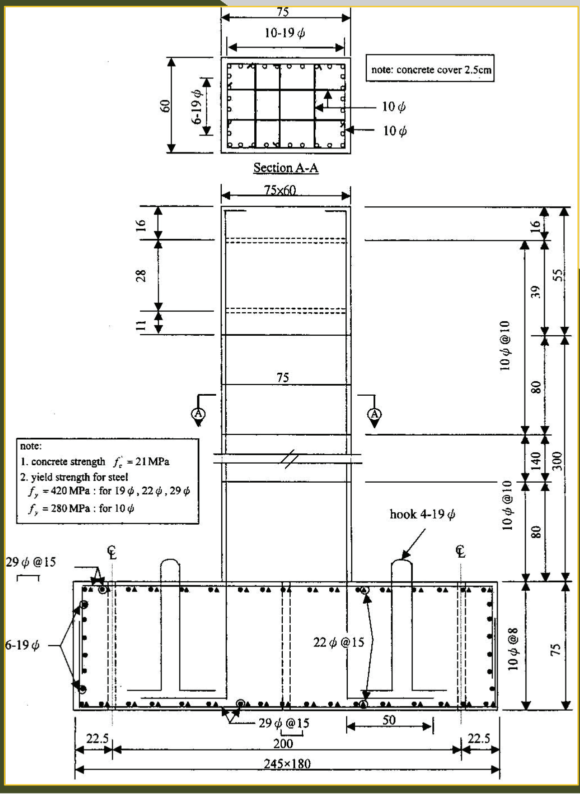
Chapter 6: Concrete
(Systematic Rehabilitation)

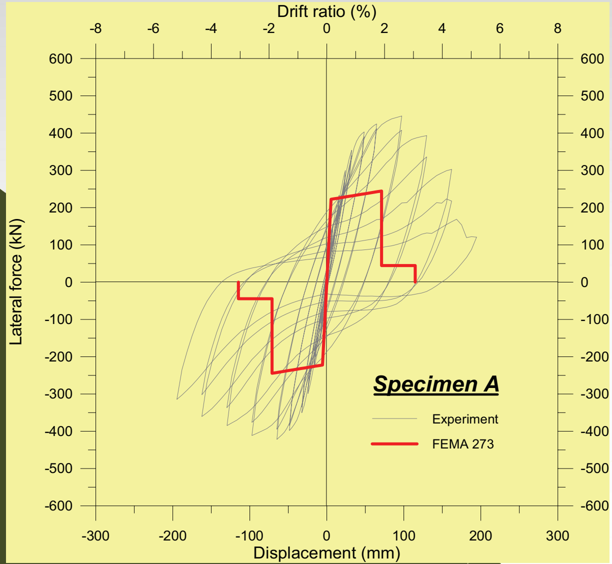
Table 6-6 *Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures—Reinforced Concrete Beams*

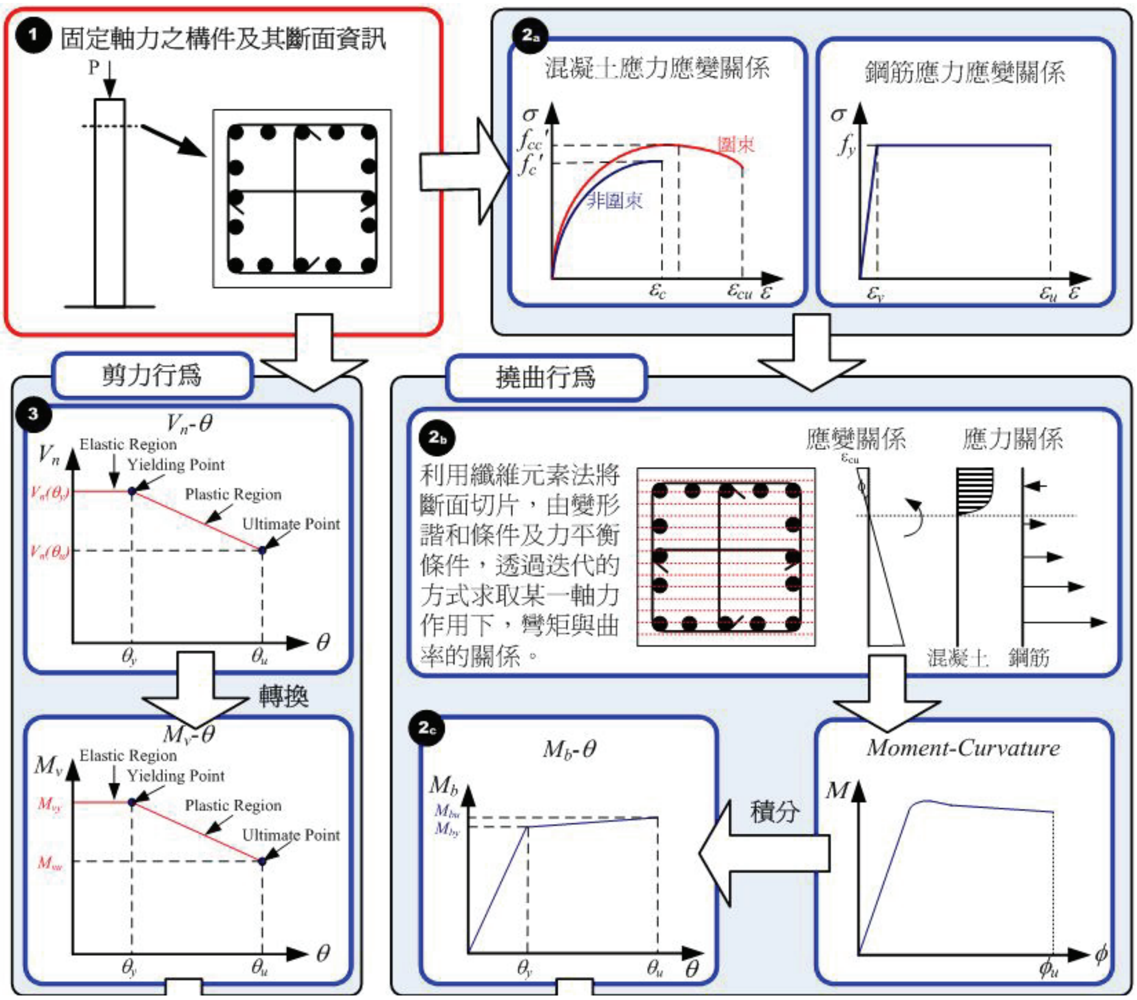
Conditions			Modeling Parameters[3] Plastic Rotation Angle, radians a	b	Residual Strength Ratio c	Acceptance Criteria[3] Plastic Rotation Angle, radians; Component Type Primary, Performance Level IO	LS	CP	Secondary LS	CP
i. Beams controlled by flexure[1]										
$\frac{\rho-\rho'}{\rho_{bal}}$	Trans. Reinf.[2]	$\frac{V}{b_w d\sqrt{f'_c}}$								
≤ 0.0	C	≤ 3	0.025	0.05	0.2	0.005	0.02	0.025	0.02	0.05
≤ 0.0	C	≥ 6	0.02	0.04	0.2	0.005	0.01	0.02	0.02	0.04
≥ 0.5	C	≤ 3	0.02	0.03	0.2	0.005	0.01	0.02	0.02	0.03
≥ 0.5	C	≥ 6	0.015	0.02	0.2	0.005	0.005	0.015	0.015	0.02
≤ 0.0	NC	≤ 3	0.02	0.03	0.2	0.005	0.01	0.02	0.02	0.03
≤ 0.0	NC	≥ 6	0.01	0.015	0.2	0.0	0.005	0.01	0.01	0.015
≥ 0.5	NC	≤ 3	0.01	0.015	0.2	0.005	0.01	0.01	0.01	0.015
≥ 0.5	NC	≥ 6	0.005	0.01	0.2	0.0	0.005	0.005	0.005	0.01
ii. Beams controlled by shear[1]										
Stirrup spacing ≤ d/2			0.0	0.02	0.2	0.0	0.0	0.0	0.01	0.02
Stirrup spacing > d/2			0.0	0.01	0.2	0.0	0.0	0.0	0.005	0.01
iii. Beams controlled by inadequate development or splicing along the span[1]										
Stirrup spacing ≤ d/2			0.0	0.02	0.0	0.0	0.0	0.0	0.01	0.02
Stirrup spacing > d/2			0.0	0.01	0.0	0.0	0.0	0.0	0.005	0.01
iv. Beams controlled by inadequate embedment into beam-column joint[1]										
			0.015	0.03	0.2	0.01	0.01	0.015	0.02	0.03

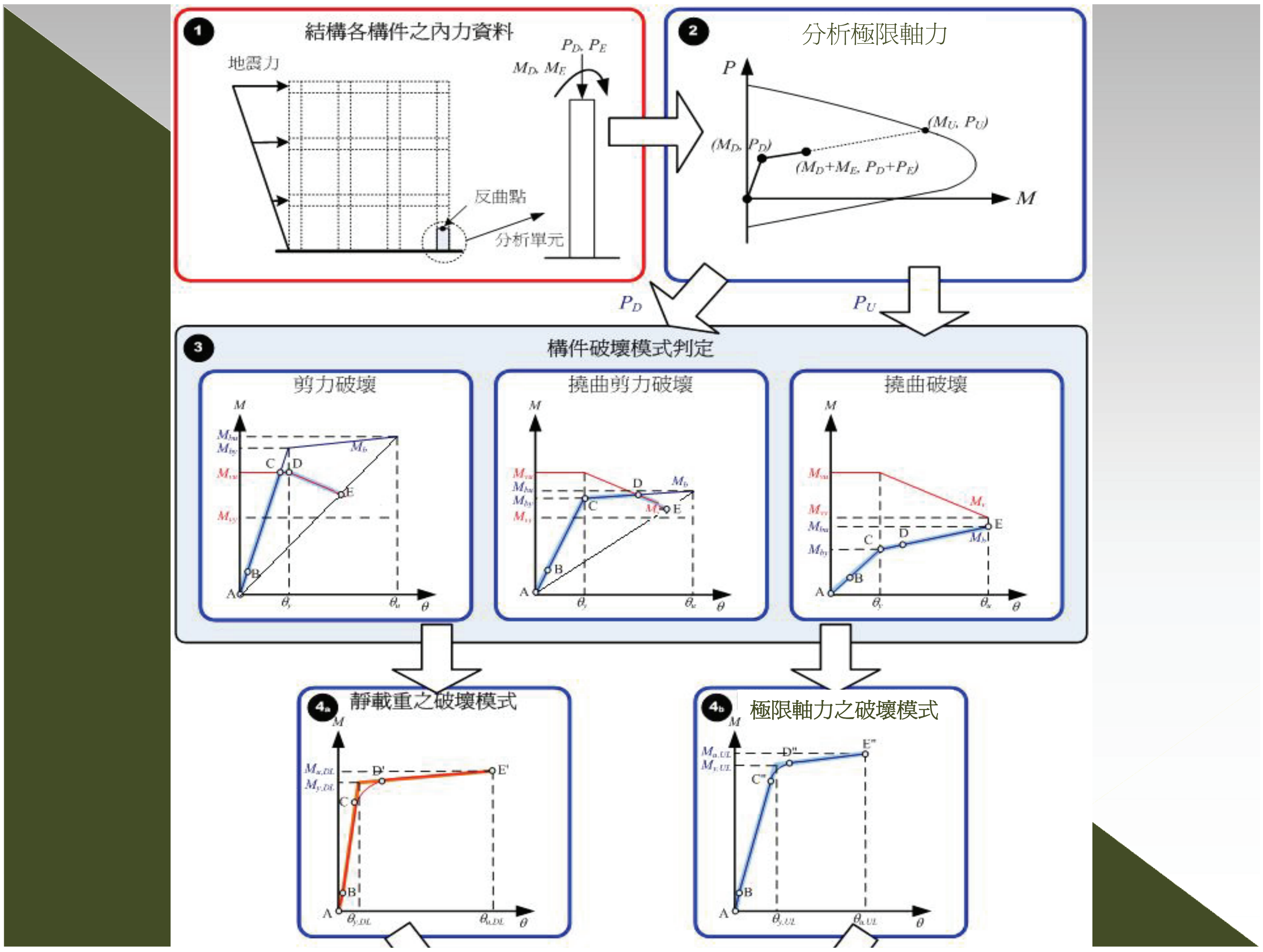
1. When more than one of the conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table.
2. Under the heading "Transverse Reinforcement," "C" and "NC" are abbreviations for conforming and nonconforming details, respectively. A component is conforming if, within the flexural plastic region, closed stirrups are spaced at ≤ d/3, and if, for components of moderate and high ductility demand, the strength provided by the stirrups (V_s) is at least three-fourths of the design shear. Otherwise, the component is considered nonconforming.
3. Linear interpolation between values listed in the table is permitted.

alytical results based on FEMA-273



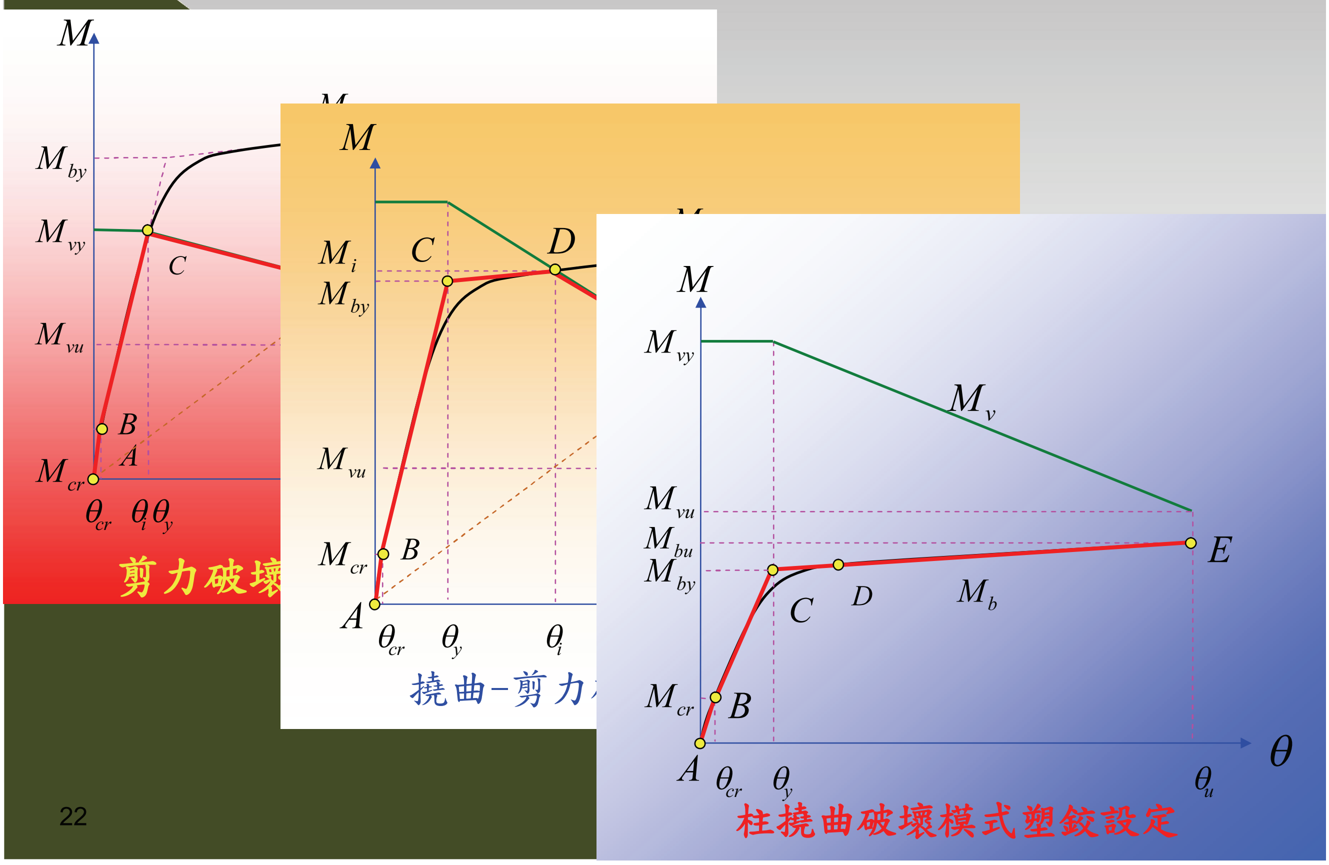






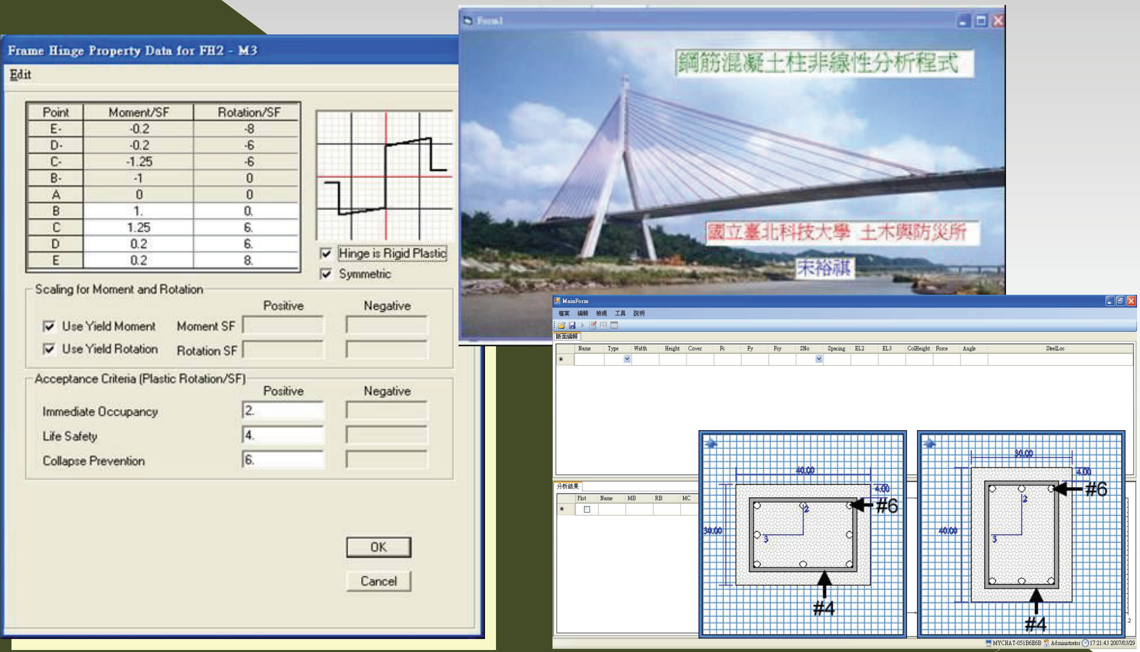
結構各構件之內力資料
地震力
反曲點
分析單元
分析極限軸力
構件破壞模式判定
剪力破壞
撓曲剪力破壞
撓曲破壞
靜載重之破壞模式
極限軸力之破壞模式

塑性鉸設定之建議



剪力破壞
撓曲−剪力
柱撓曲破壞模式塑鉸設定

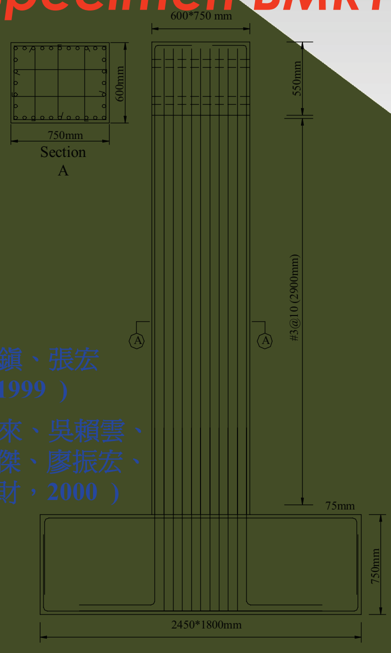
塑性鉸設定



四、側推分析與橋柱反覆載重實驗之比對

圓形與矩形單柱試體

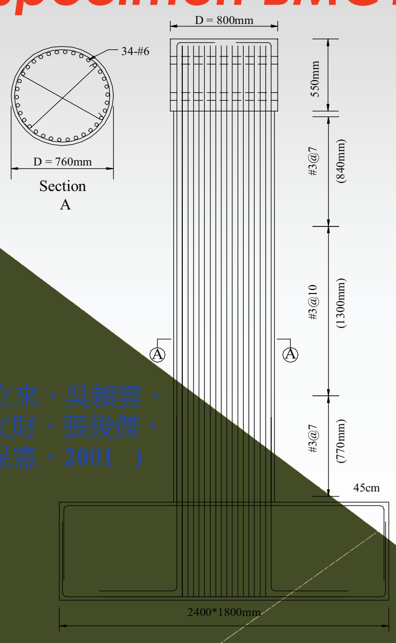
Specimen BMR1-R



（張國鎮、張宏輔，1999）

（鍾立來、吳賴雲、張俊傑、廖振宏、王文財，2000）

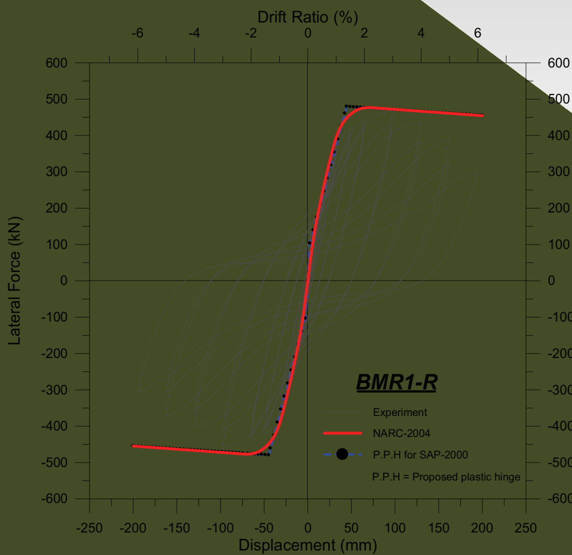
Specimen BMC1



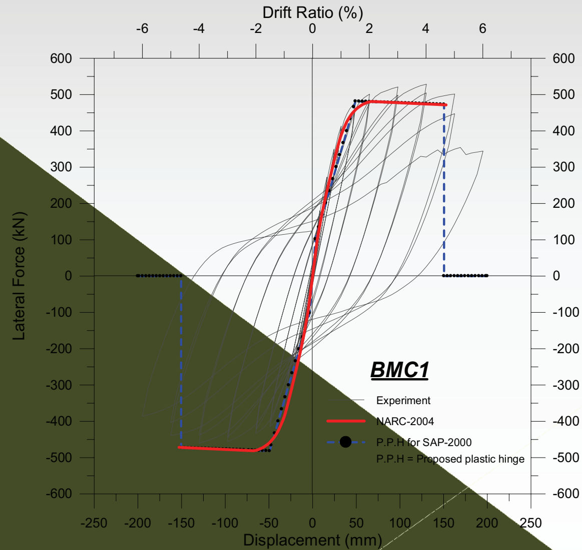
（鍾立來、吳賴雲、王文財、張俊傑、余保憲，2001）

反覆載重試驗與側推分析結果

Specimen BMR1-R

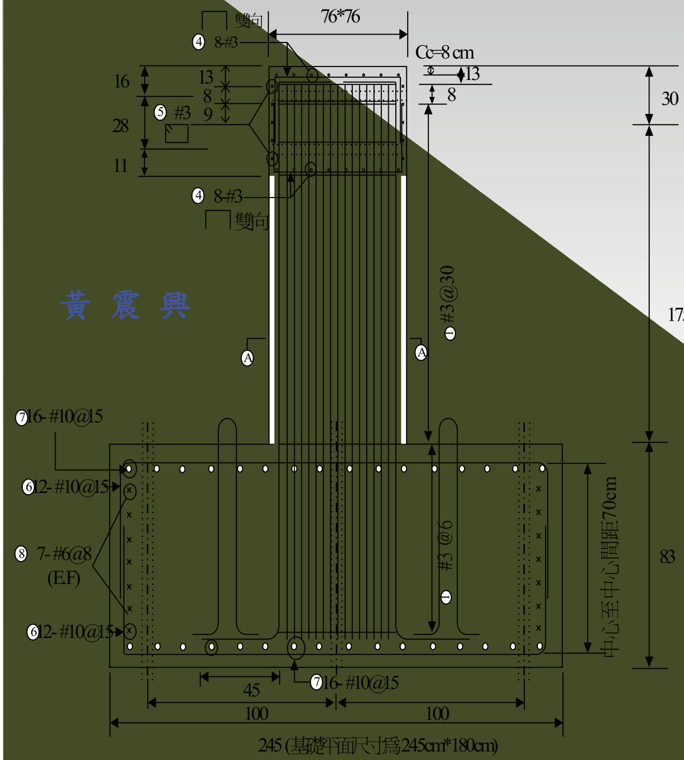


Specimen BMC1



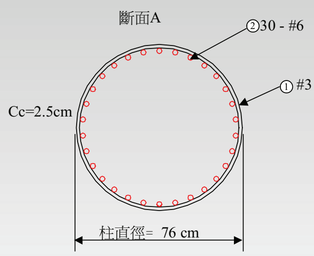
(宋裕祺、劉光晏、蘇進國、蔡益超、張國鎮，2004年)

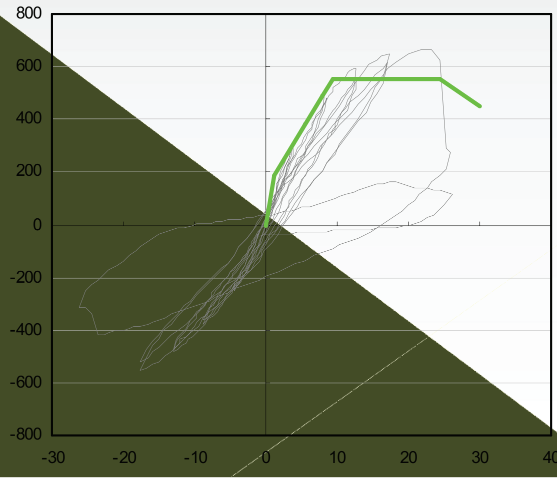
圓形單柱試體–剪力破壞



黃 震 興

BMCS





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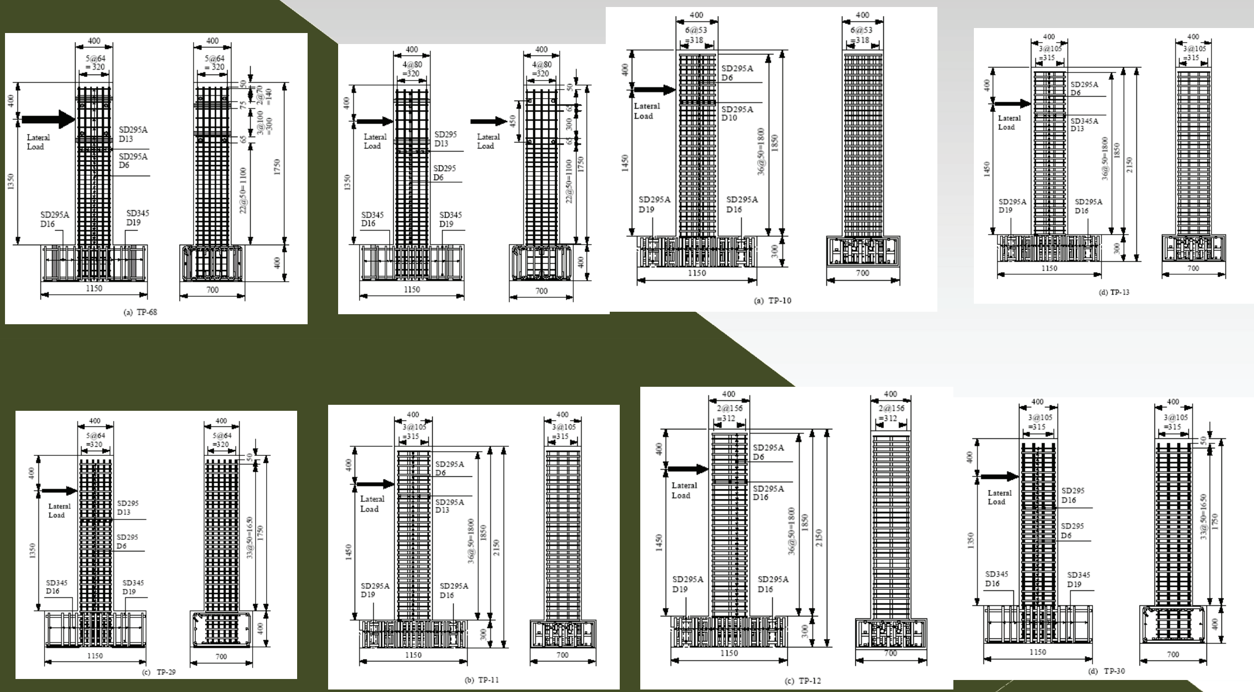
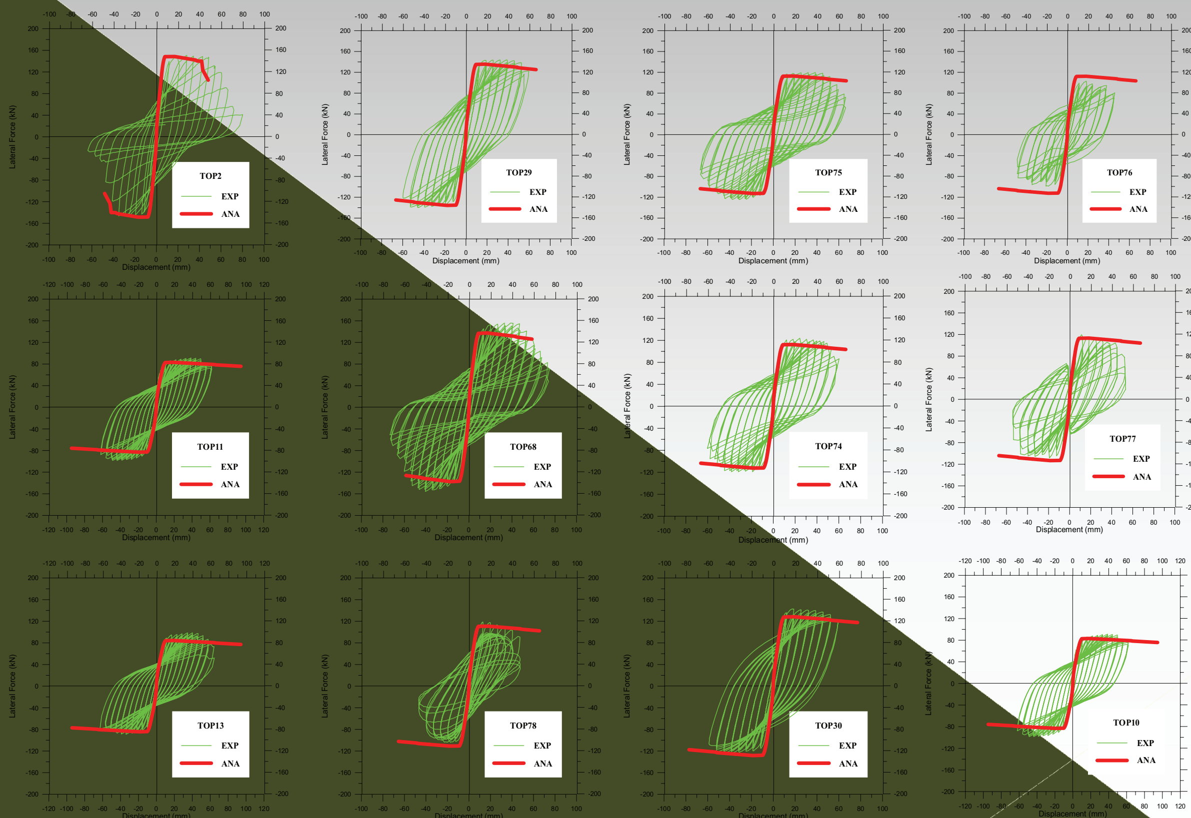
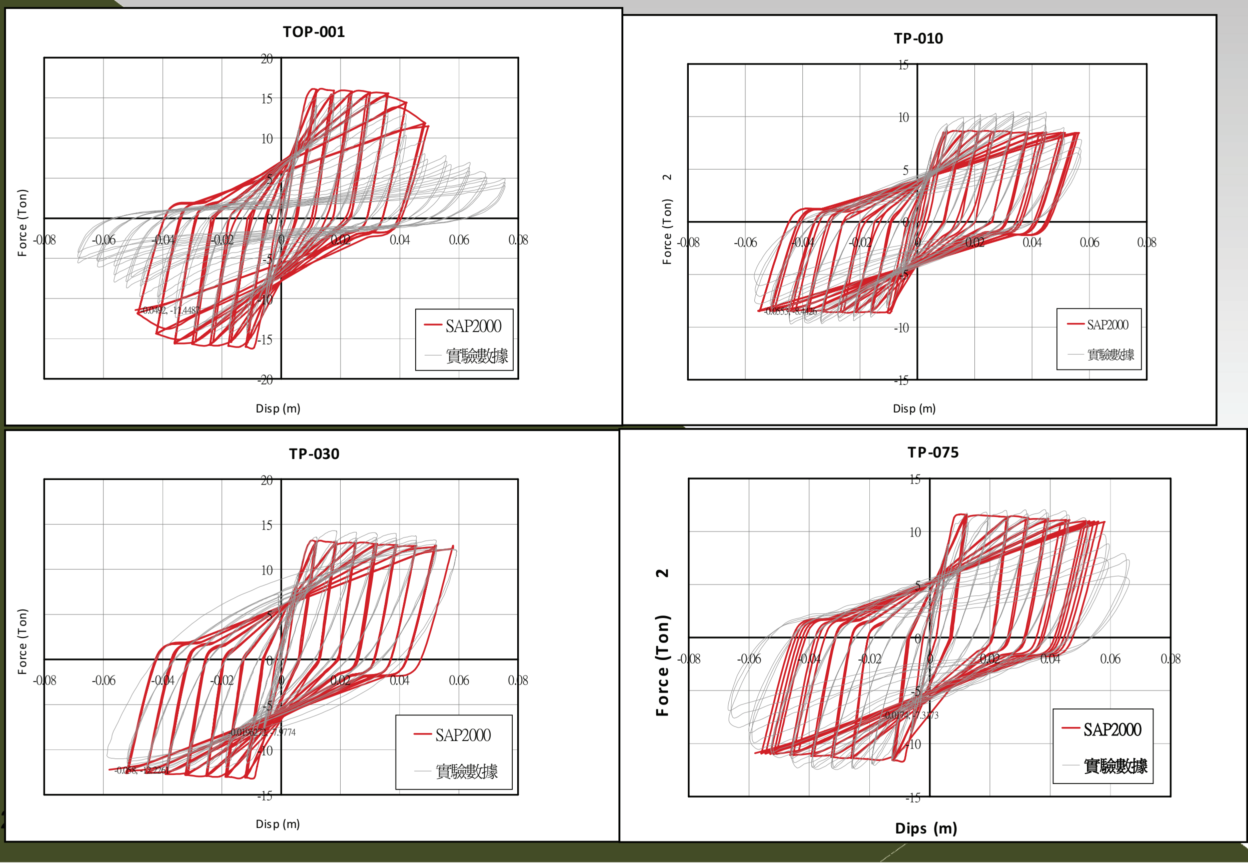


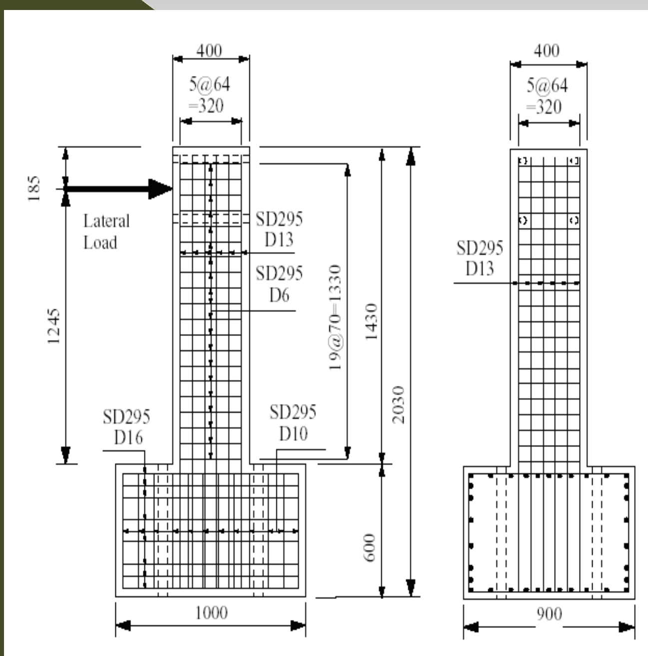
Pushover Analysis Results of Columns

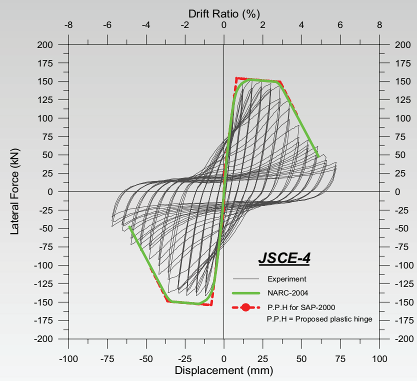


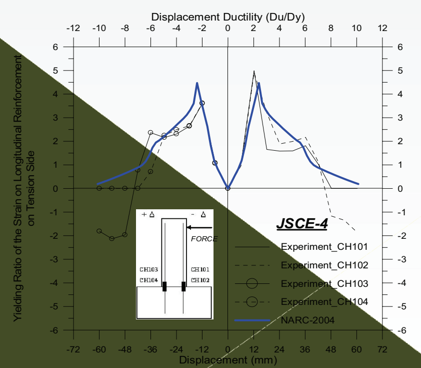
RC柱遲滯迴圈之分析與比較



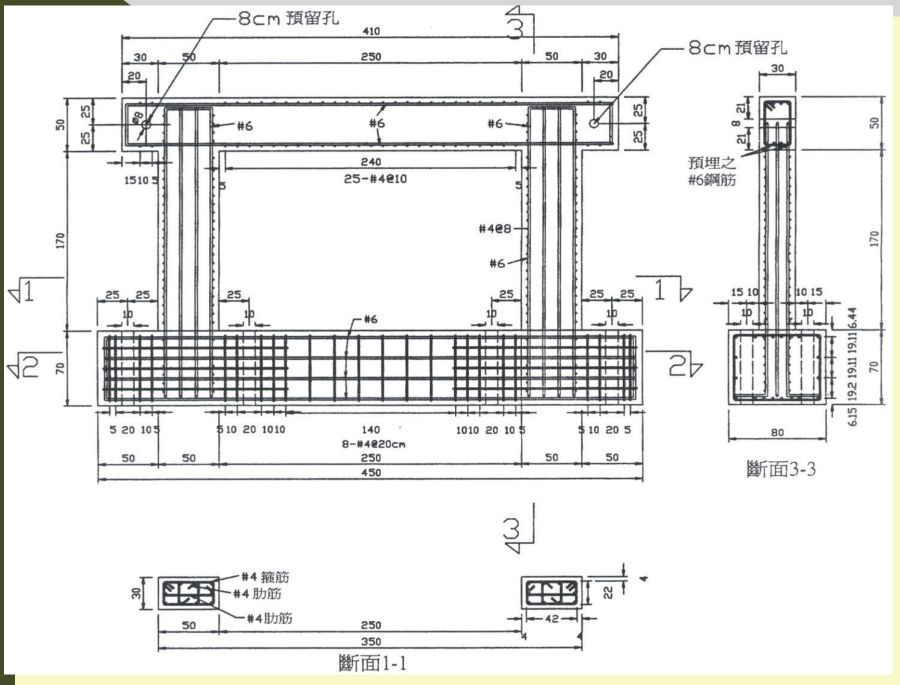
日本土木學會JSCE-4試體



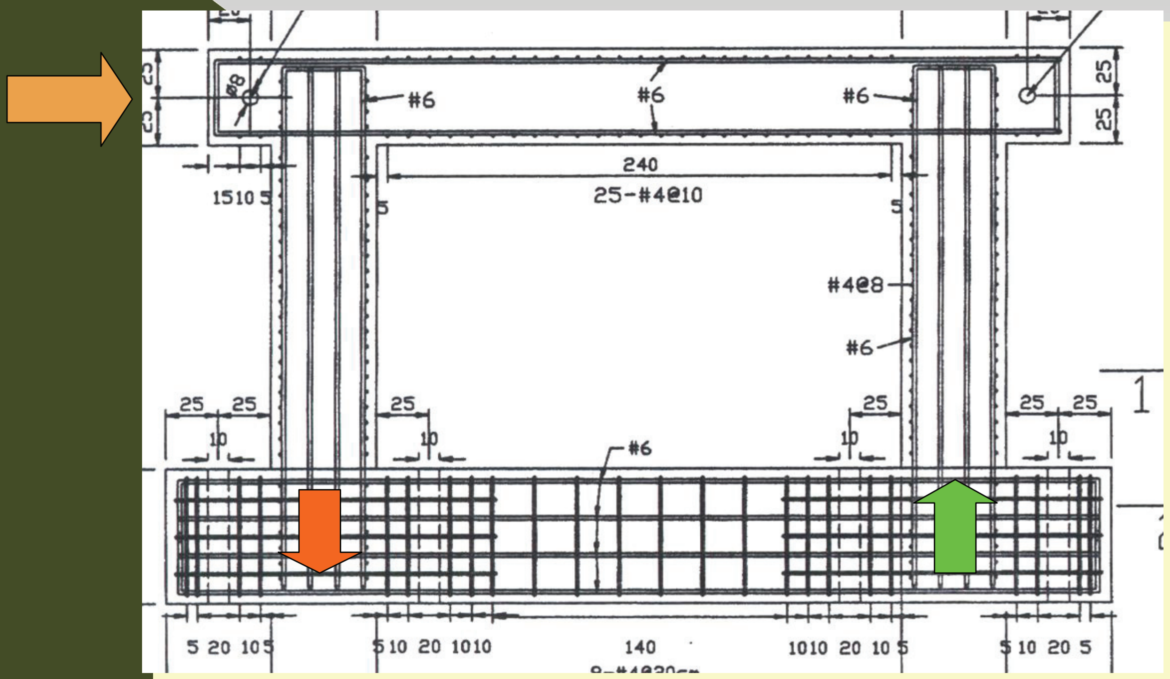




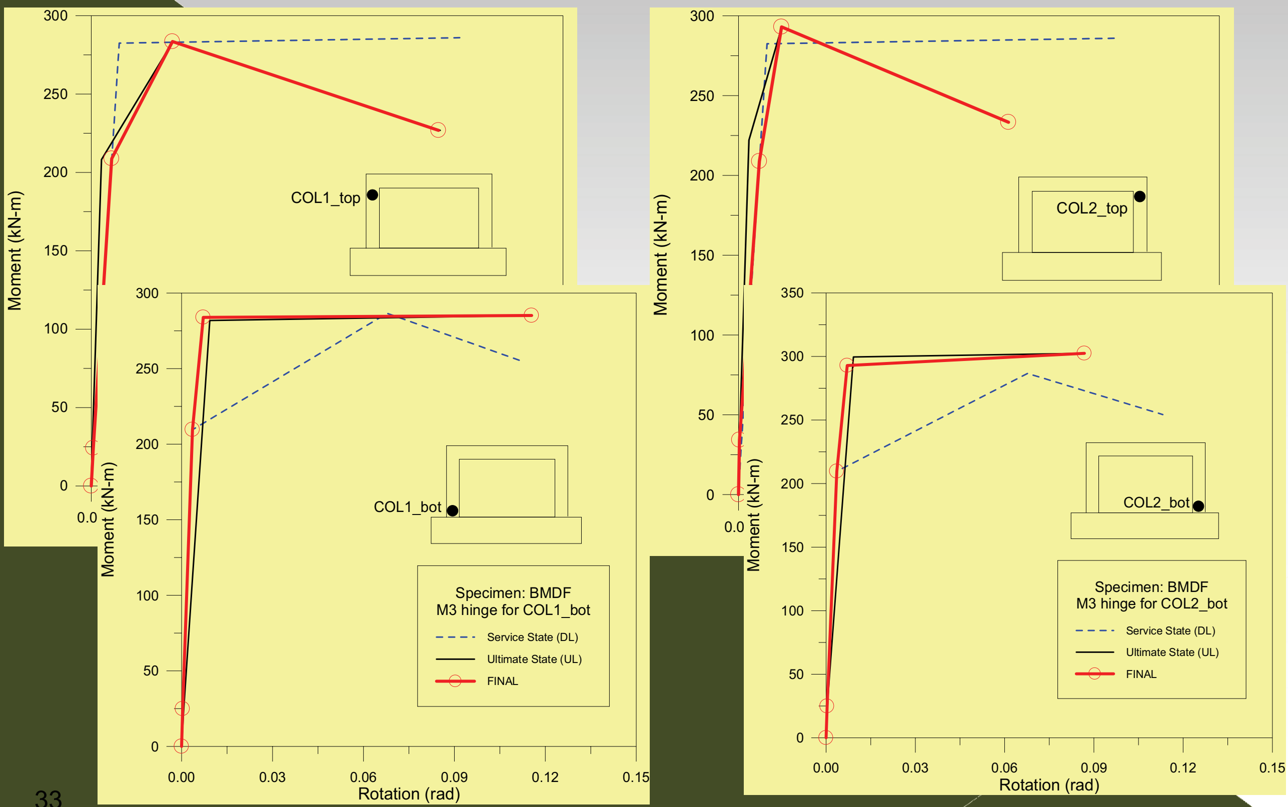
■國家地震工程研究中心BMDF試體



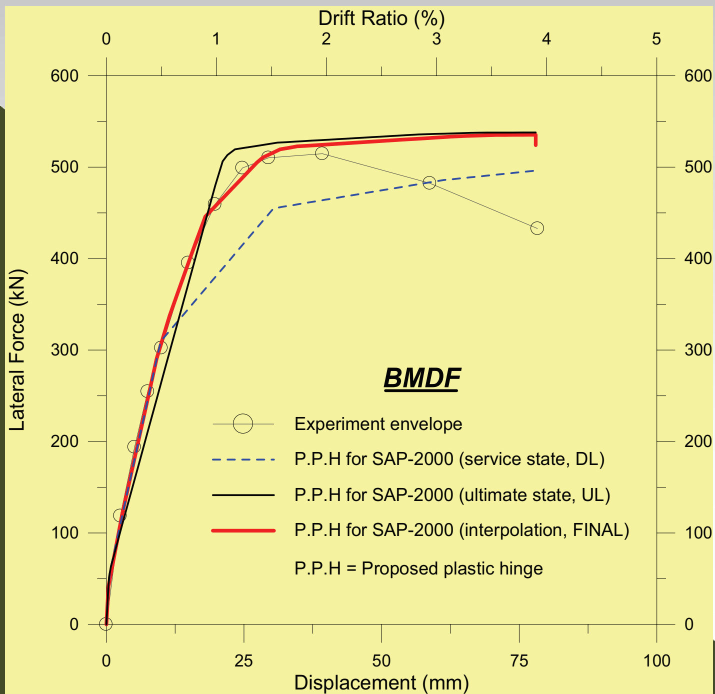
Frame Structure –varying axial force



Plastic Hinge

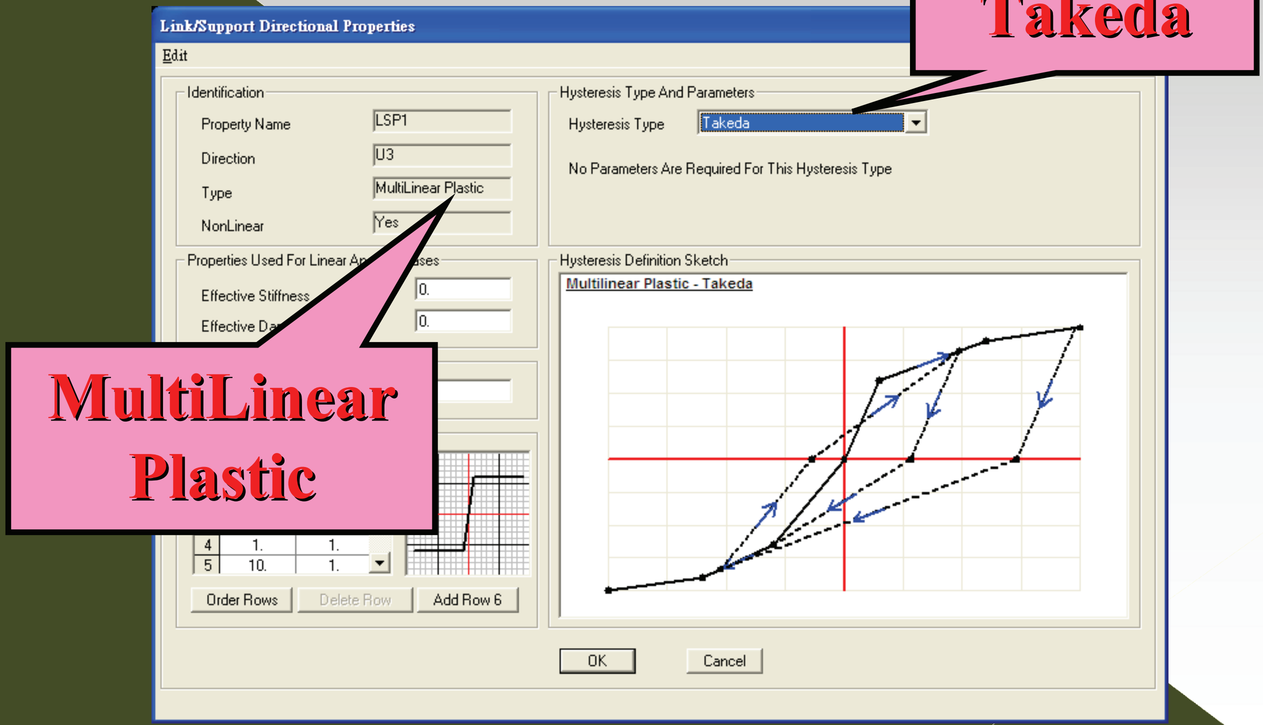


Analysis Results

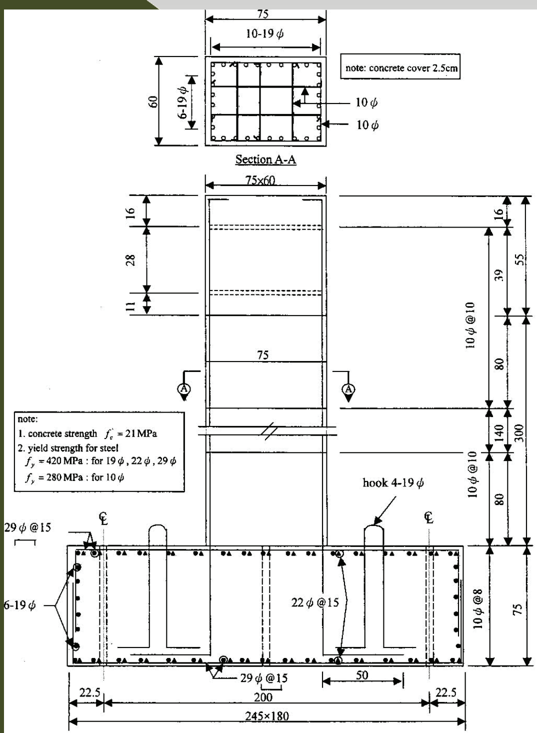


五、非線性動力歷時分析與橋柱擬動態實驗之比對

Time history analysis

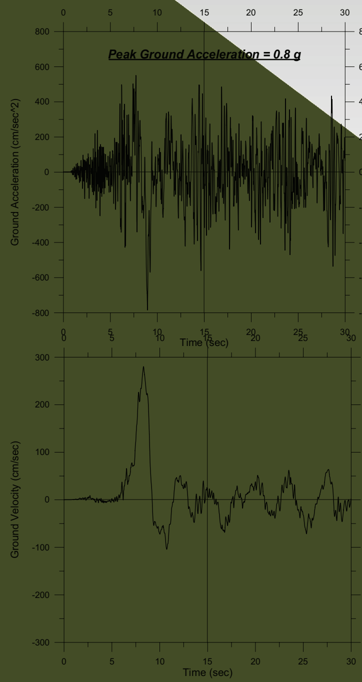


Pseudodynamic test of reinforced concrete columns

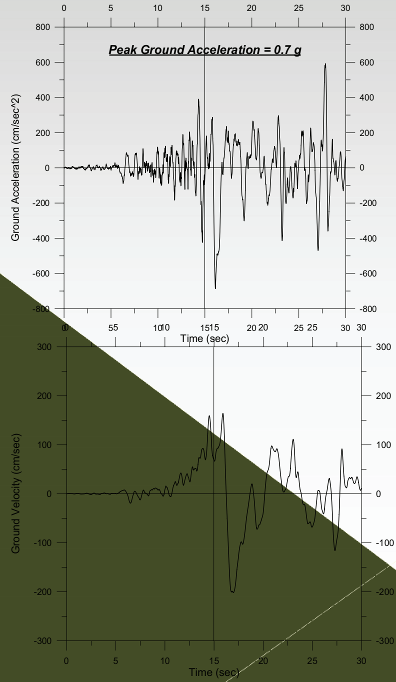


Ground acceleration and velocity for TCU075、TCU102

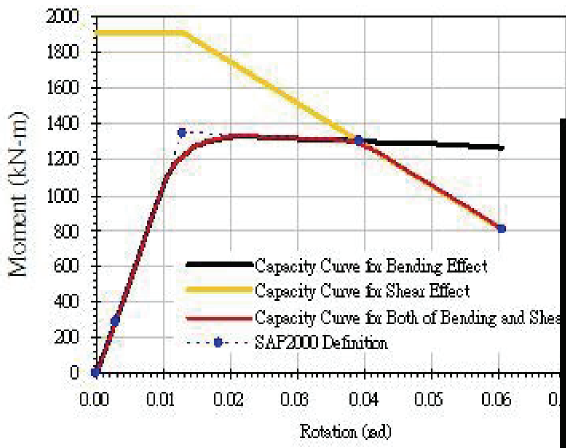
Ground acceleration and velocity for TCU075

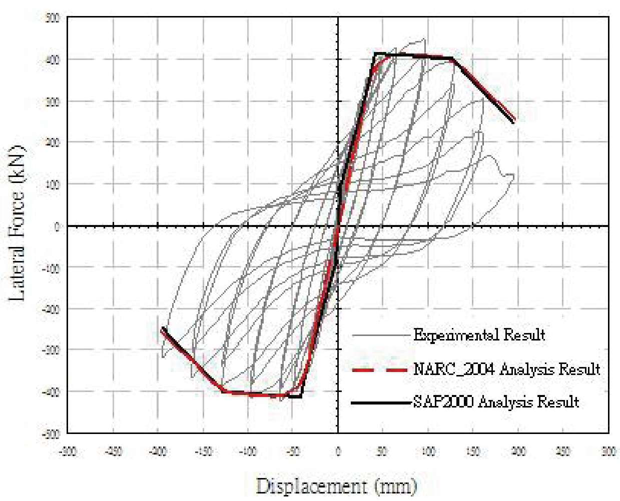


Ground acceleration and velocity for TCU102



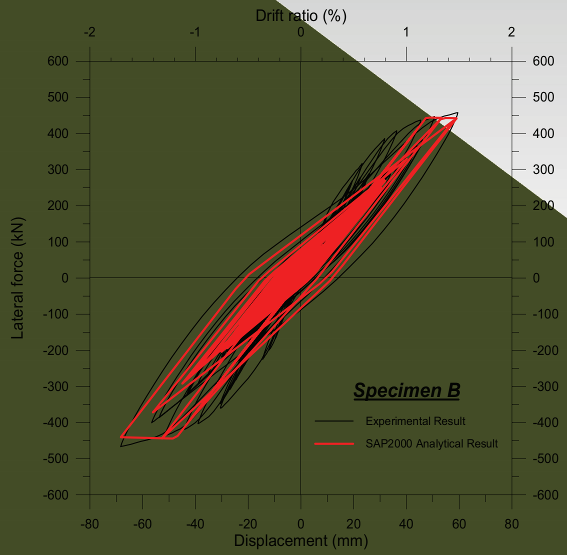
Specimen A – cyclic loading test and analysis result

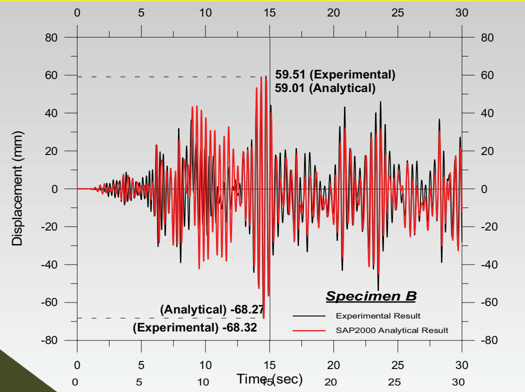


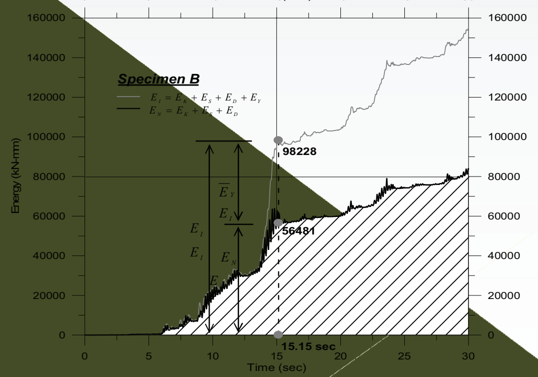


Specimen B –

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

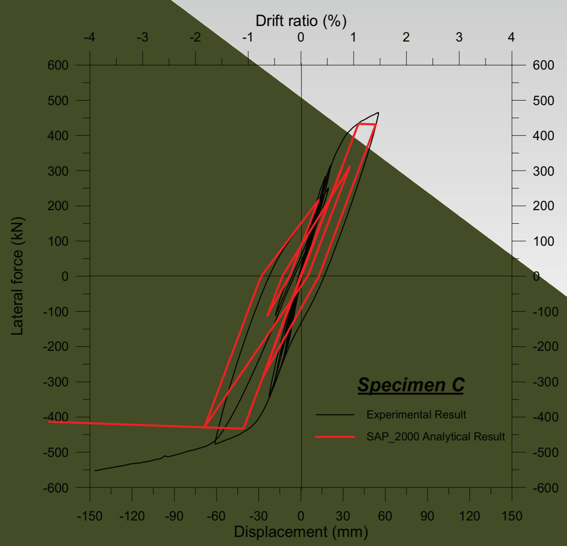


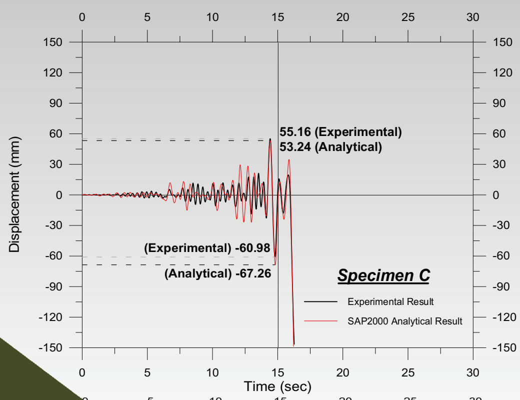


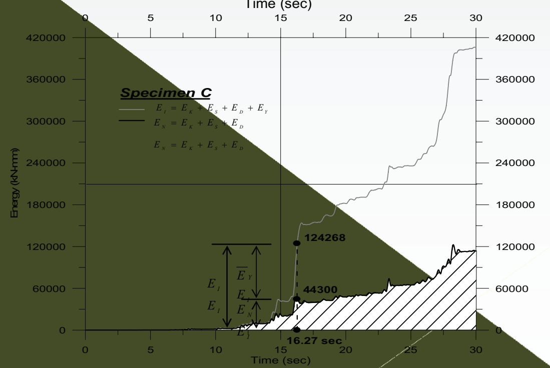


Specimen C –

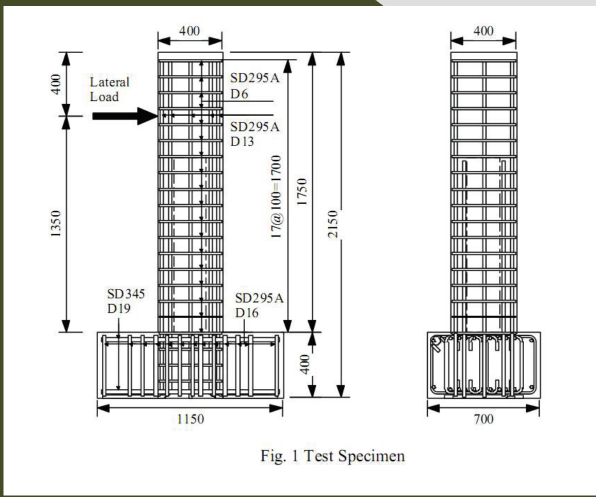
Pseudodynamic and analysis results for as-built Specimen subject to TCU102

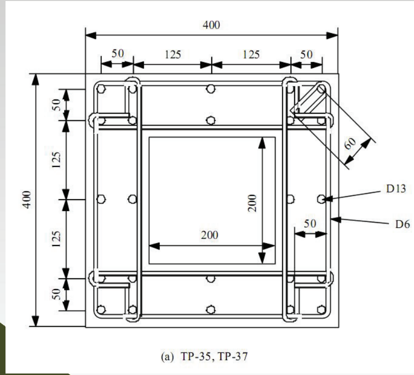




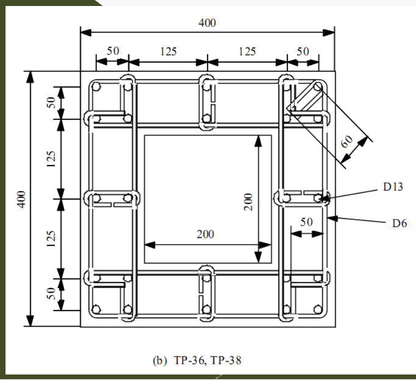


中空矩形單柱試體





TP35

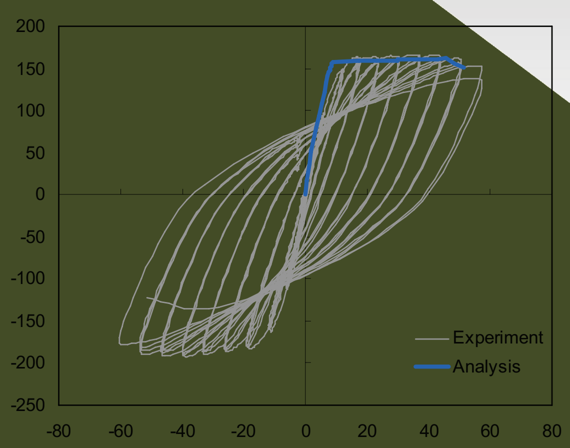


TP36

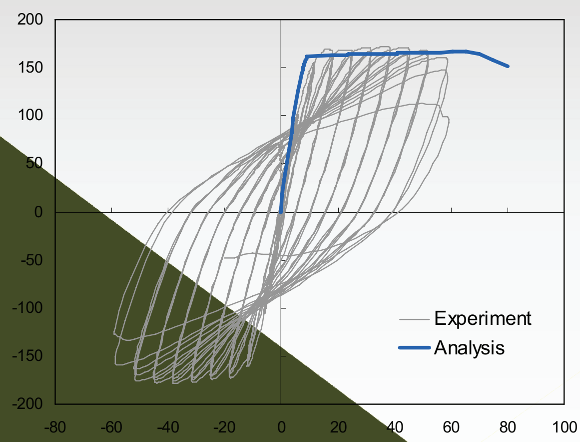
Kawashima

反覆載重試驗與側推分析結果

TP35

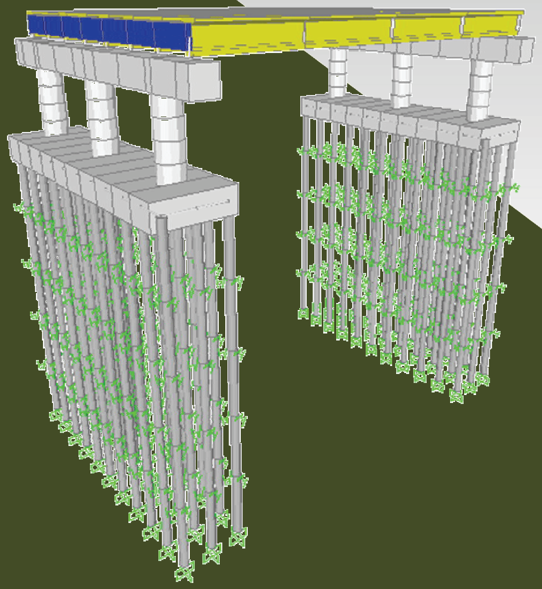


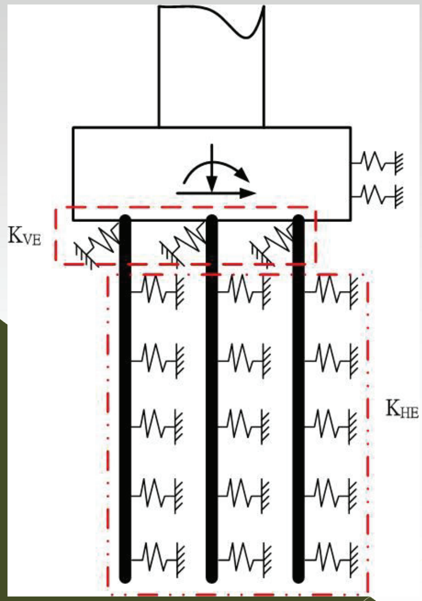
TP36



六、結構分析模式建立之注意要項

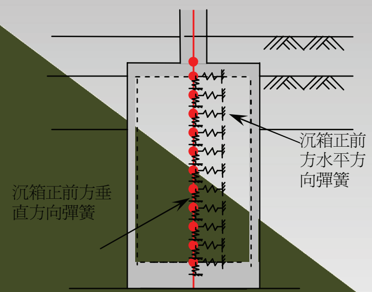
1.橋墩含樁基礎裸露或土壤液化之耐震能力評估

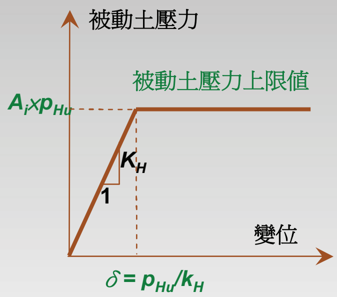




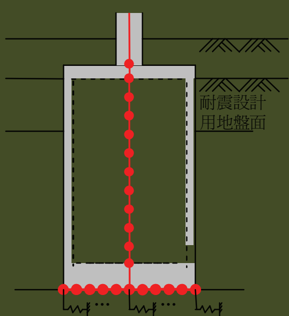
基礎彈簧分析模式

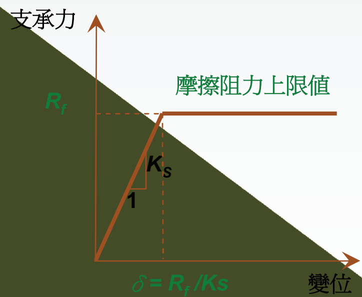
2.橋墩含沉箱基礎裸露或土壤液化之之耐震能力評估





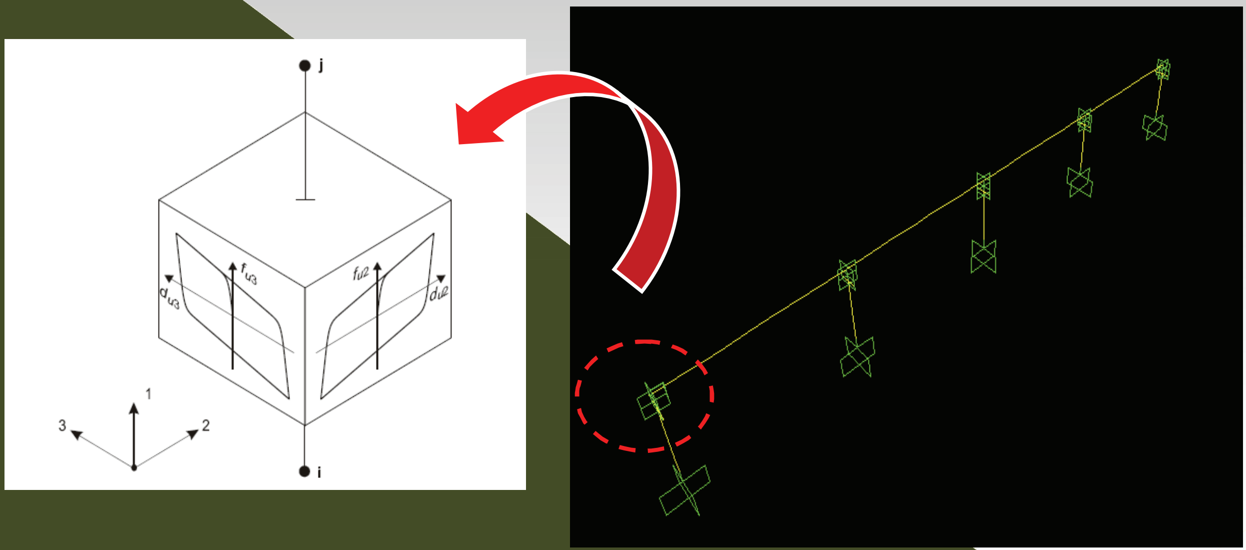
沉箱底面垂直與水平方向彈簧示意圖





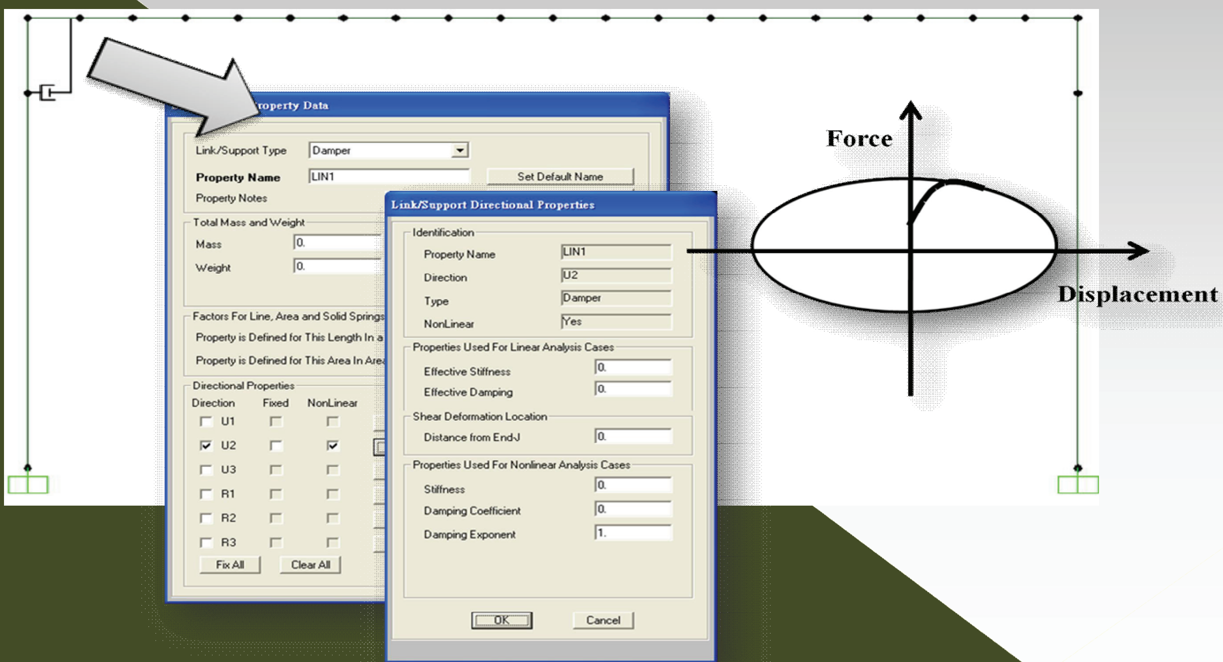
沉箱底面垂直方向彈簧

3.採用隔震支承之橋梁耐震能力評估



採用隔震支承之橋梁模式

3.採用阻尼器消能之橋梁耐震能力評估



採用阻尼器之橋梁模式

七、耐震性能評估流程

以側推分析為基準之橋梁耐震性能評估流程

- 建立結構物之容量曲線
- 轉換成結構物之容量譜
- 選定容量譜上任意一結構性能點非彈性譜位移與譜加速度之座標值，計算有效黏滯性阻尼比

$$\beta_{eff} = \frac{63.7\kappa\left(a_y d_{pi} - d_y a_{pi}\right)}{a_{pi} d_{pi}} + \beta_{basic}$$

- 計算阻尼比異於5%之修正係數

$$B_1 \ or \ B_s = \frac{(S_a)_{elastic}}{(S_a)_{inelastic}}$$

- 求算該結構性能點對應的地表加速度

$$PGA = f(a_{pi}, T, B_s or B_1)$$

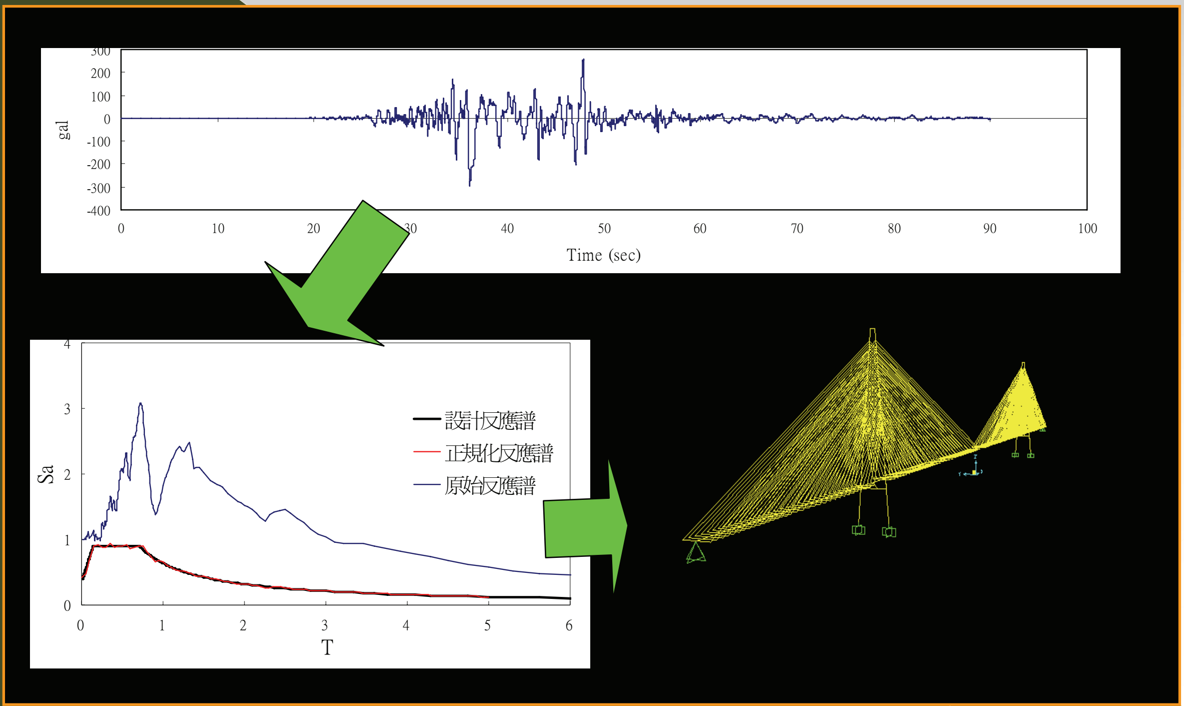
以非線性動力歷時分析為基準之橋梁耐震性能評估流程

步驟1：選取橋梁工址附近之三組原始地震歷時資料，並依橋梁耐震設計規範規定之橋梁工址設計反應譜為依據，將三組原始地震歷時資料製作成與設計反應譜相諧合(spectrum compatible)之三組人造地震歷時資料。

步驟2：將各組人造地震歷時資料從小至大逐步調整地震歷時資料的尖峰值(即最大地表加速度PGA)，並據此進行非線性動力歷時分析。

步驟3：記錄各種結構性能與PGA的關係，並繪製PGA與各種結構性能之關係圖，藉由關係圖進行橋梁耐震性能評估。

動力歷時分析



八、耐震性能是否合格之判別

強度準則

一般橋梁 (公路橋梁耐震設計規範 I=1.0) 之性能目標

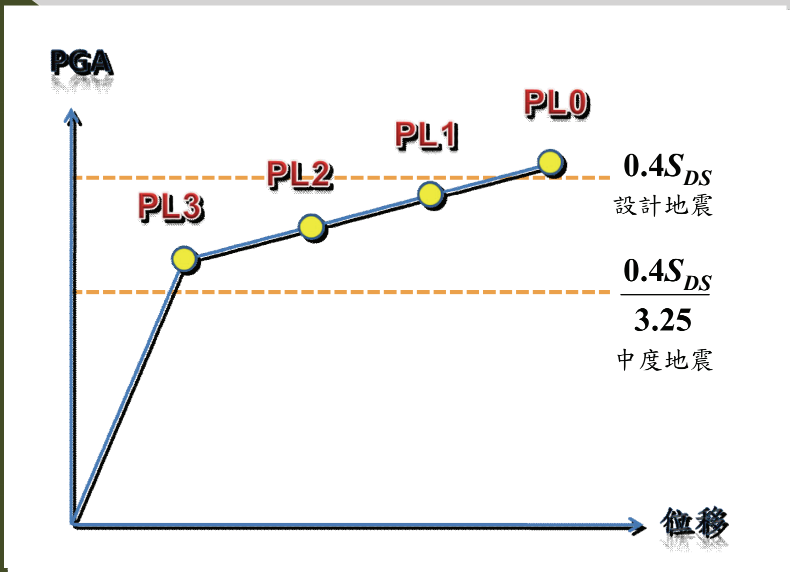
地震等級	採用之設計規範版本		
	84及89年版	49及76年版	49年版以前
中度地震	PL3	PL3	PL3
設計地震	PL2	PL1	PL0

重要橋梁 (公路橋梁耐震設計規範 I=1.2) 之性能目標

地震等級	採用之設計規範版本		
	84及89年版	49及76年版	49年版以前
中度地震	PL3	PL3	PL3
設計地震	PL2	PL1	PL1

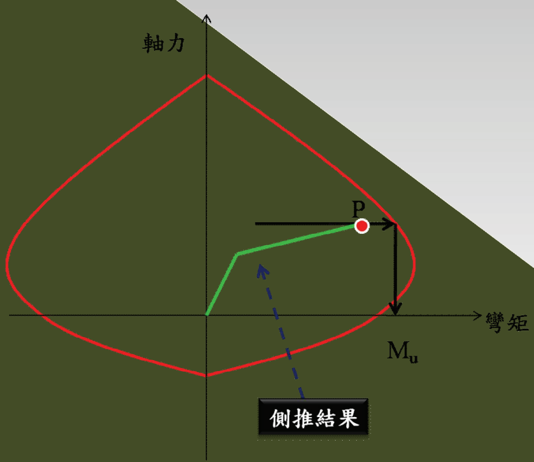
『國家地震中心，公路橋梁耐震評估及補強準則之研究成果報告』

性能目標



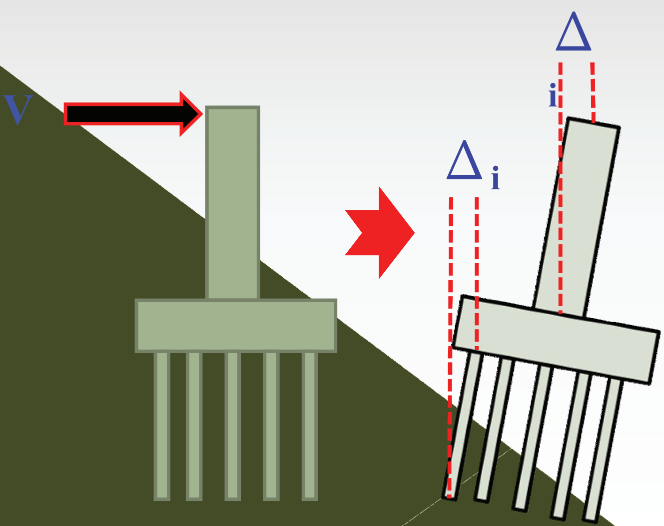
『國家地震中心，公路橋梁耐震評估及補強準則之研究成果報告』

位移準則-基樁



橋墩構材或基樁構材

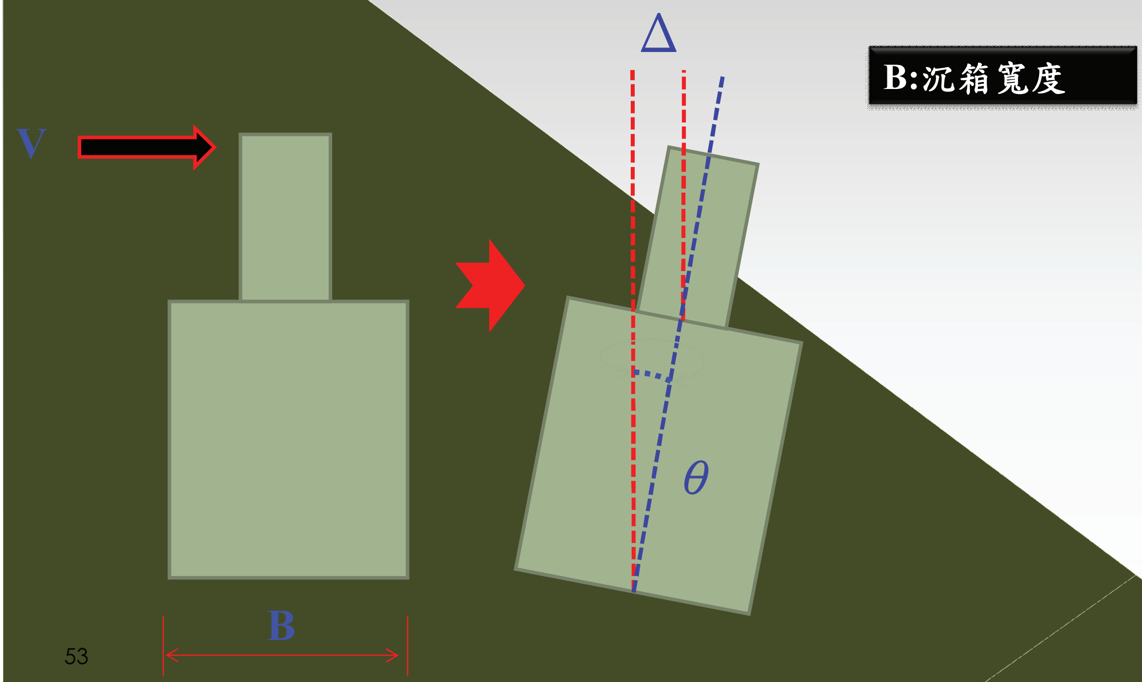
$$\Delta_i \le 0.2\frac{M_u}{P}$$



P:側推所得之軸力
M_u:側推所得軸力對應之彎矩

位移準則-沉箱

$$\begin{cases} \Delta \le \min\left(0.01B, \quad 5cm\right) \\ \theta \le 0.005rad \end{cases}$$



九、結論

綜合上述分析，提出下列幾點結論與建議：

1. 根據本文所研提之鋼筋混凝土構材塑性鉸分析與設定，並配合Takeda遲滯迴圈之設定所得之側推分析與非線性動力歷時分析成果，與國內外各研究機構從事之反覆載重試驗與擬動態試驗成果比對，顯示本文所研提塑性鉸分析與設定之正確性應可被確認。
2. 本文所提的改良式評估方法能夠完全展現整體橋梁結構自彈性狀態發展到非彈性狀態乃至於幾近崩塌狀態之全方位結構性能與最大地表加速度之關係，有效地提昇既有橋梁的耐震能力評估之技術。
3. 依據本文所述理論，研究團隊成功開發SERCB for Bridge視窗化輔助分析系統，提供工程師執行既有橋梁耐震能力評估之用，可有效降低既有橋梁耐震能力評估的困難度，對於既有RC橋梁之維護、補強或維修方式等有所助益。
4. 相關研究成果亦可供為新設橋梁耐震設計之參考。

報告完畢

敬請指教

國家地震工程研究中心
National Center for Research on Earthquake Engineering

國立臺北科技大學
National Taipei University of Technology