三段式耐震間柱之設計試驗、分析與應用方法

蔡克銓 / 許仲翔
李昭賢 /吳安傑/ 金步遠

含鋼板阻尼器或挫屈束制支撐構架之
耐震設計與分析技術研討會

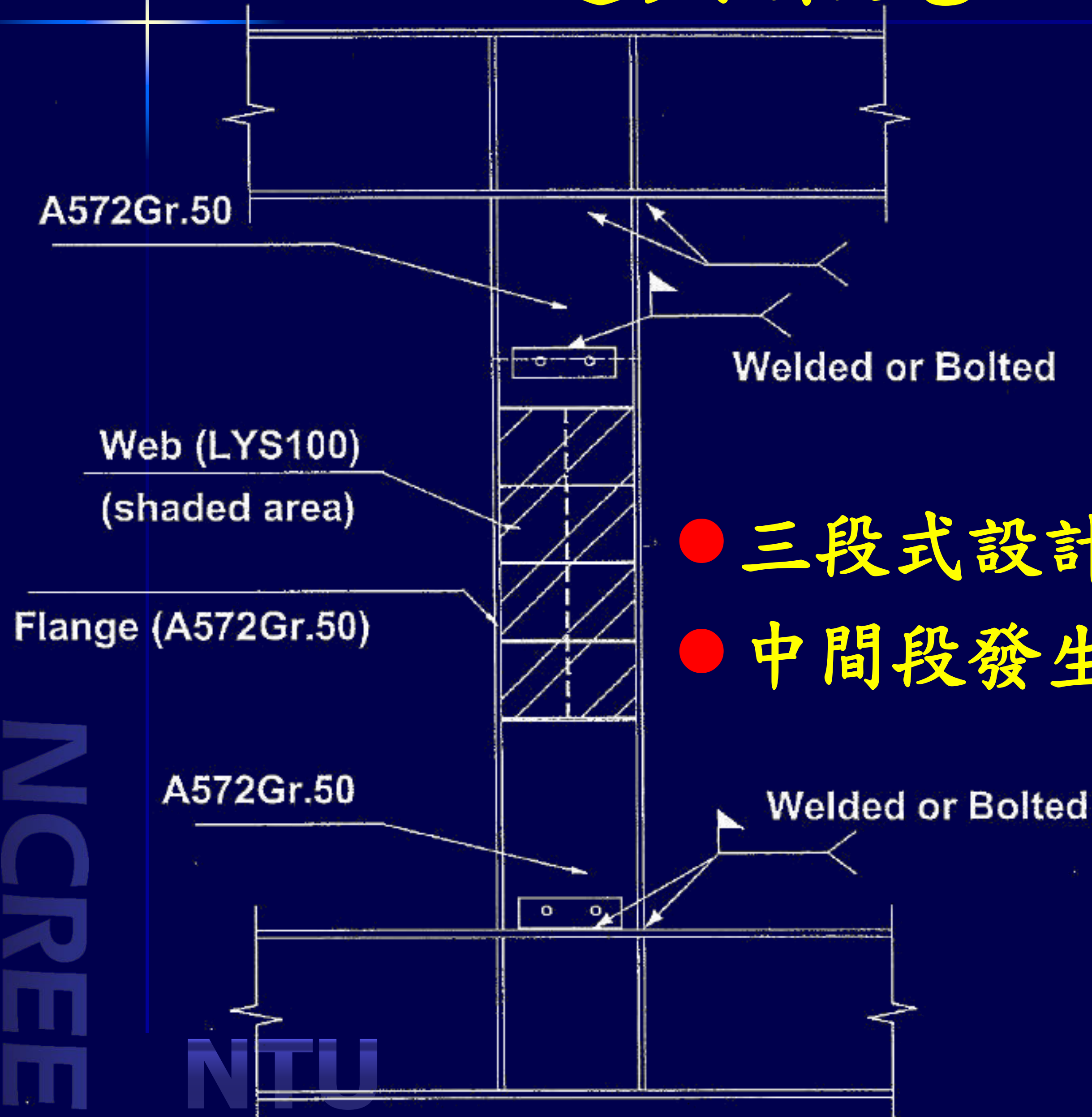
國家地震工程研究中心 / 國立台灣大學

2016/11/03

耐震間柱使用案例

南昌路與和平西路交接路口
(2016)

王宏遠與蔡克銓(2000)



- 三段式設計之耐震間柱
- 中間段發生剪力降伏以消能

利用鋼板阻尼器成為耐震間柱

Stub Column

Steel Panel Damper
鋼板阻尼器 (SPD)

SPD之加工方式

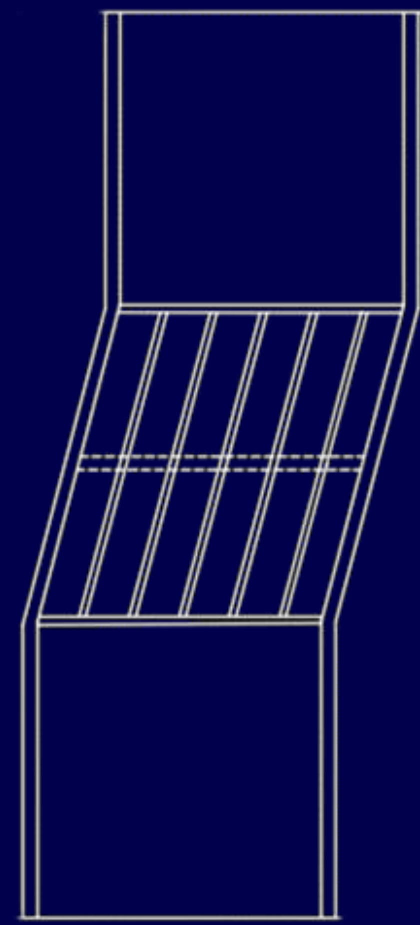
$$t_{w,IC} \le t_{w,EJ}$$

兩片相同的翼板
兩種厚度的腹板
再加上加勁板

Elastic Joint (EJ)

Inelastic Core (IC)

Elastic Joint (EJ)



試體規劃-加勁板的配置

End-Stiffeners :

Separating the core and joint

Buckling restraining Stiffeners

5 Longitudinal

SPD-5L1T

1 Transverse

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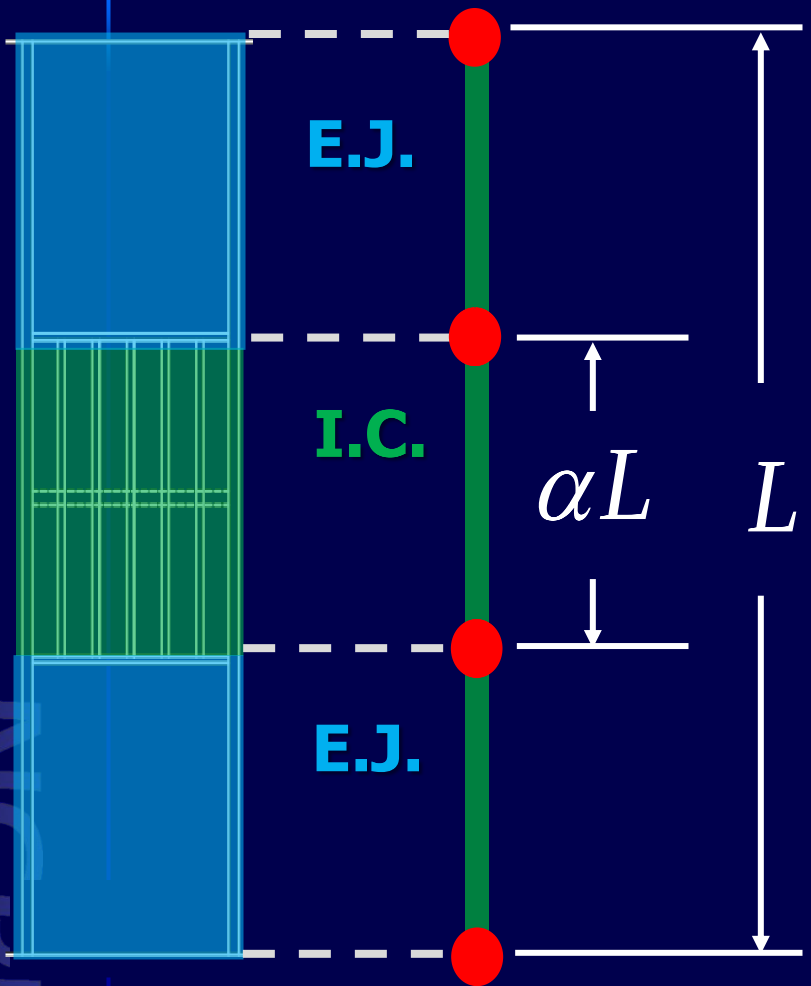
SPD力學性質容易預測



剪力強度由**I.C.**段決定

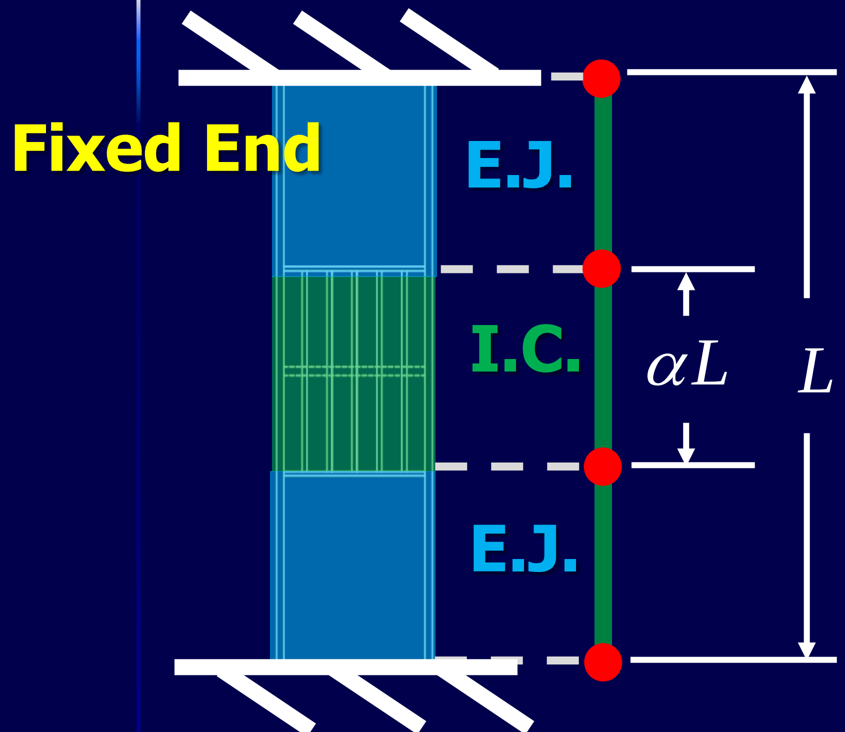
$$V_{\max} = R_y \omega 0.6 F_y A_{v,IC}$$

Material Overstrength

Strain Hardening

$$A_{v,IC} = d \times t_{w,IC}$$

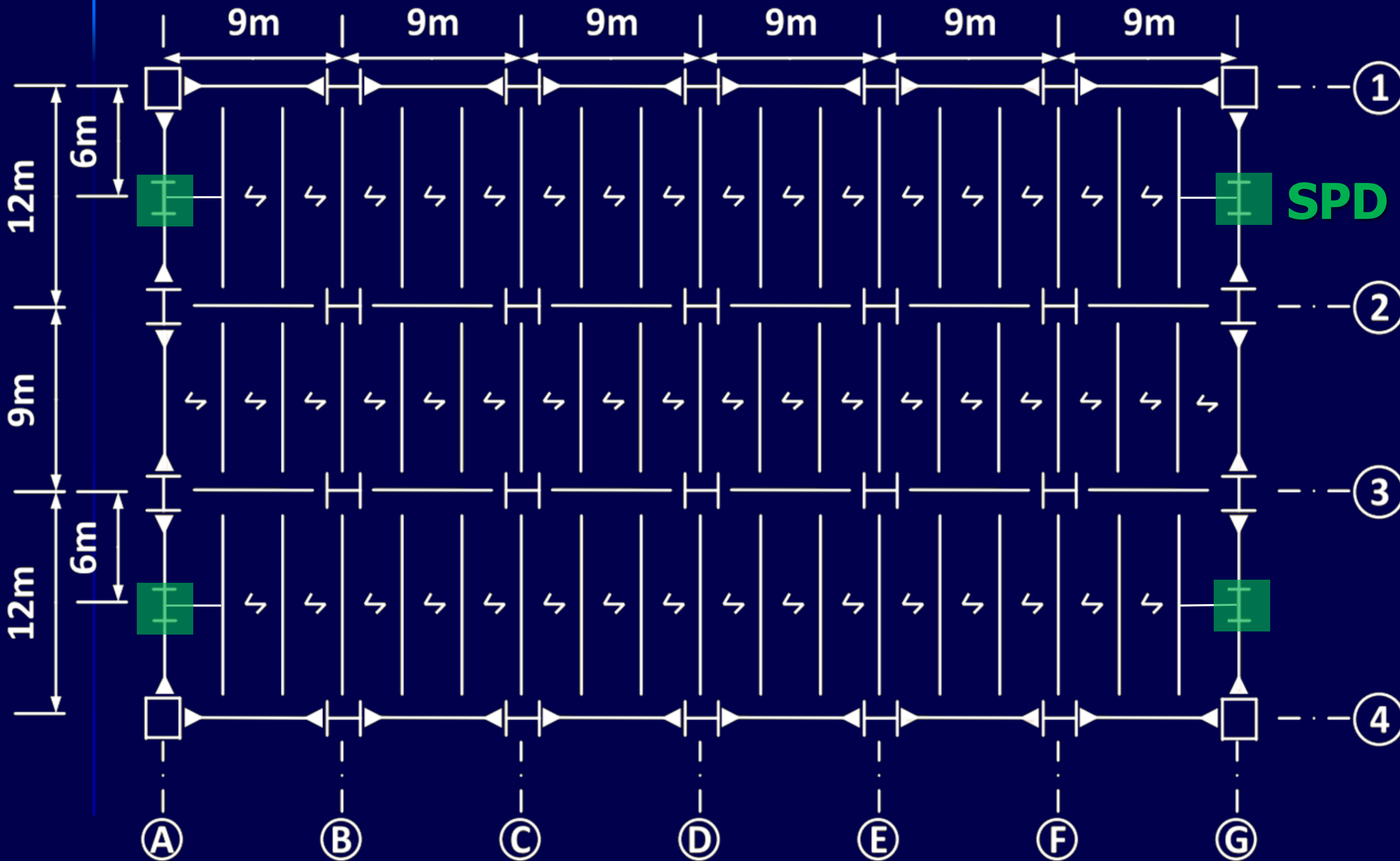
SPD等效勁度容易估算



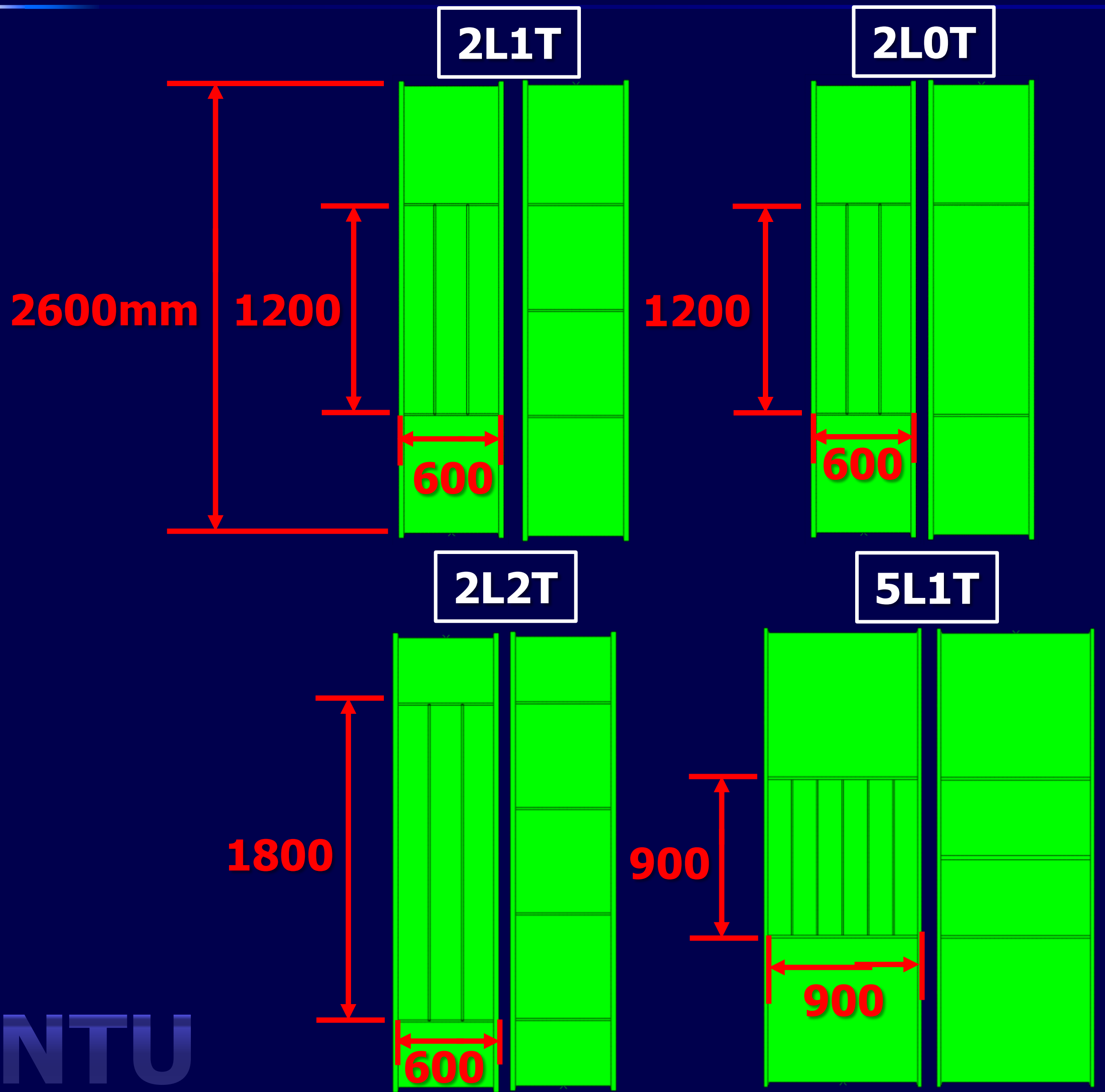
K_{eff} : 等效勁度
α : 核心段與全長之比值
A_v : 核心段斷面積
I : 核心段慣性矩

$$K_{eff} = \frac{1}{1/K_{IC} + 2/K_{EJ}} = \frac{1}{\left[\frac{\alpha L}{GA_{V,IC}} + \frac{(\alpha L)^3}{12EI_{IC}}\right] + \left[\frac{(1-\alpha)L}{GA_{V,EJ}} + \frac{L^3 - (\alpha L)^3}{12EI_{EJ}}\right]}$$

樓層平面圖



SPD試體尺寸



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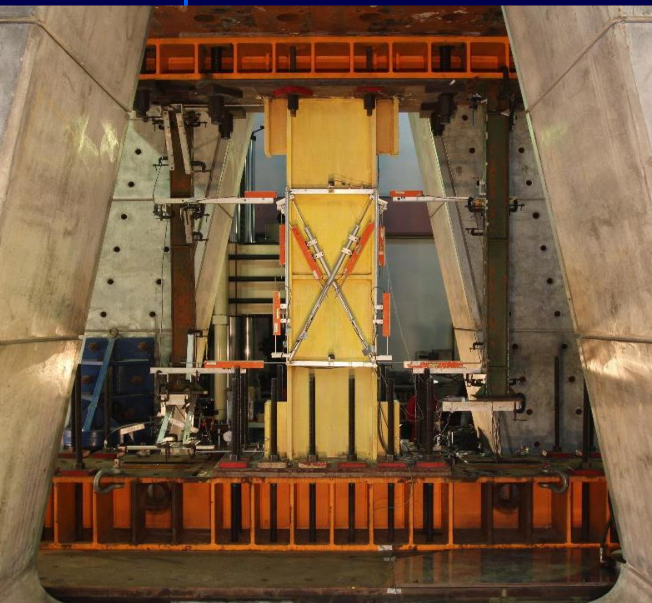
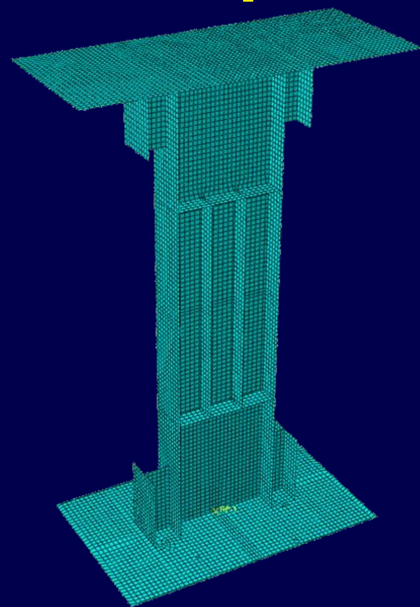
設計、試驗與分析方法

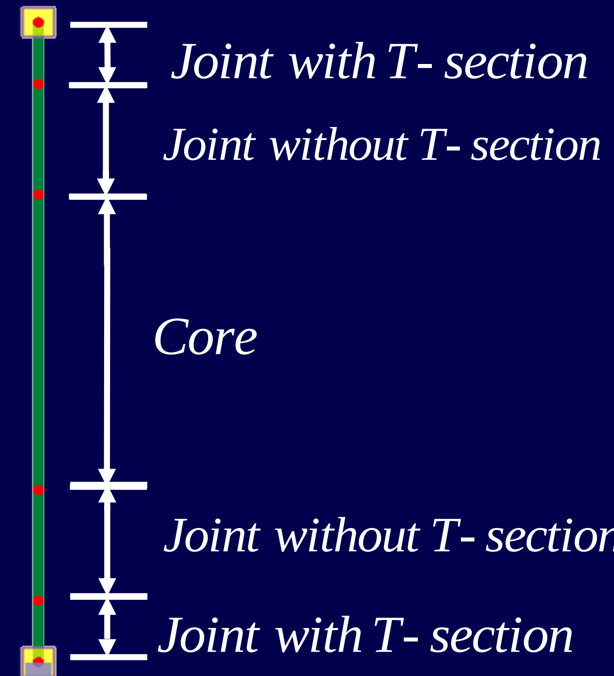
Design Method → Tests

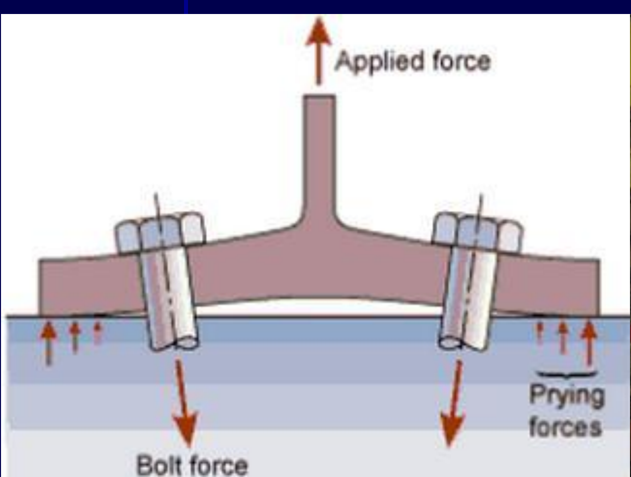


Abaqus



PISA3D





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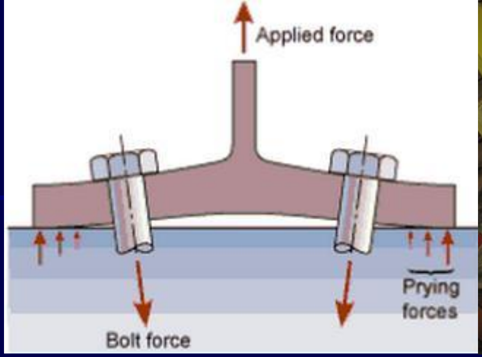
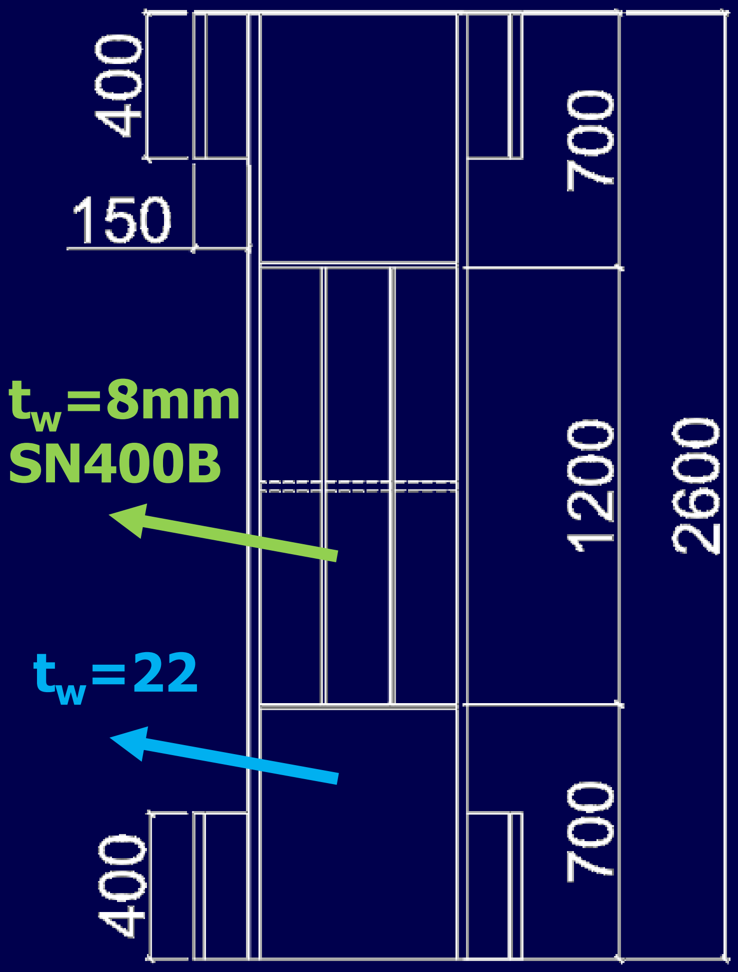
SPD-2L1T



H600x250xt_wx30(A992)

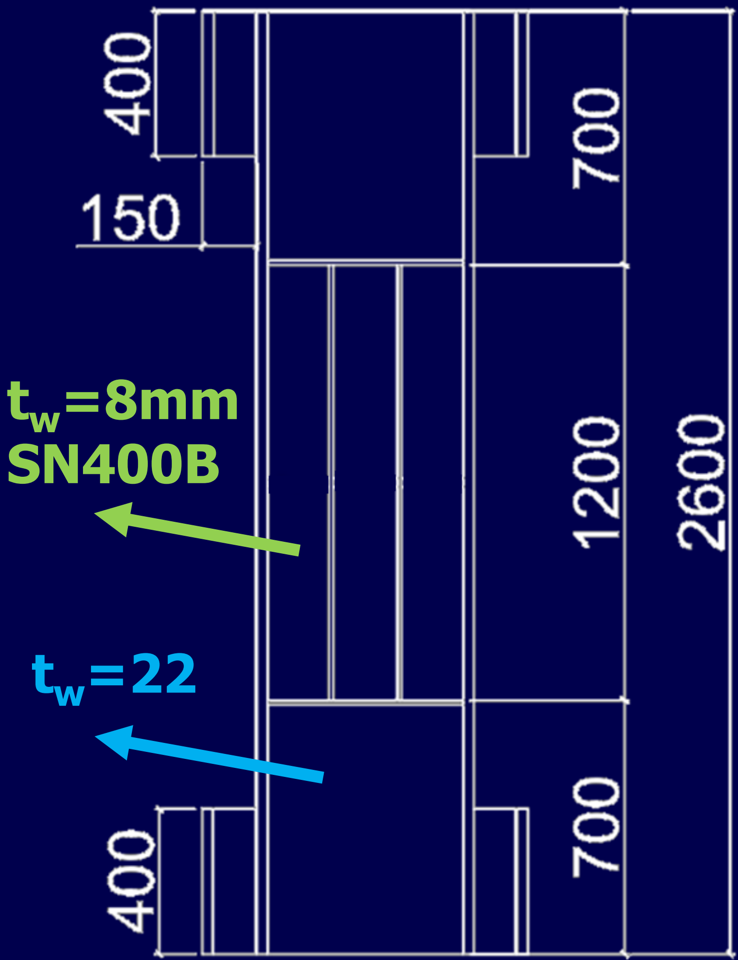


t_w=8mm SN400B

t_w=22

SPD-2L0T

$H600x250xt_w x30(A992)$



需求與容量比

Demand to Capacity Ratio = DCR

$$\text{DCR} = \frac{\text{Demand}}{\text{Capacity}}$$

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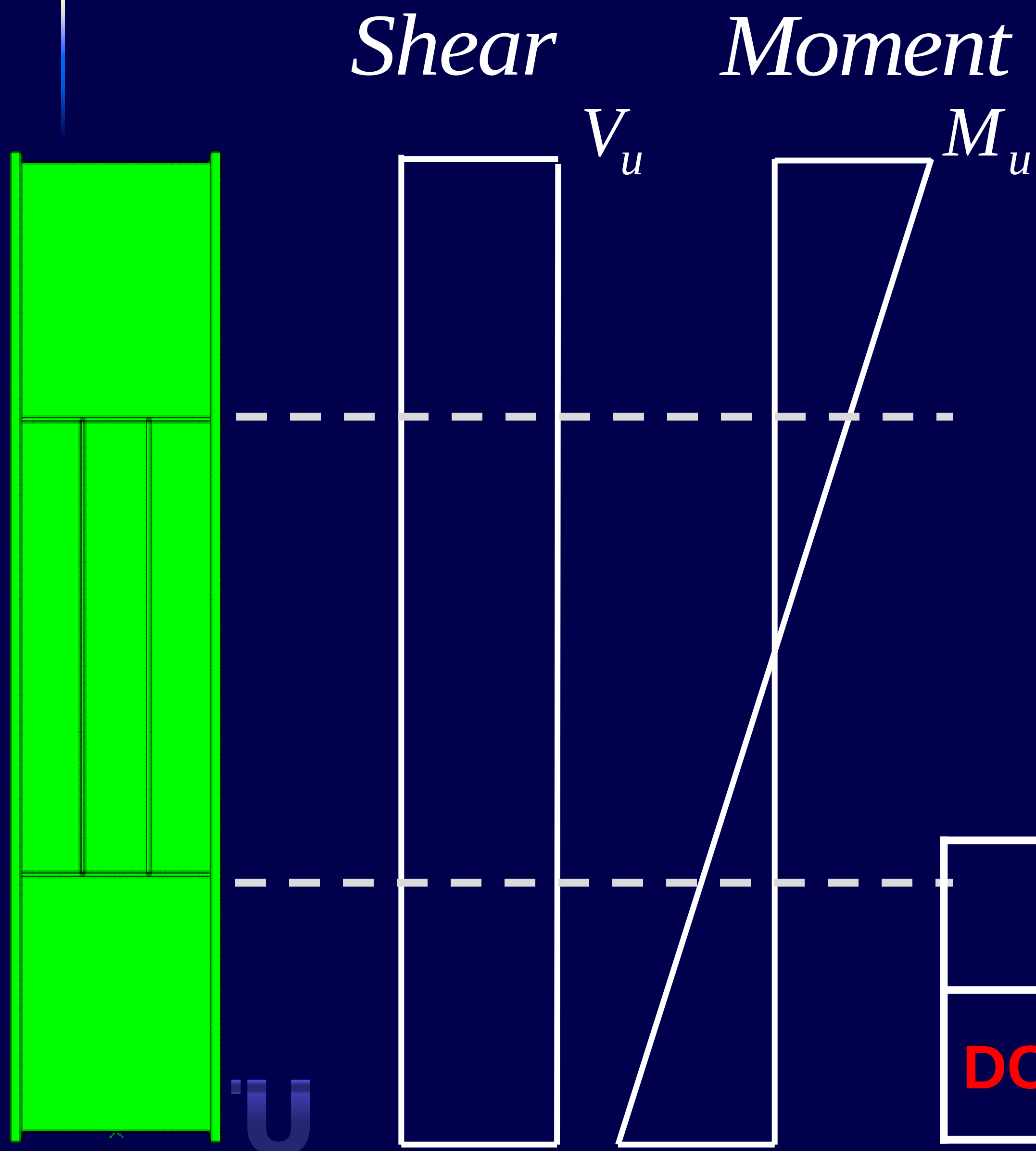
端部連接段容量設計-保持彈性



$$\phi M_{n,EJ} \geq M_u$$

$$\phi V_{n,EJ} \geq V_u$$

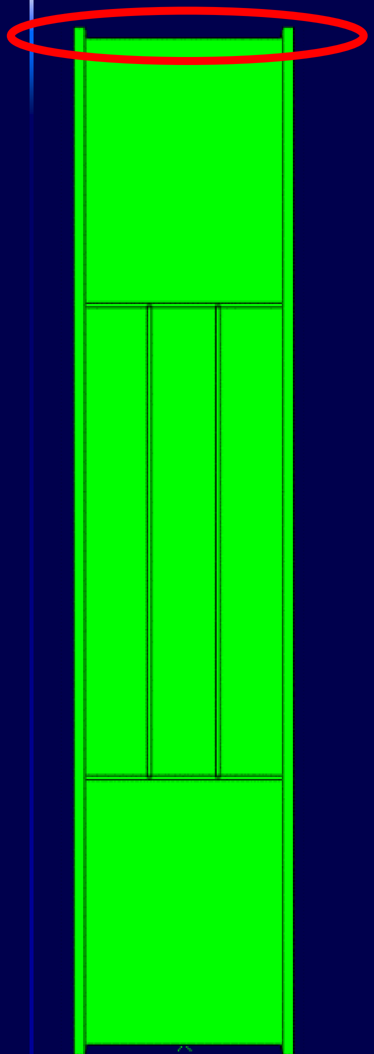
$$\phi = 0.9$$

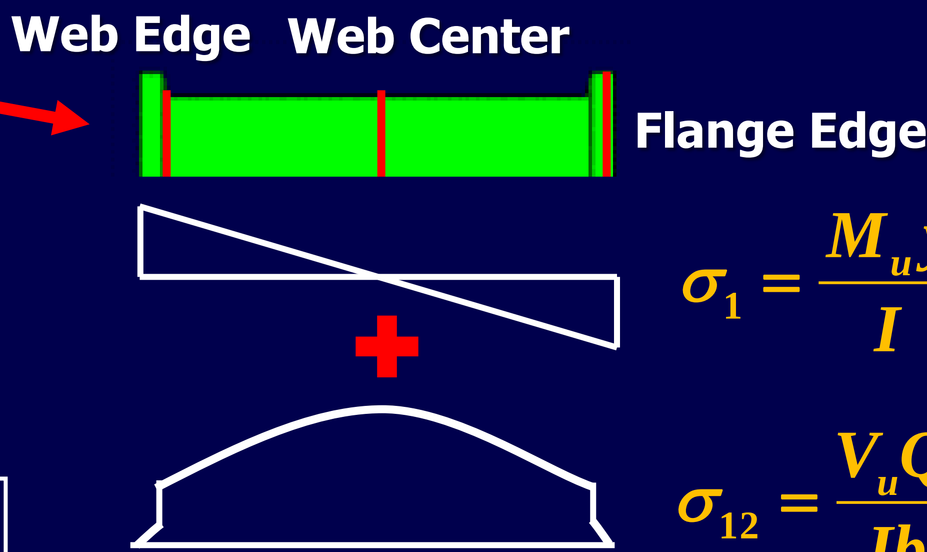
$$V_u = \omega R_y V_y$$

$$M_u = V_u \frac{L_{SPD}}{2}$$

	Moment	Shear
DCRs	0.99	0.56

容量設計-von Mises降伏準則





$$\sigma_1 = \frac{M_u y}{I}$$

$$\sigma_{12} = \frac{V_u Q}{Ib}$$

$$\sigma_2 = 0$$

$$\sigma = \sqrt{\sigma_1{}^2 + \sigma_2{}^2 - \sigma_1\sigma_2 + 3\sigma_{12}{}^2} \le \phi\sigma_y$$

$$\phi = 0.9$$

	Web Edge	Web Center	Flange Edge
$\sigma(tf/cm^2)$	3.26	2.15	3.27
$\phi\sigma_y(tf/cm^2)$	3.15	3.15	3.15
DCRs	1.03	0.68	1.03

長向加勁板間距設計

$$R_w = \frac{b}{t_w}\sqrt{\frac{12(1-\nu^2)\tau_y}{k_e \pi^2 E}} \le 0.2$$

Designed by Abaqus

	SPD-2L1T	SPD-2L0T
R_w	0.31	0.32

Capacity of Stiffened Steel Shear Panels as a Structural Control Damper
H.B. Ge, K. Kaneko and T. Usami (2008)

b : short side of separated web
t_w : thickness of core web
τ_y : yielding shear stress of core web

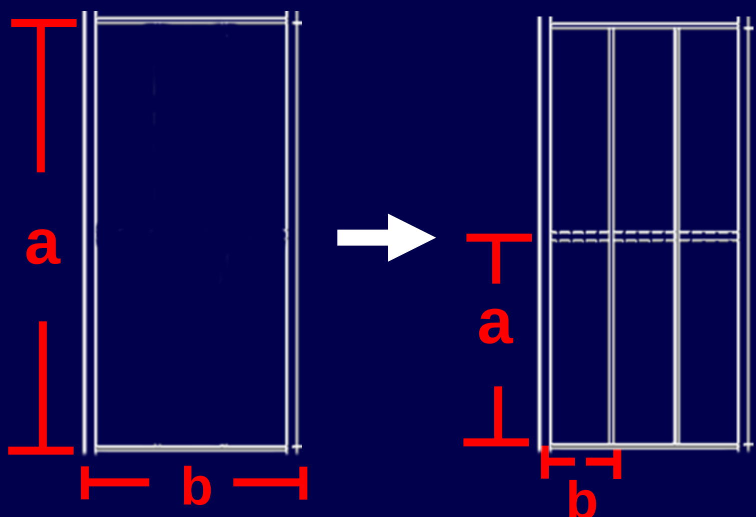
$$K_e \text{ range from 4 to 6}$$

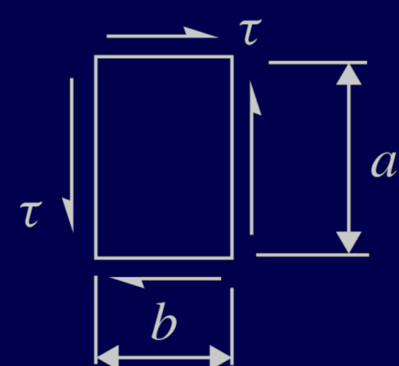
$$K_e = 3.96 + 2.24\left(\frac{b}{a}\right)^2$$

$$0 \le \frac{b}{a} \le 1$$

a : longside

b : shortside





長向加勁板斷面設計

$$\frac{\gamma}{\gamma^*} \geq 1.0$$

Capacity of Stiffened Steel Shear Panels as a Structural Control Damper
H.B. Ge, K. Kaneko and T. Usami (2008)

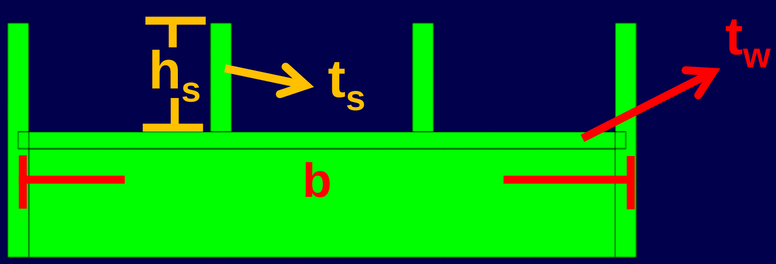
$$\gamma = \frac{EI_s}{bD} \qquad I_s = \frac{t_s h_s^{\ 3}}{12} \qquad D = \frac{Et_w^{\ 3}}{12(1-\nu^2)}$$

$$\gamma^* = \frac{3(27.3 n_L^{\ 0.6}\alpha - 23.3\alpha)}{(0.2 n_L^{\ 0.7} - 0.60/\alpha + 0.52/\alpha^2)}$$

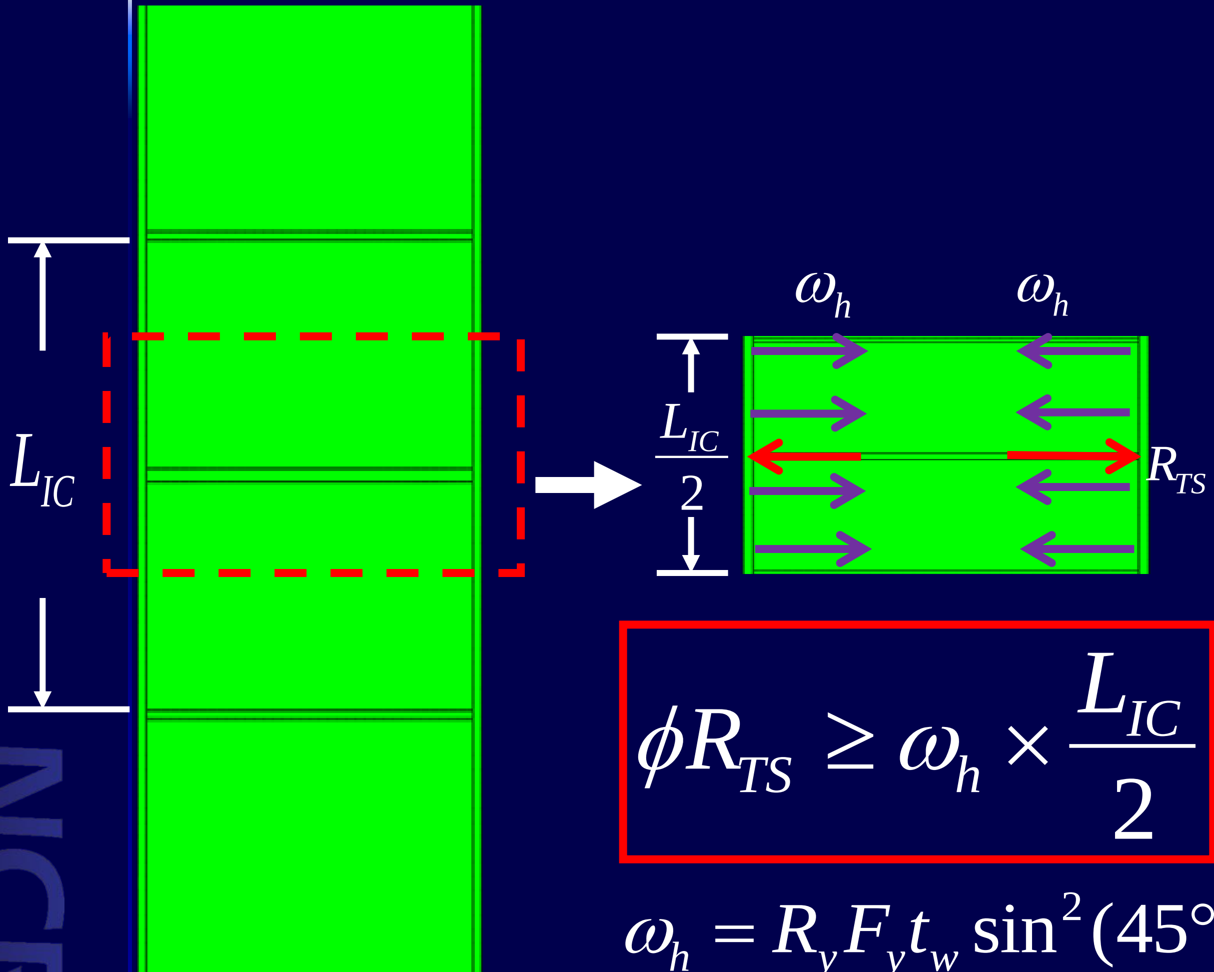
t_s(mm)	12
h_s(mm)	100
γ / γ^*	1.5

n_L : *number of longitudinal stiffeners*

$$\alpha = \frac{b}{a} \leq 1.0$$



短向加勁板之設計



$$\phi R_{TS} \geq \omega_h \times \frac{L_{IC}}{2}$$

$$\omega_h = R_y F_y t_w \sin^2(45°)$$

$$R_{TS} = t_s \times h_s \times F_y$$

$$\phi = 0.9$$

t_s(mm)	25
h_s(mm)	100
ϕR_{TS} (kN)	788
$\omega_h L_{IC}/2$ (kN)	780
DCR	**0.99**

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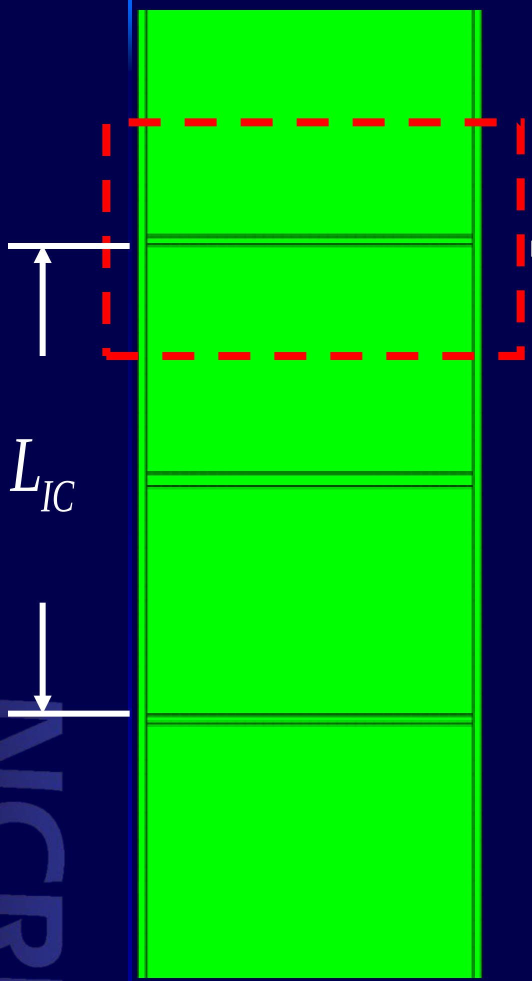
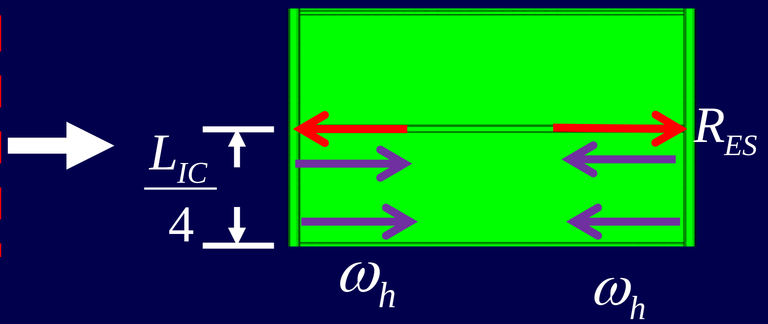
端部加勁板之設計





t_s(mm)	12
h_s(mm)	100
ϕR_{ES} (kN)	378
$\omega_h L_{IC}/4$ (kN)	386
DCR	**1.05**

$$\phi R_{ES} \geq \omega_h \times \frac{L_{IC}}{4}$$

$$\omega_h = R_y F_y t_w \sin^2(45°)$$

$$R_{TS} = t_s \times h_s \times F_y$$

$$\phi = 0.9$$

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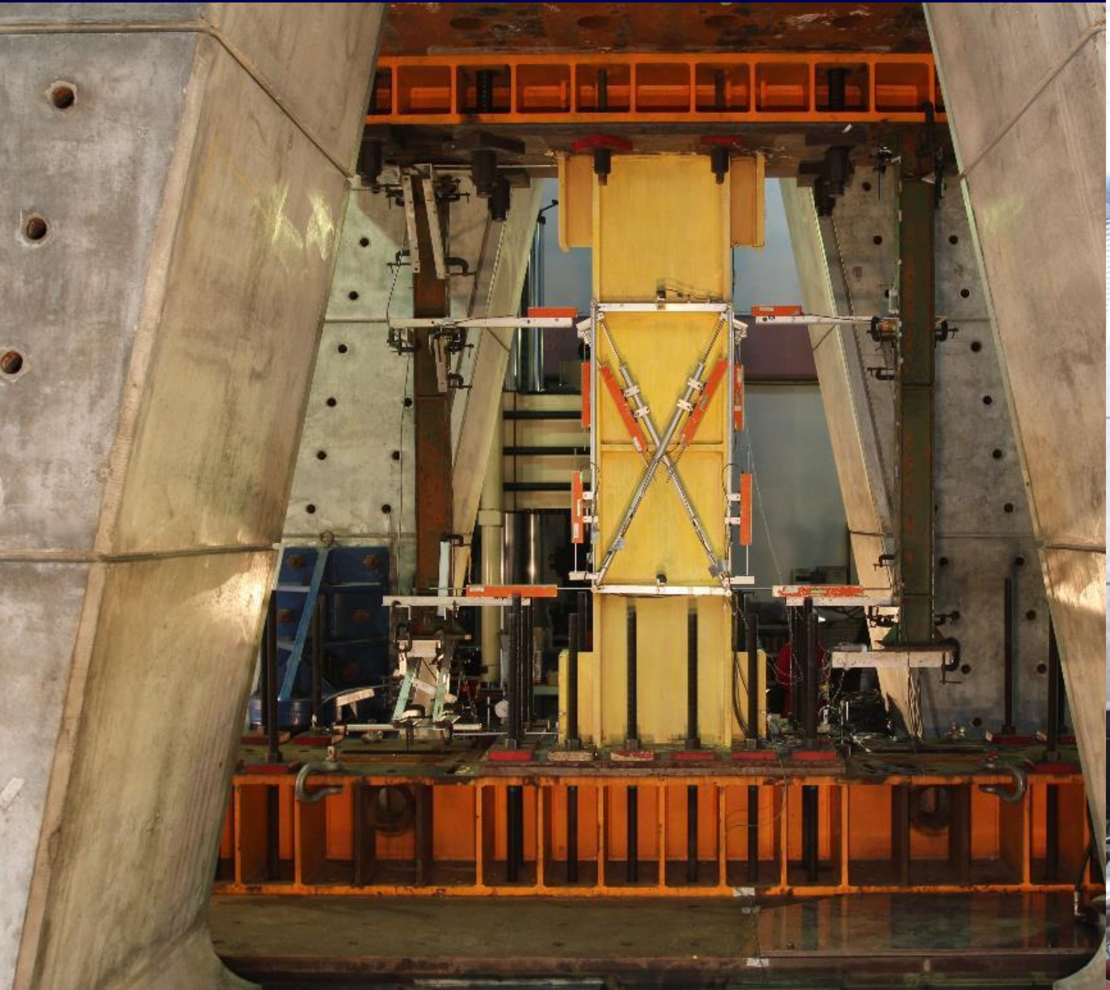
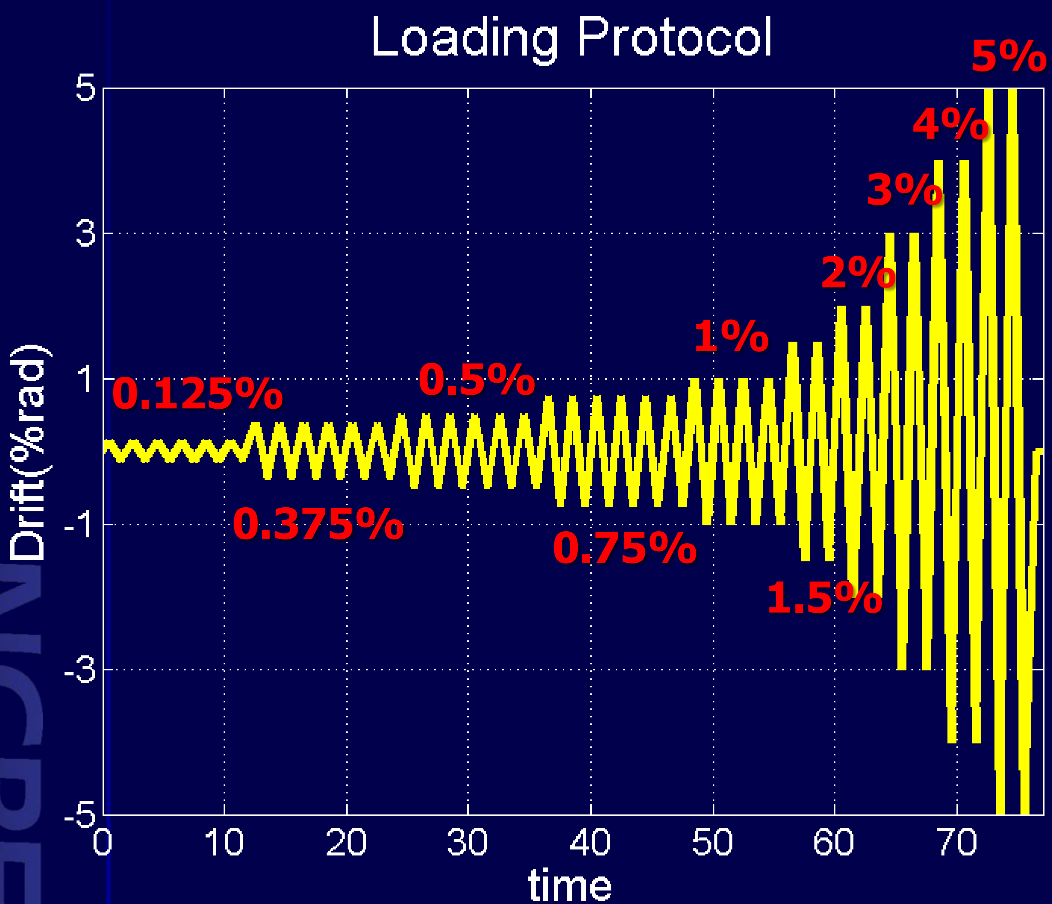
國震中心MATS試驗系統

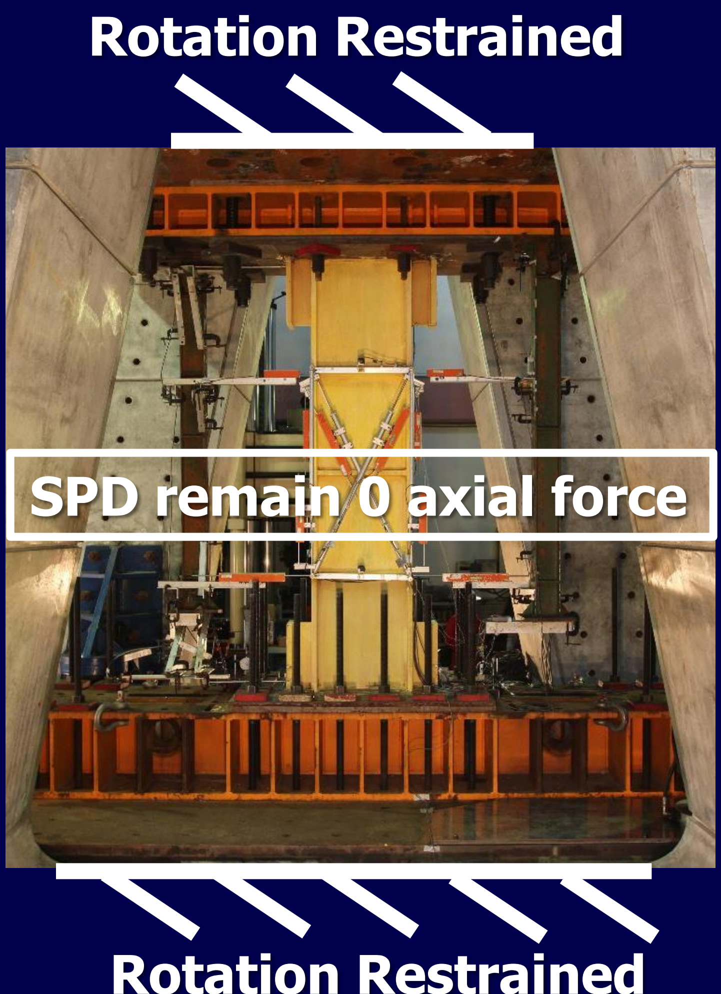


試驗加載歷時

Based on AISC341-10

Story Height: 3.6m





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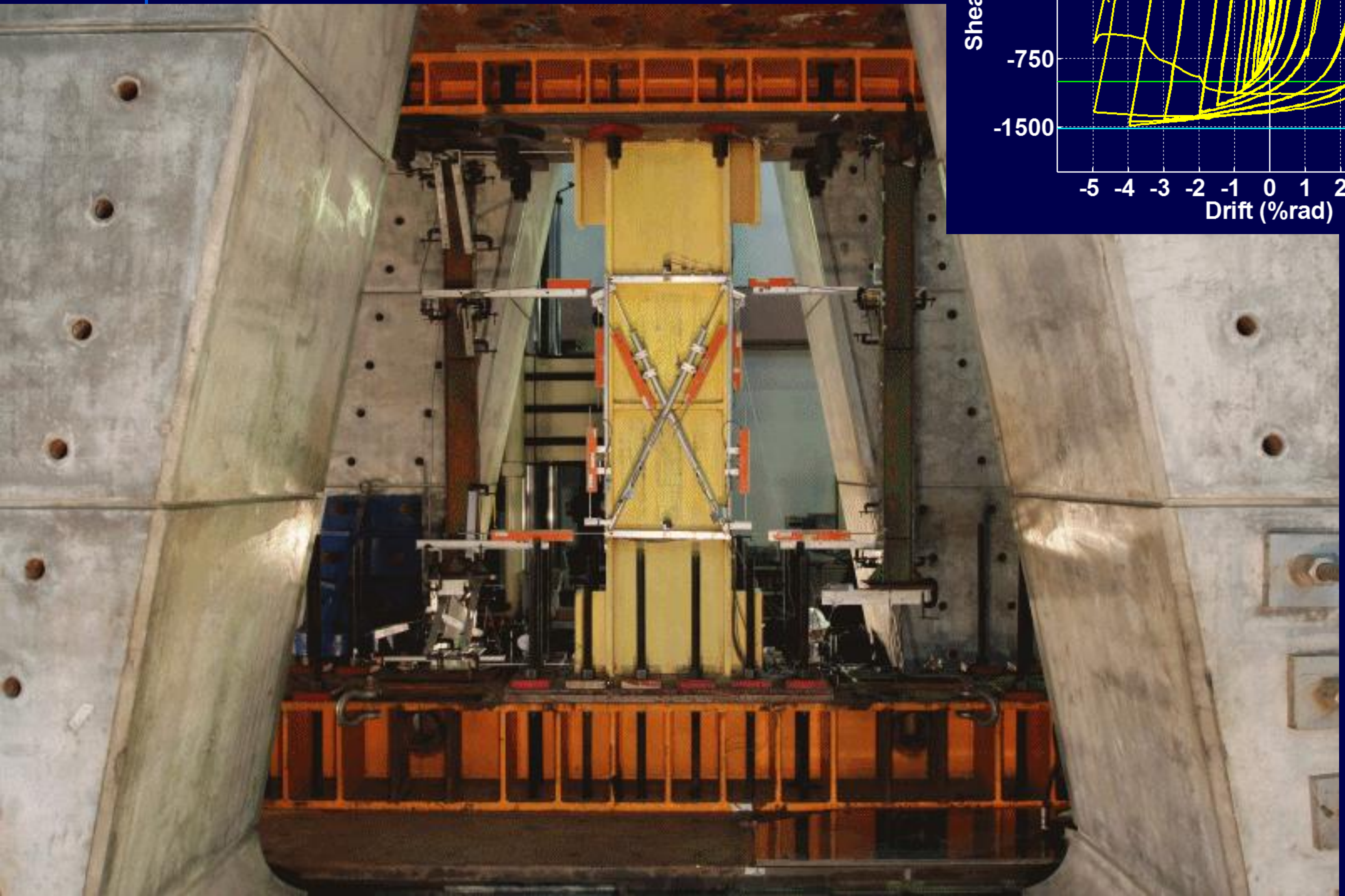
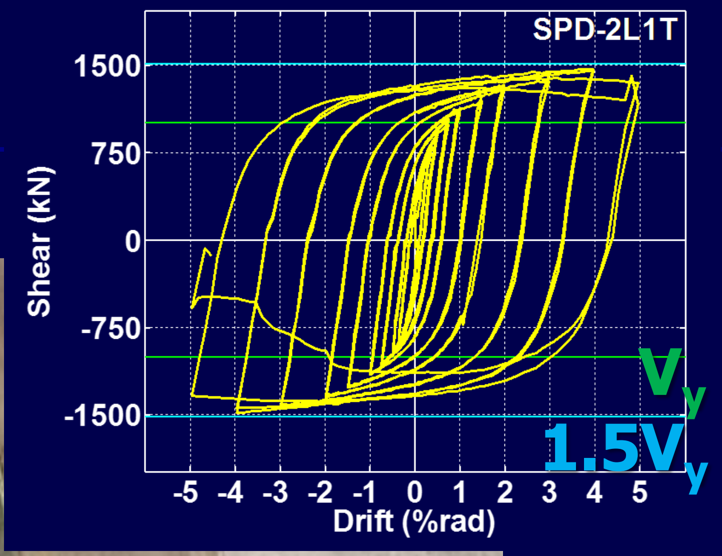
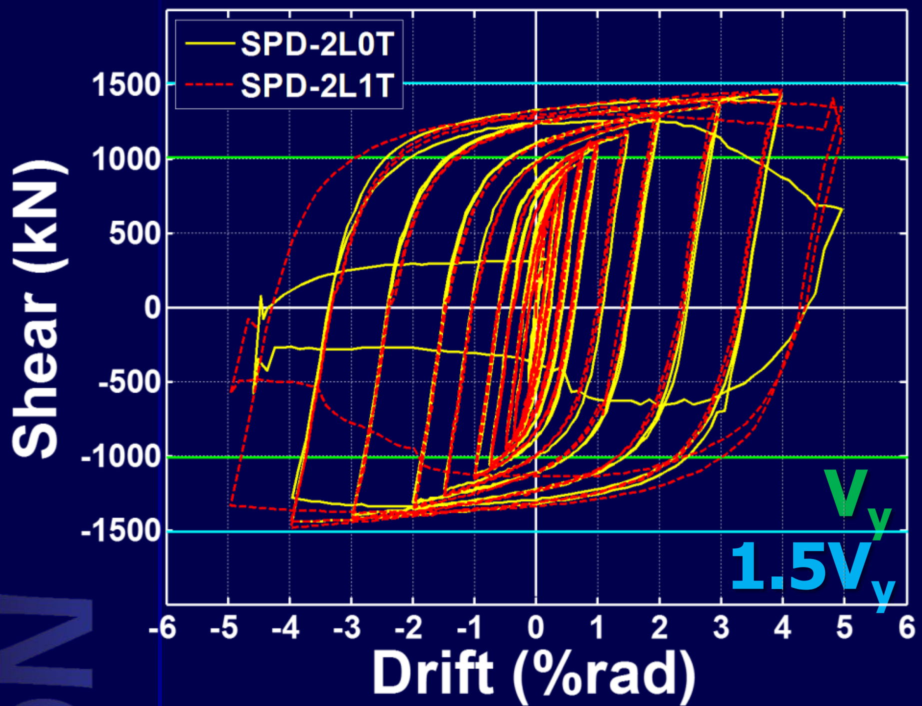
SPD-2L1T

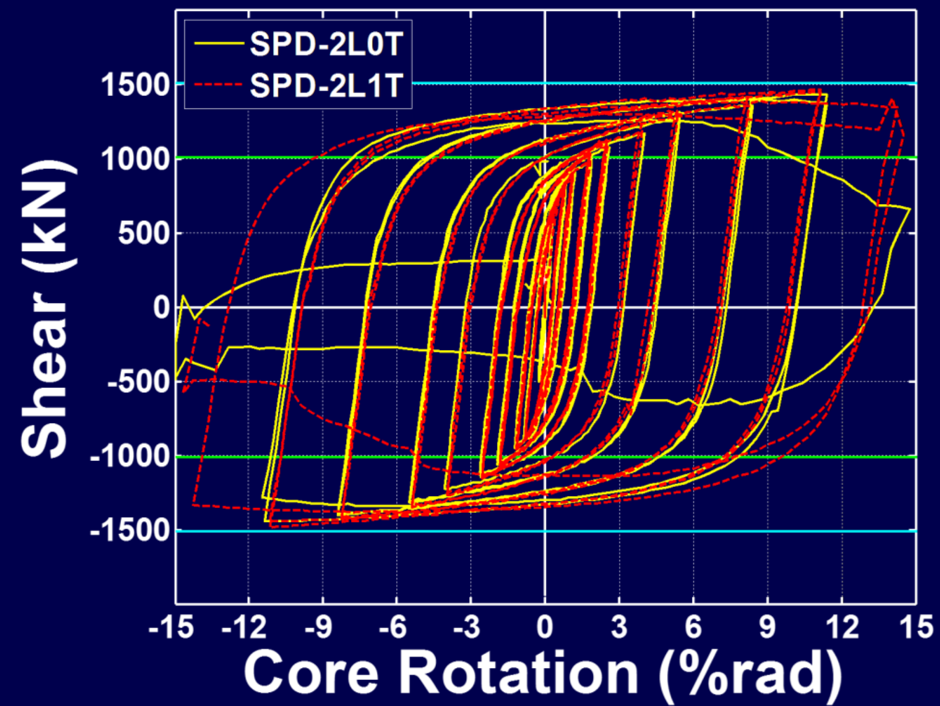



SPD-2L1T
1500
750
0
-750
-1500
Shear (kN)
-5 -4 -3 -2 -1 0 1 2 3 4 5
Drift (%rad)
V_y
$1.5V_y$

SPD-2L1T vs. 2L0T

Core rotation capacities are close in both test





	Maximum Core rotation (%rad)
SPD-2L1T	11.1
SPD-2L0T	11.3

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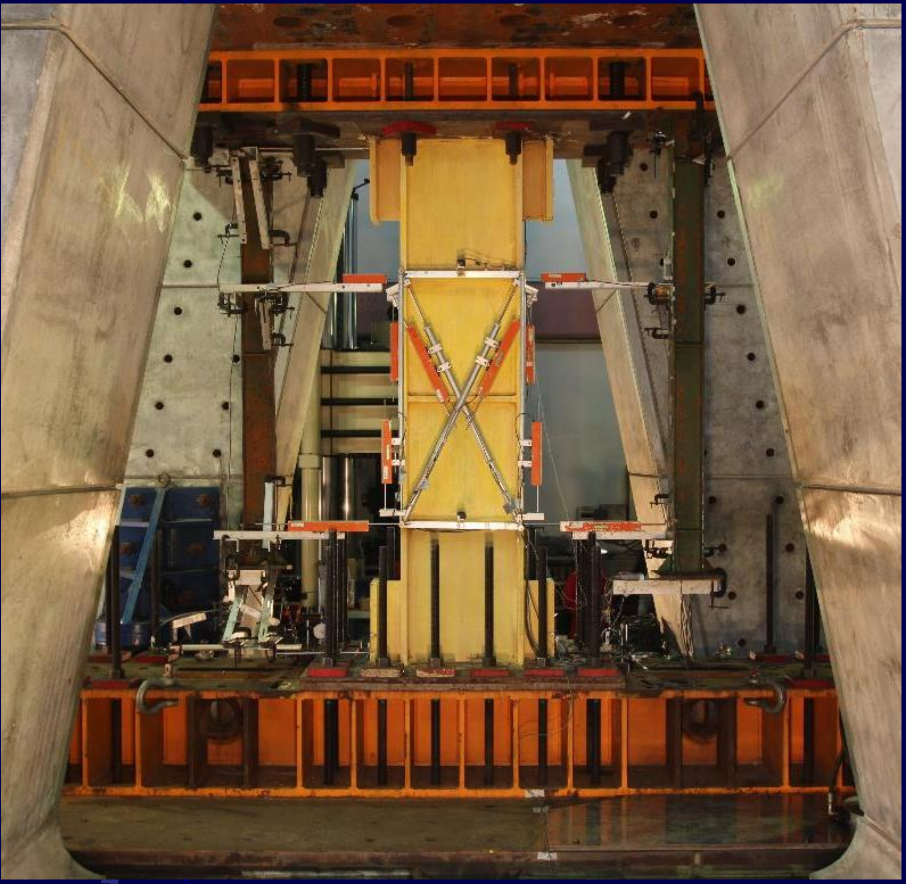
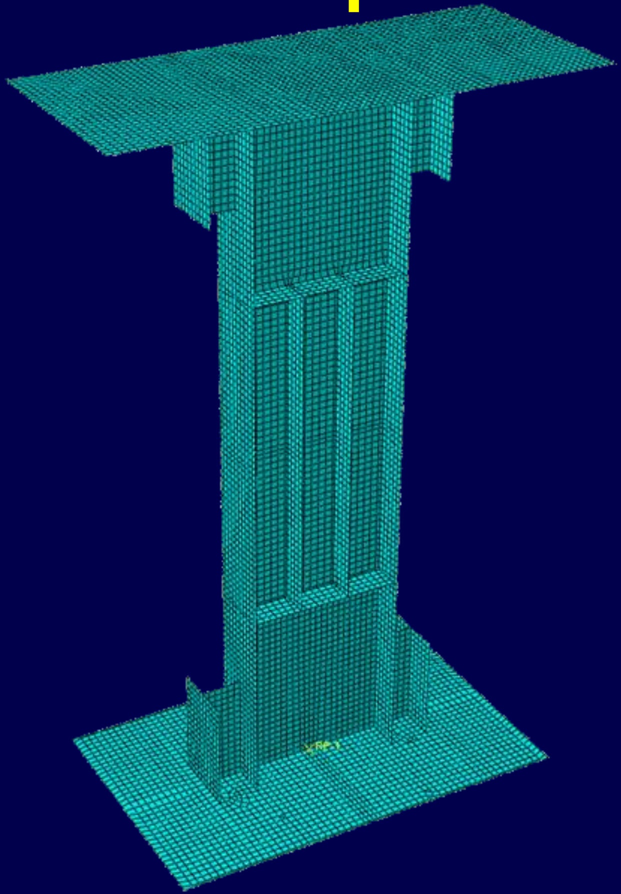
試體SPD-2L1T - Abaqus模型

Tests



Shell Elements
Mesh Size: 25mm
Number of Elements: 11033

Abaqus



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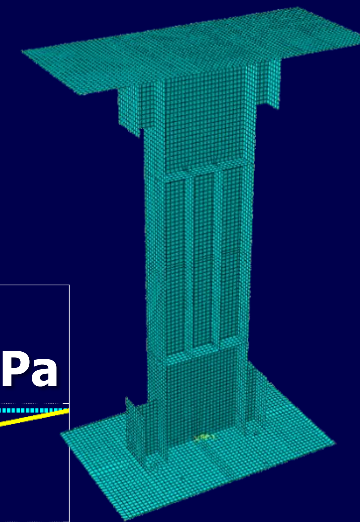
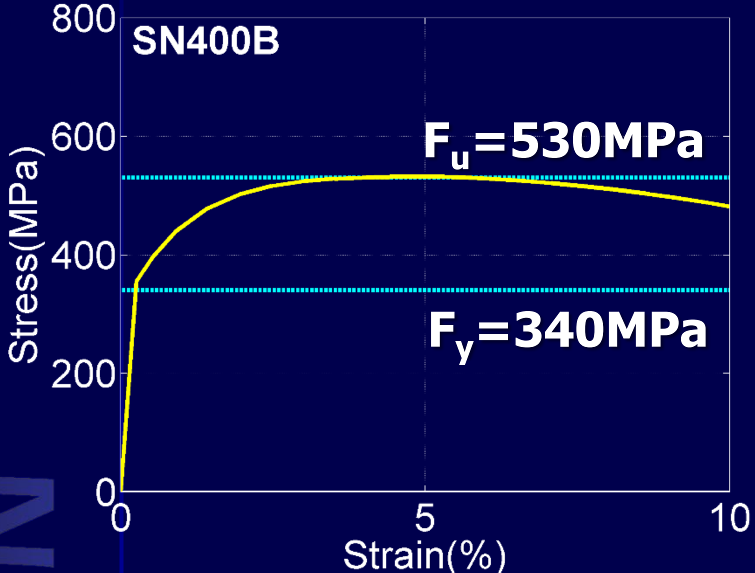
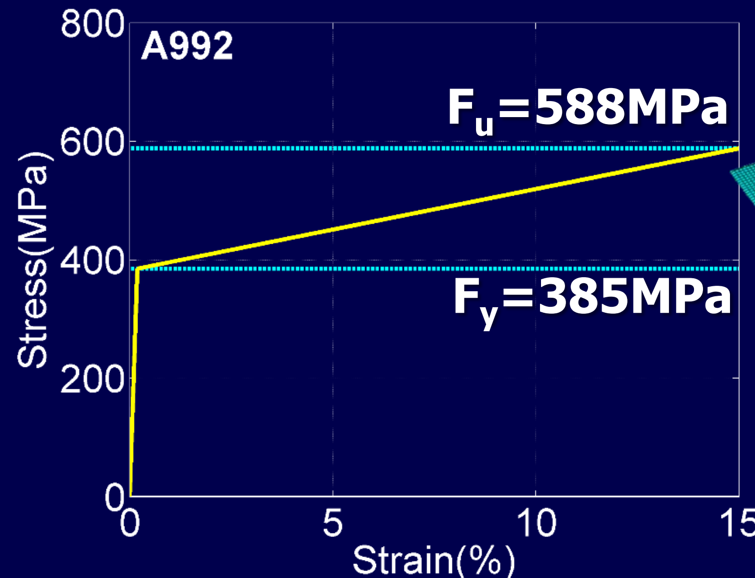
Abaqus模型材料參數設定



Core Web of SPDs

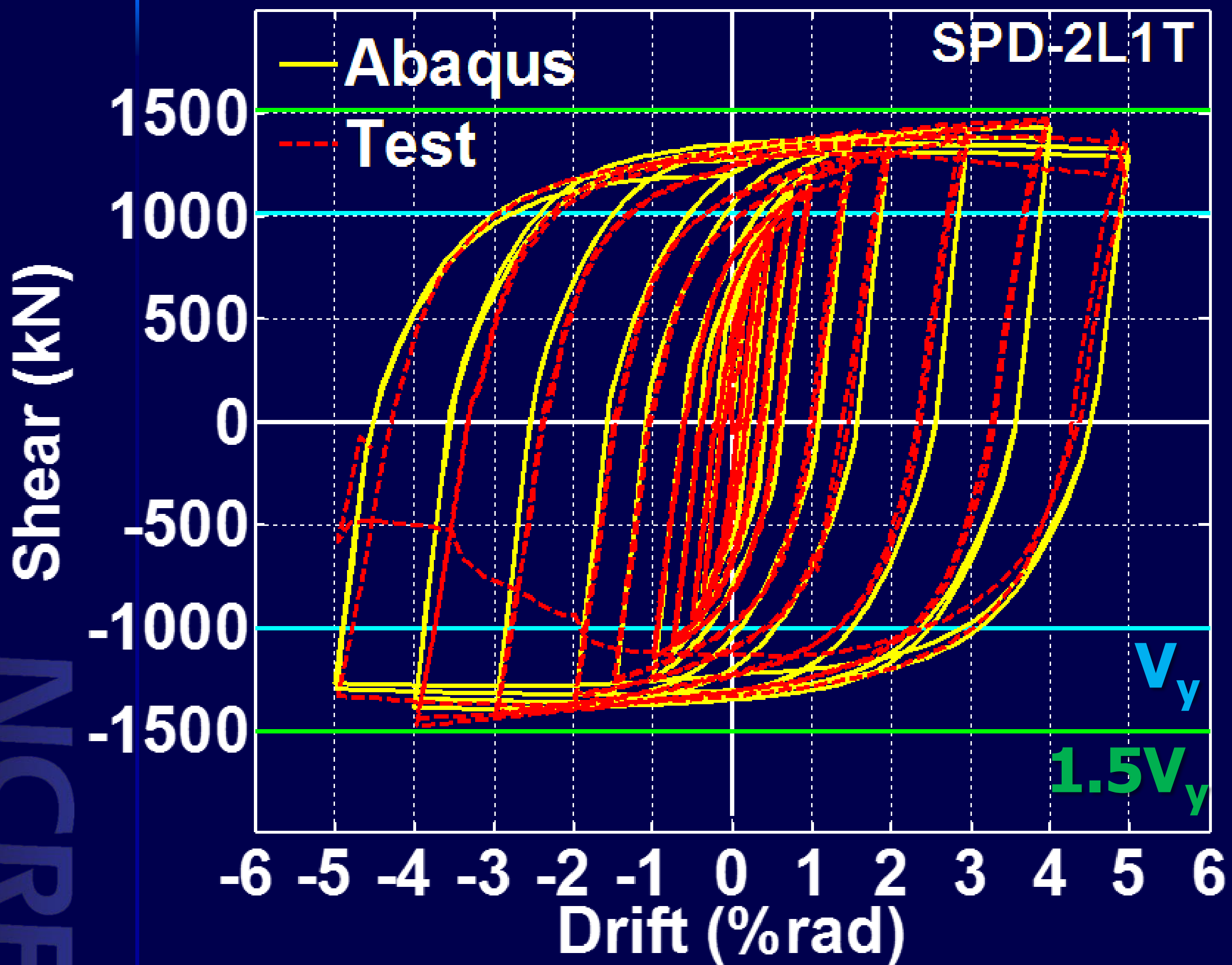


Rest of all



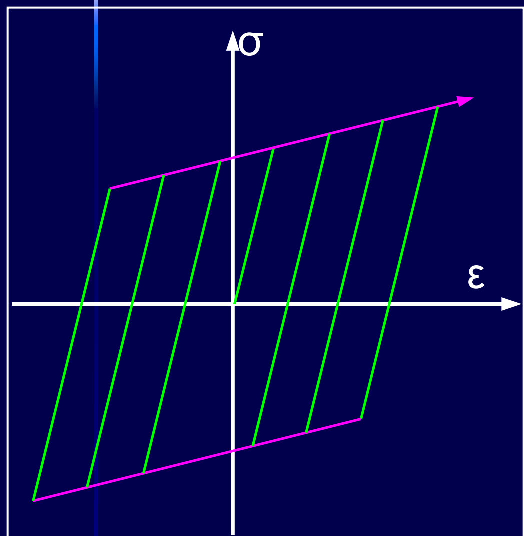
Material	Fy(MPa)	Fu(MPa)	C1	γ_1	Q	b
CNS SN400B	340	530	20	100	0.1	5
ASTM A992	385	588				

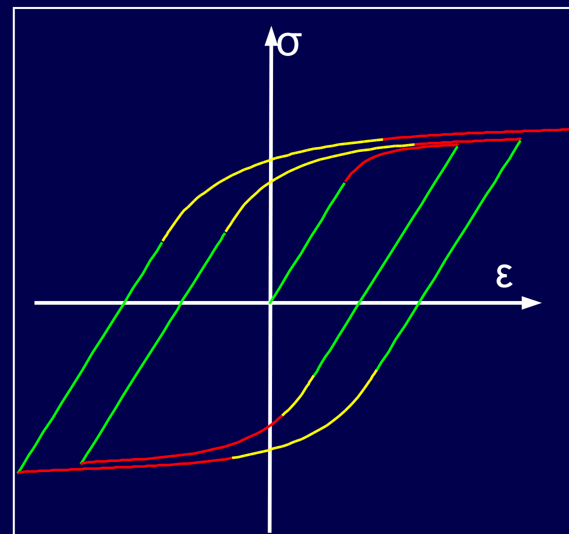
Abaqus可準確模擬試驗結果

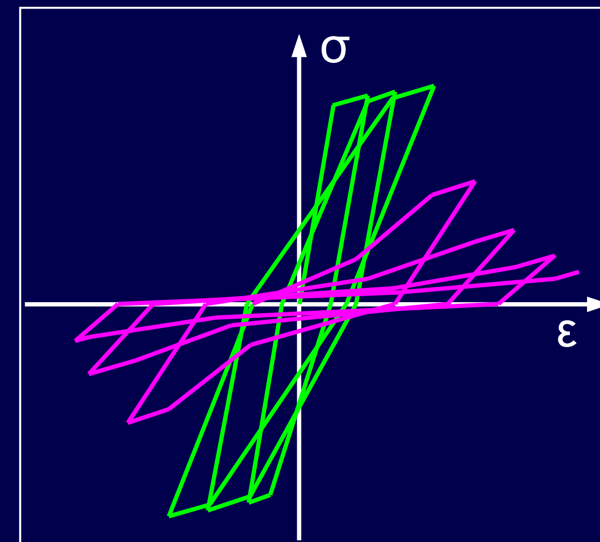


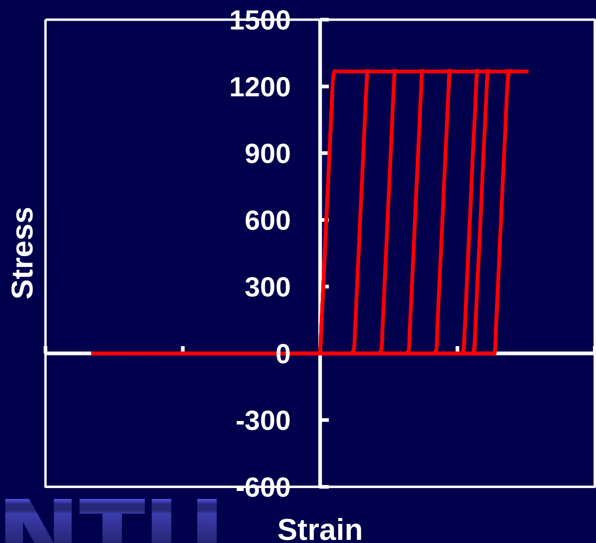
Constrain out of plane deformations

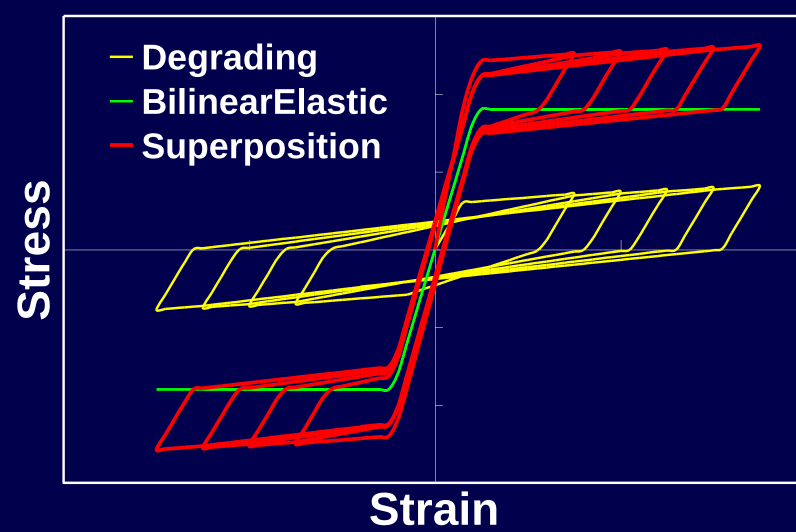
PISA3D程式庫 – 結構材料





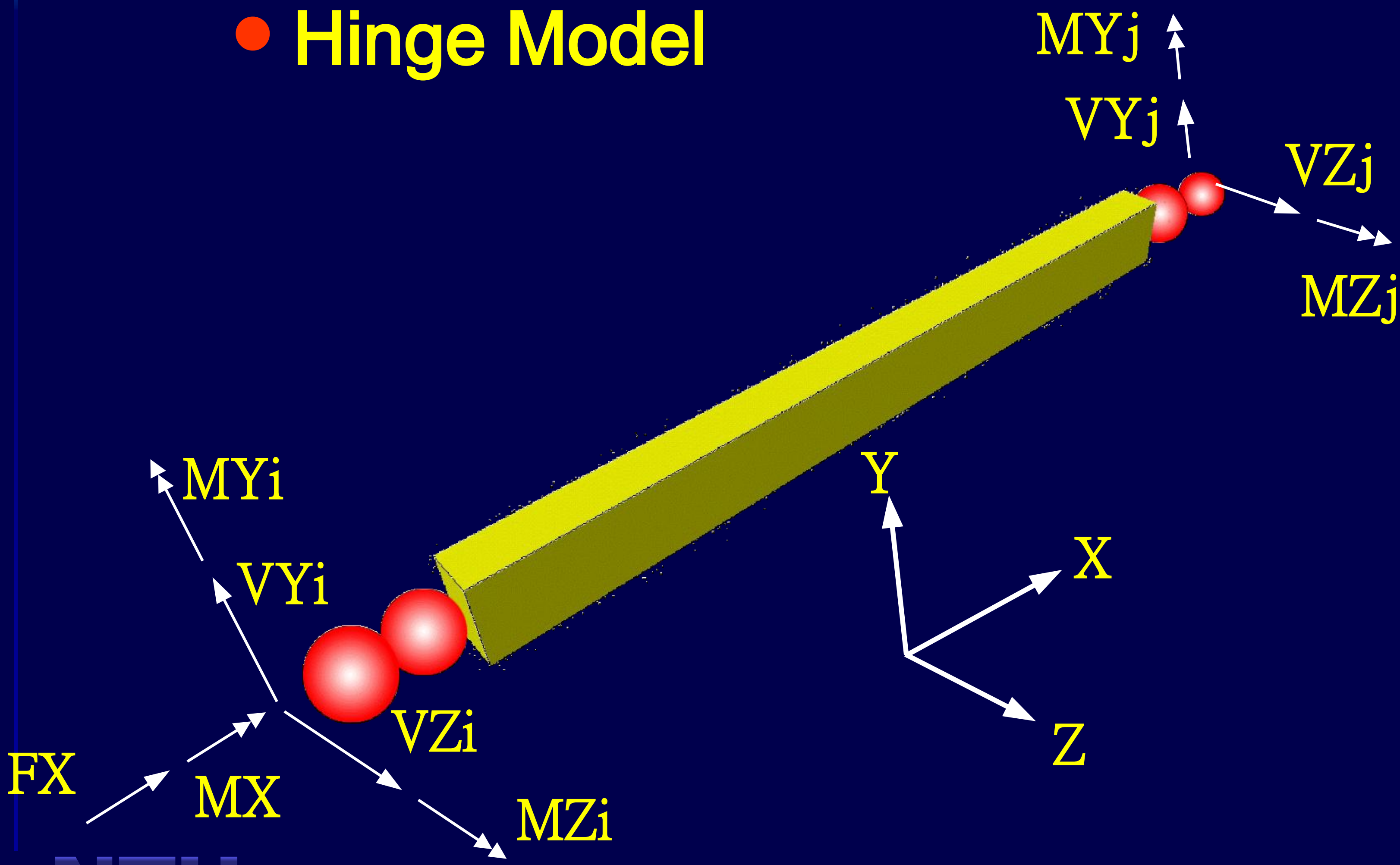




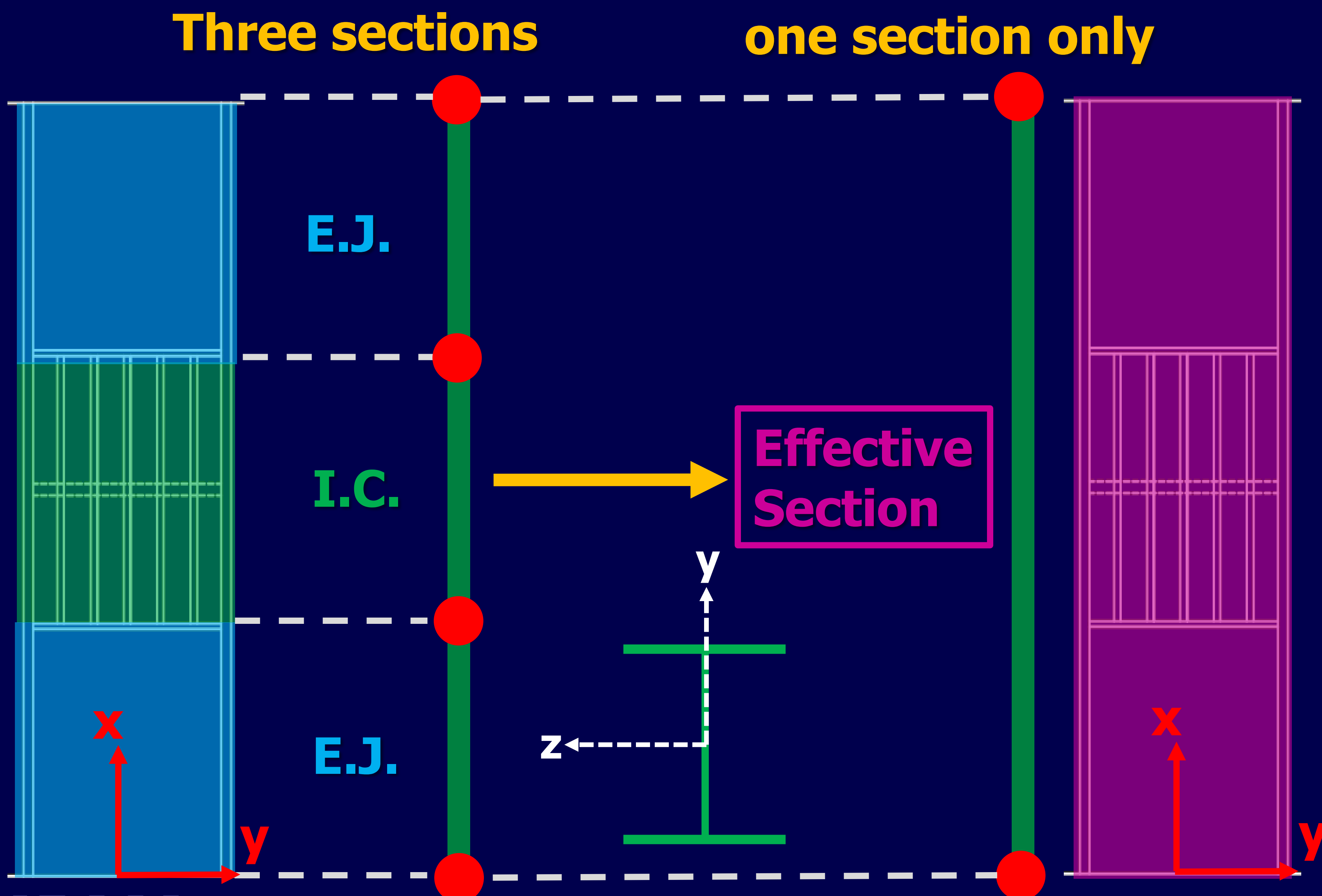


PISA3D BeamColumn Element

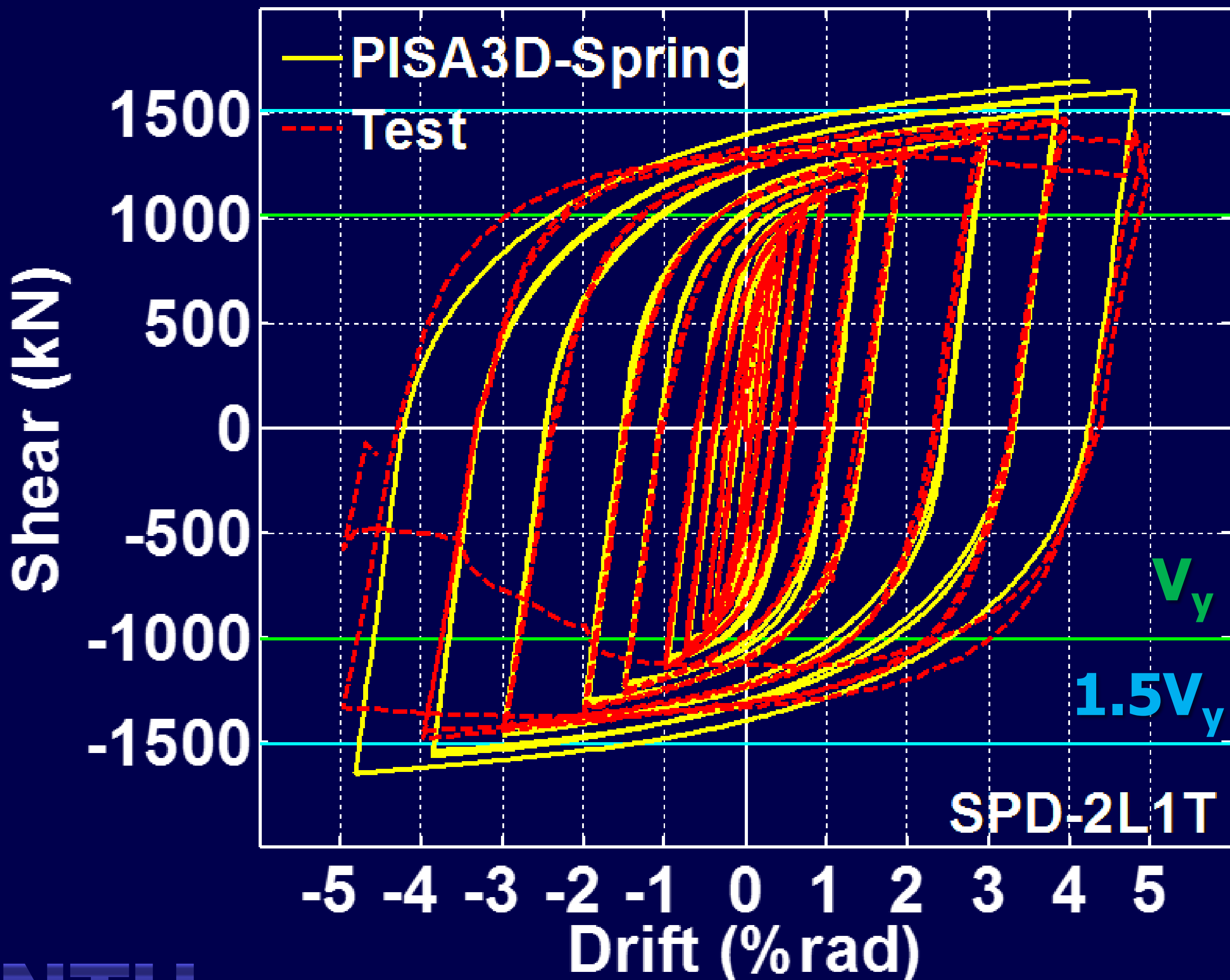
- Hinge Model



PISA3D中SPD模型簡化方法



PISA3D可供模擬試驗結果



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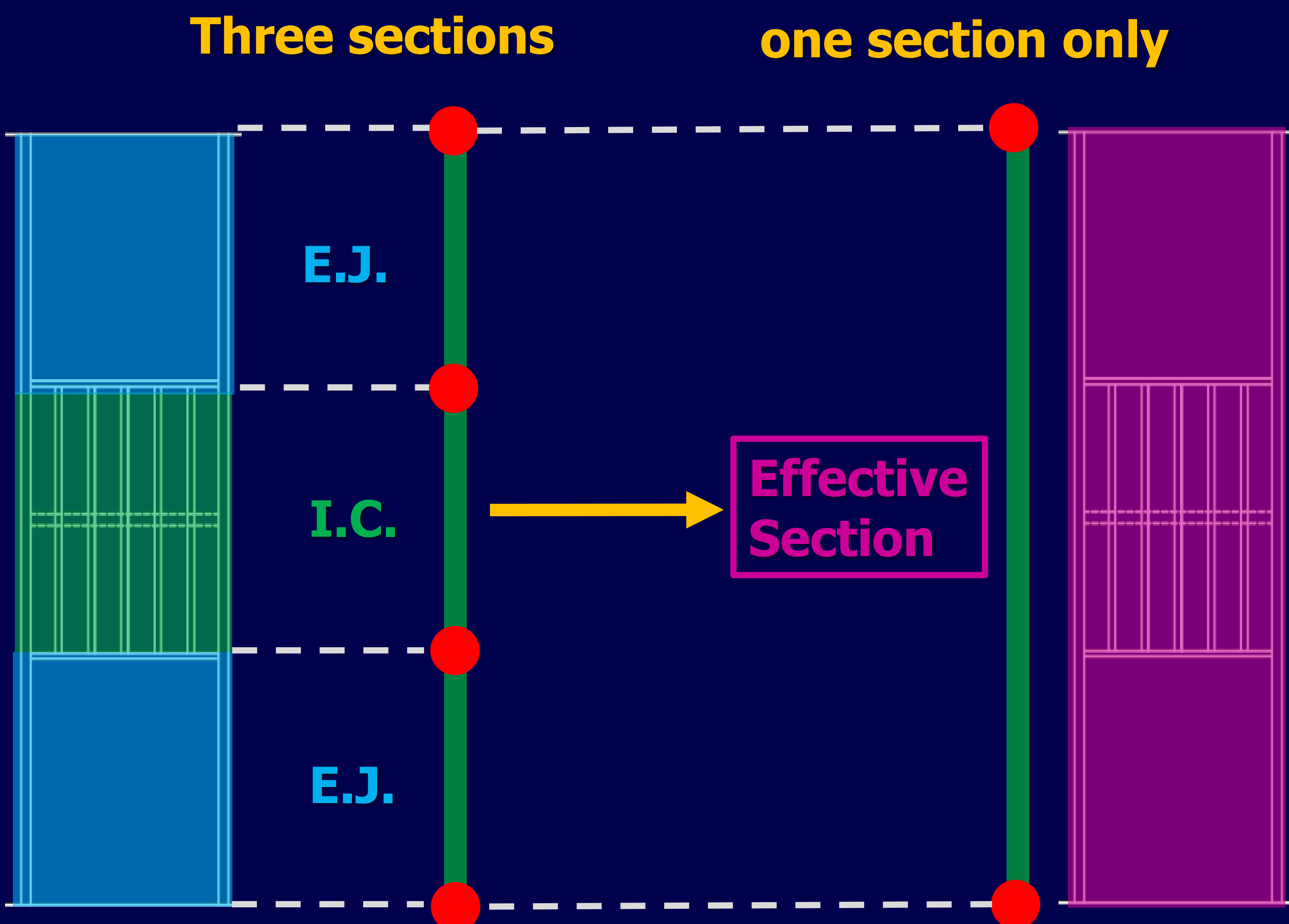

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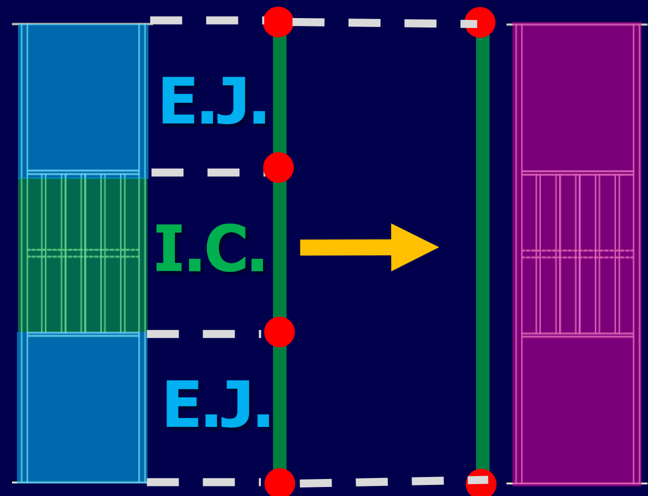


1. Effective Stiffness
2. Yielding Force

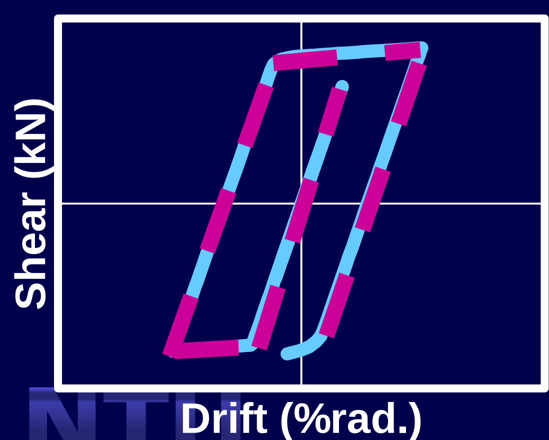
$$A_{vy,\mathrm{Eff}} = Q_{vy} A_{vy,IC}$$

One Element

Three Elements

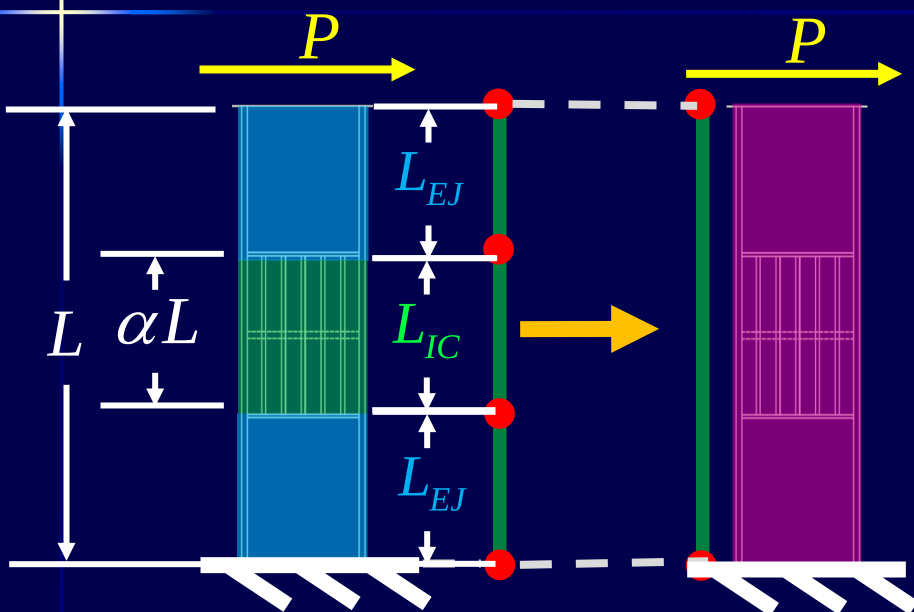
$A_{vy,Eff}$: shear area of y axis

Q_{vy}: effective coefficient of shear area



$A_{axial,Eff}$	$A_{vy,Eff}$	$A_{vz,Eff}$	$J_{,Eff}$	$I_{y,Eff}$	$I_{z,Eff}$
Q_A	Q_{vy}	Q_{vz}	Q_J	Q_{My}	Q_{Mz}

等效剪力面積

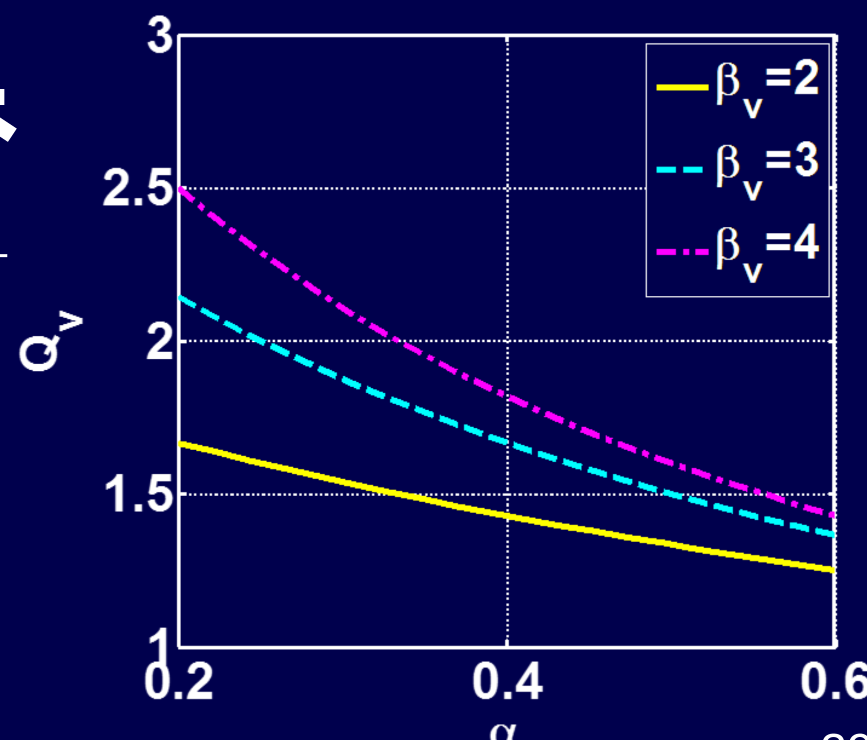


$$Q_{vy} = \frac{\beta_{vy}}{1+\alpha(\beta_{vy}-1)}$$

$$A_{vy,\mathrm{Eff}} = Q_{vy} A_{vy,IC}$$

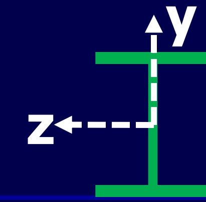
$$\Delta_{Shear} = \frac{2PL_{EJ}}{GA_{\mathrm{vy,EJ}}} + \frac{PL_{\mathrm{IC}}}{GA_{\mathrm{vy,IC}}} = \frac{PL}{GA_{vy,\mathrm{Eff}}}$$

$$\beta_{vy} = \frac{A_{\mathrm{vy,EJ}}}{A_{vy,IC}} \qquad \alpha = \frac{L_{IC}}{L}$$

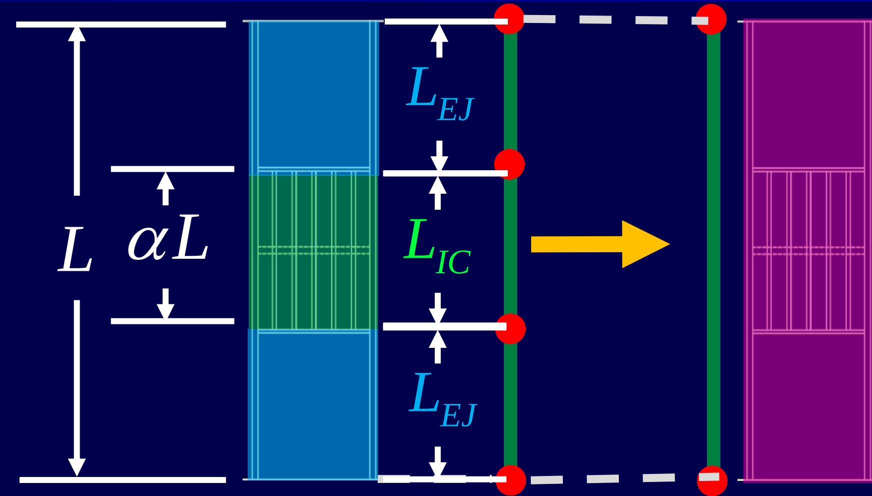


NCREE NTU

等效斷面



$A_{vy,Eff}$	$A_{vz,Eff}$	$A_{axial,Eff}$	$J_{,Eff}$
Q_{vy}	Q_{vz}	Q_A	Q_J
Series System			



$$A_{vy,\mathrm{Eff}} = Q_{vy} A_{vy,IC}$$

$$Q_{vy} = \frac{\beta_{vy}}{1+\alpha(\beta_{vy}-1)}$$

$$\beta_{vy} = \frac{A_{vy,\mathrm{EJ}}}{A_{vy,IC}}$$

$$A_{\mathrm{axial,Eff}} = Q_A A_{\mathrm{axial},IC}$$

$$Q_A = \frac{\beta_A}{1+\alpha(\beta_A-1)}$$

$$\beta_A = \frac{A_{\mathrm{axial,EJ}}}{A_{\mathrm{axial},IC}}$$

$$J_{\mathrm{Eff}} = Q_J J_{IC}$$

$$Q_J = \frac{\beta_J}{1+\alpha(\beta_J-1)}$$

$$\beta_J = \frac{J_{\mathrm{EJ}}}{J_{IC}}$$

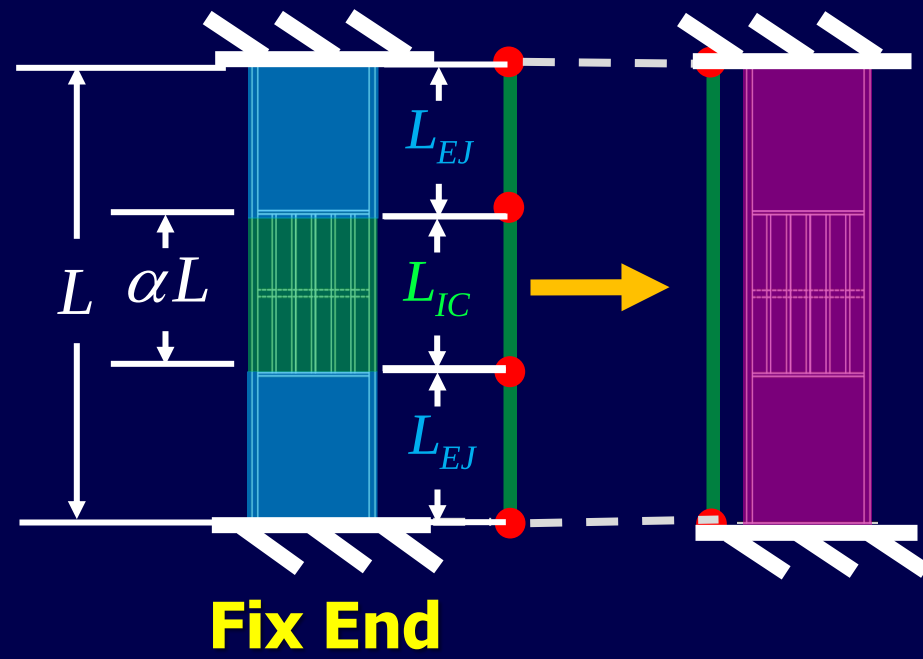
等效斷面慣性矩

$I_{y,Eff}$	$I_{z,Eff}$
Q_{my}	Q_{mz}

$$\beta_{mz} = \frac{I_{z,EJ}}{I_{z,IC}}$$

$$I_{z,Eff} = Q_{mz} I_{z,IC}$$

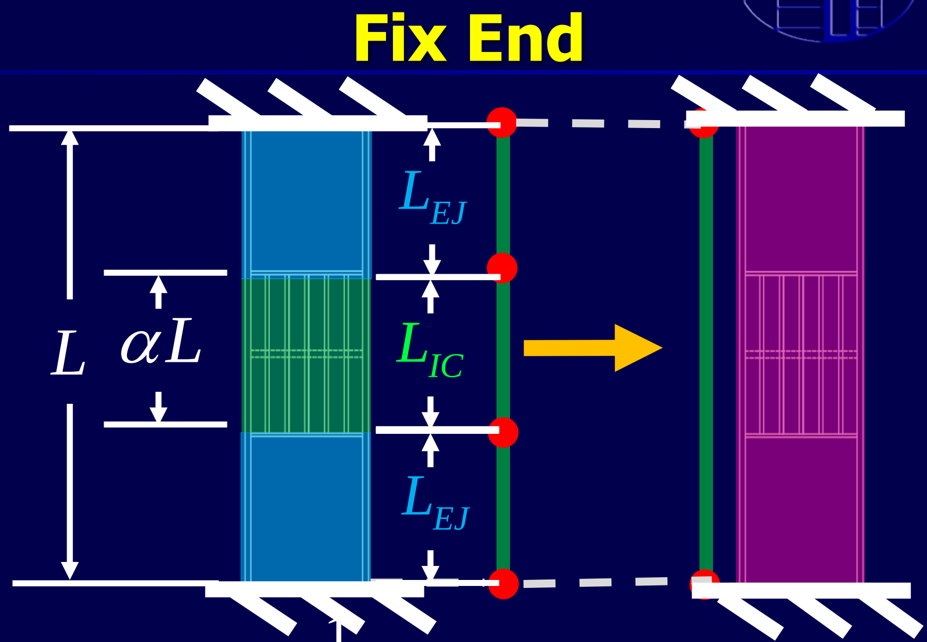
$$Q_{mz} = \frac{\beta_{mz}}{1+\alpha^3(\beta_{mz}-1)}$$



$$\Delta_{Moment} = 2\int \frac{Mm}{EI_{EJ}} dx + \int \frac{Mm}{EI_{IC}} dx = \int \frac{Mm}{EI_{Eff}} dx$$

NCREE NTU

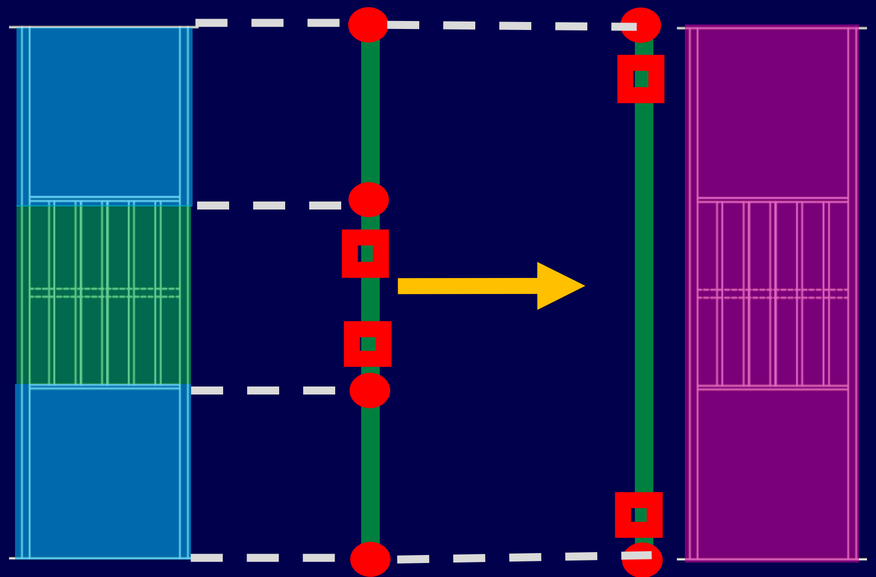
等效勁度



$$K_{eff} = \frac{1}{1/K_{IC} + 2/K_{EJ}} = \frac{1}{\left[\dfrac{\alpha L}{GA_{V,IC}} + \dfrac{(\alpha L)^3}{12EI_{IC}}\right] + \left[\dfrac{(1-\alpha)L}{GA_{V,EJ}} + \dfrac{L^3-(\alpha L)^3}{12EI_{EJ}}\right]}$$

$$= \frac{1}{\dfrac{L}{\mathrm{G}(A_{\mathrm{vy,eff}})} + \dfrac{L^3}{12\mathrm{E}(I_{z,eff})}} = \frac{1}{\dfrac{L}{\mathrm{G}(Q_{vy}A_{\mathrm{vy},IC})} + \dfrac{L^3}{12\mathrm{E}(Q_{mz}I_{z,IC})}}$$

等效剪力降伏應力與塑鉸位置



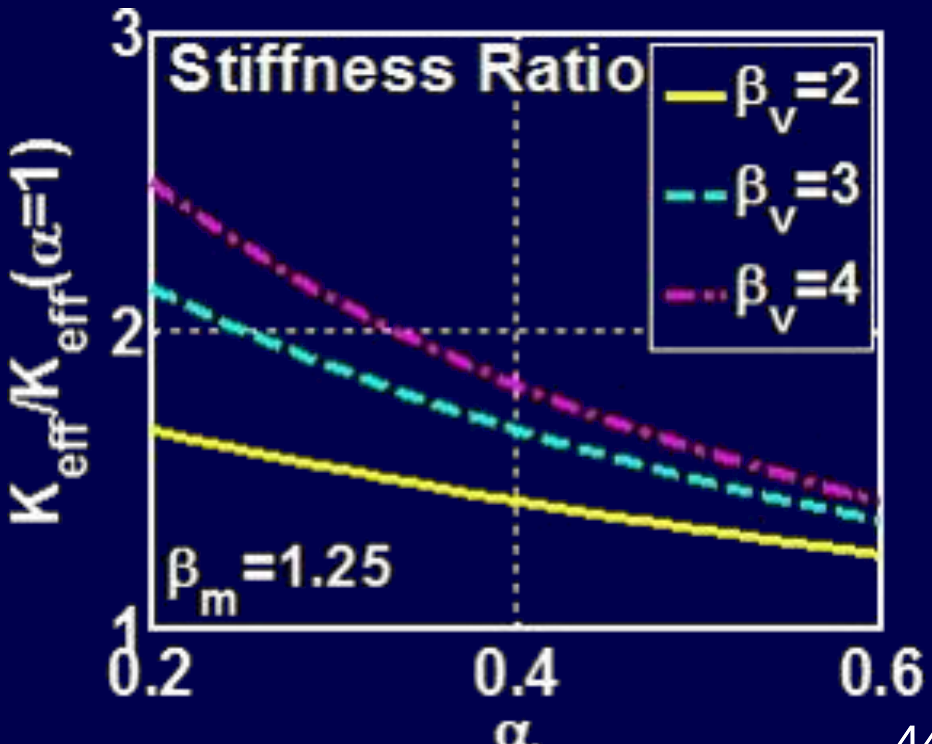
Shear Yielding

$$V_y = A_{vy,IC}\,0.6F_{y,core} = A_{vy,eff}\,0.6F_{y,eff}$$

$$F_{y,Eff} = \frac{A_{vy,IC}}{A_{vy,eff}} F_y = \frac{1}{Q_{vy}} F_{y,core}$$

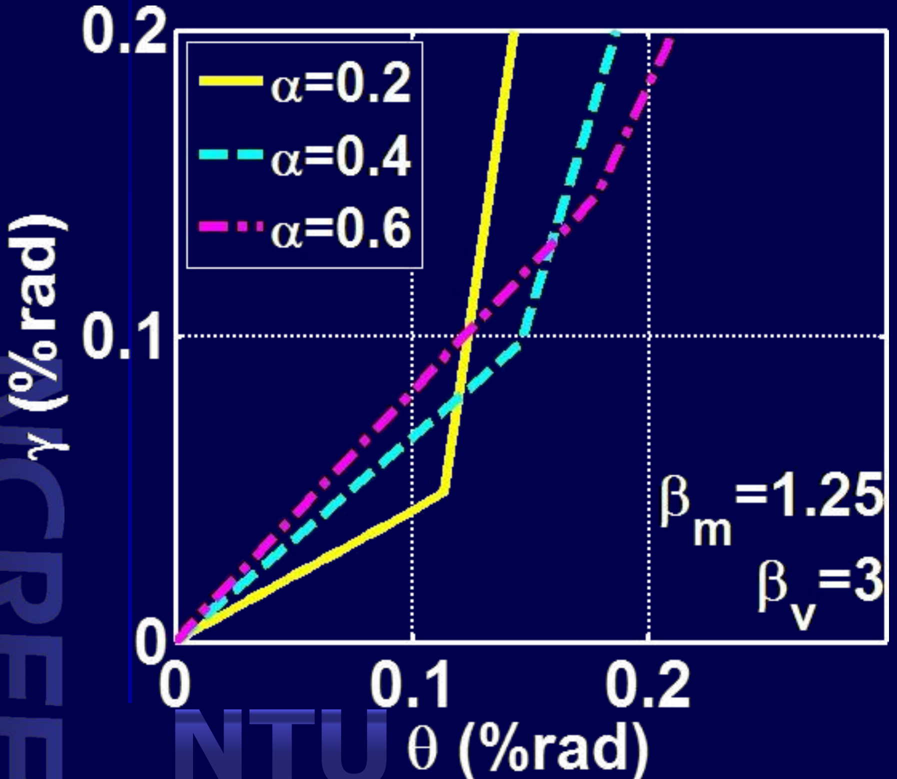
核心段相對長度對有效勁度之效應

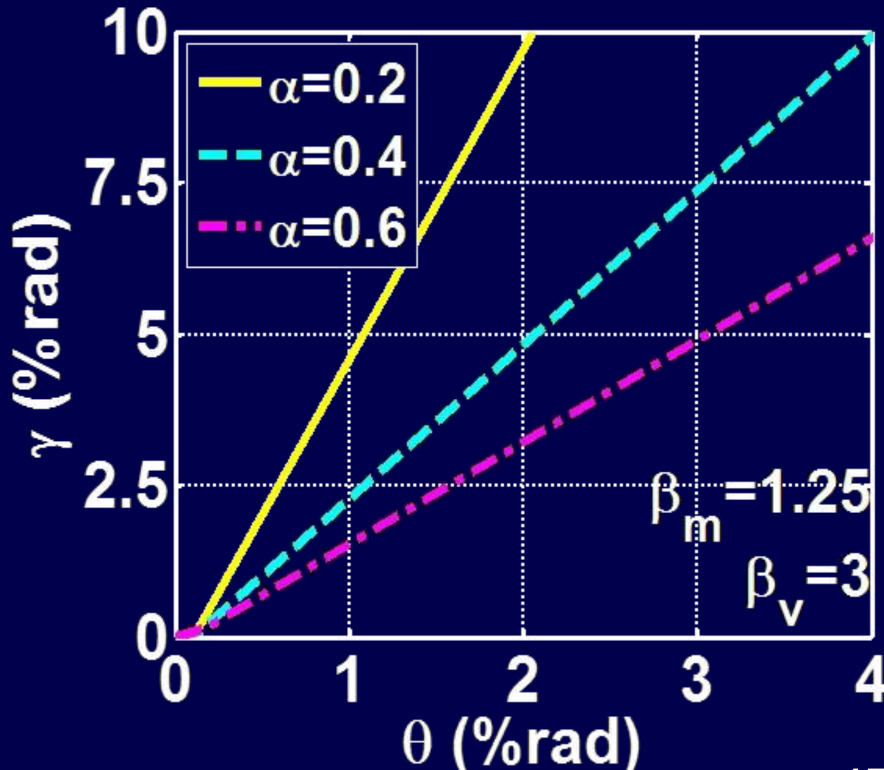
- 縱軸為彈性勁度介於0.2至0.6與全高皆為核心段時(=1)之比值，橫軸為核心段相對長度
- 隨著核心段相對長度縮短與連接段斷面剛度上升，SPD彈性勁度可增加150%
- 可用增加連接段長度與斷面剛度來有效增加SPD之勁度而保持強度不變



核心段相對長度與核心段變形關係

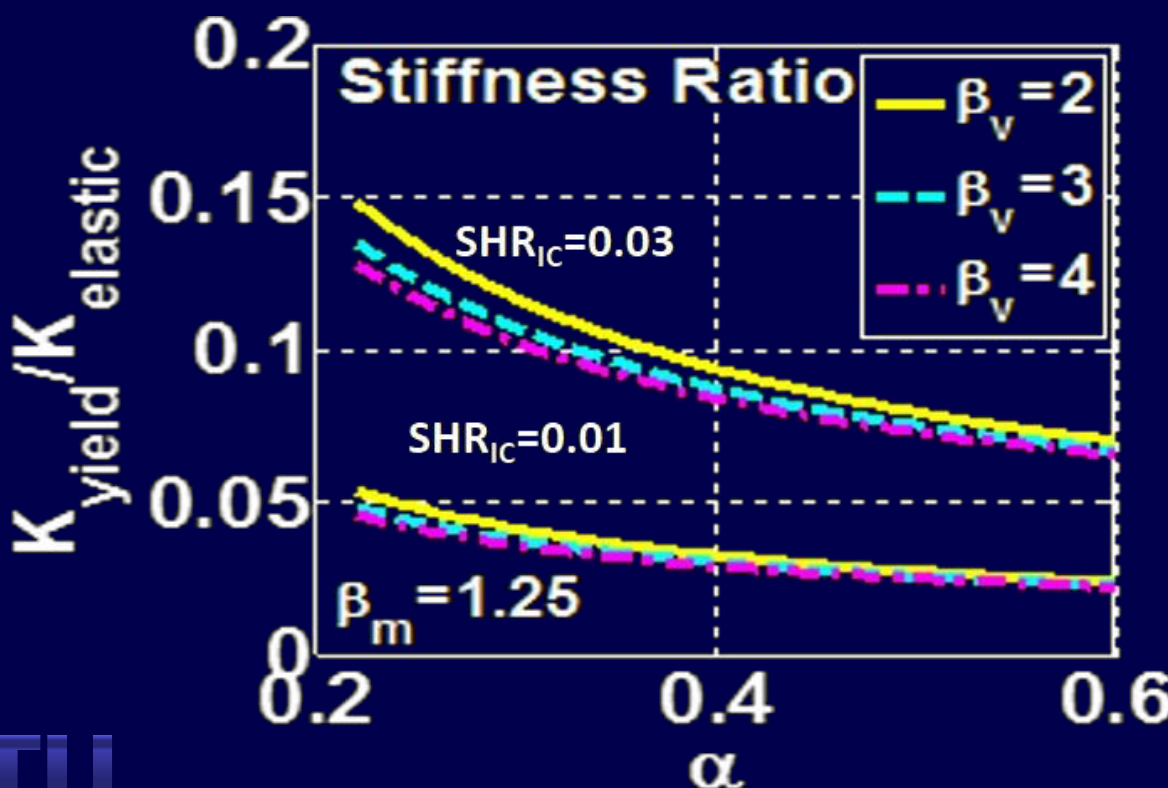
- 左圖為小變形時核心段變形與整體側位移角關係，右圖則為大變形
- 在大變形下顯示γ 與θ 比例約為1/α ，可假設SPD變形集中在核心段而忽略連接段之變形





核心段相對長度對整體應變硬化之效應

- 縱軸為降伏後與彈性勁度比值，橫軸為核心段相對長度
- SHR_{IC}為0.01時，整體SPD降伏後勁度比隨著核心段相對長度之減短從0.02上升至0.05
- SHR_{IC}為0.03時，整體SPD降伏後勁度比便可從0.07上升至0.15



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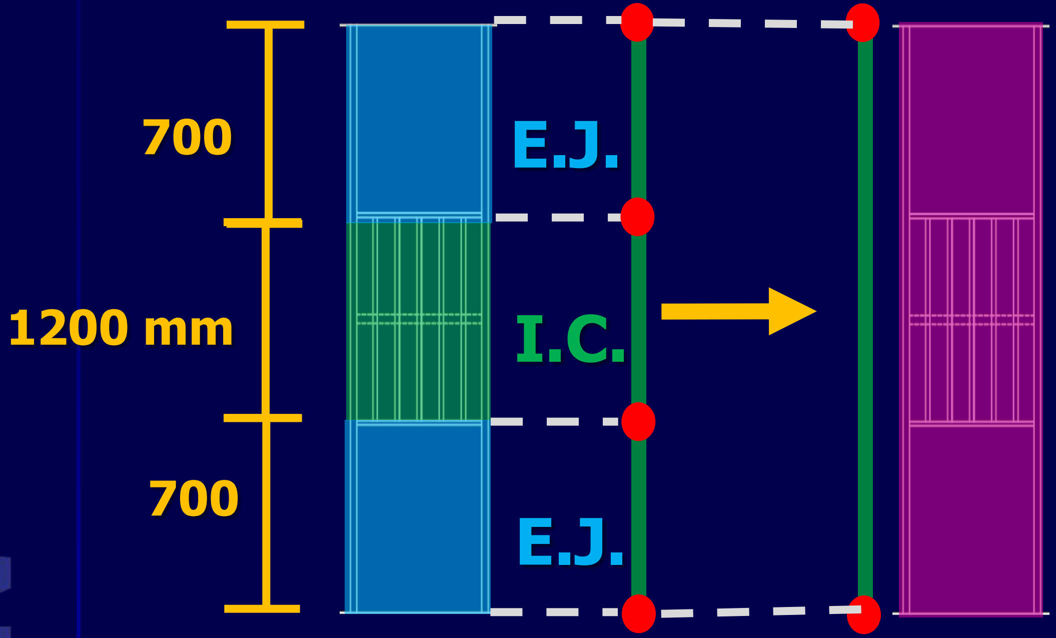
Example in PISA3D

IC: H900x250x7x30

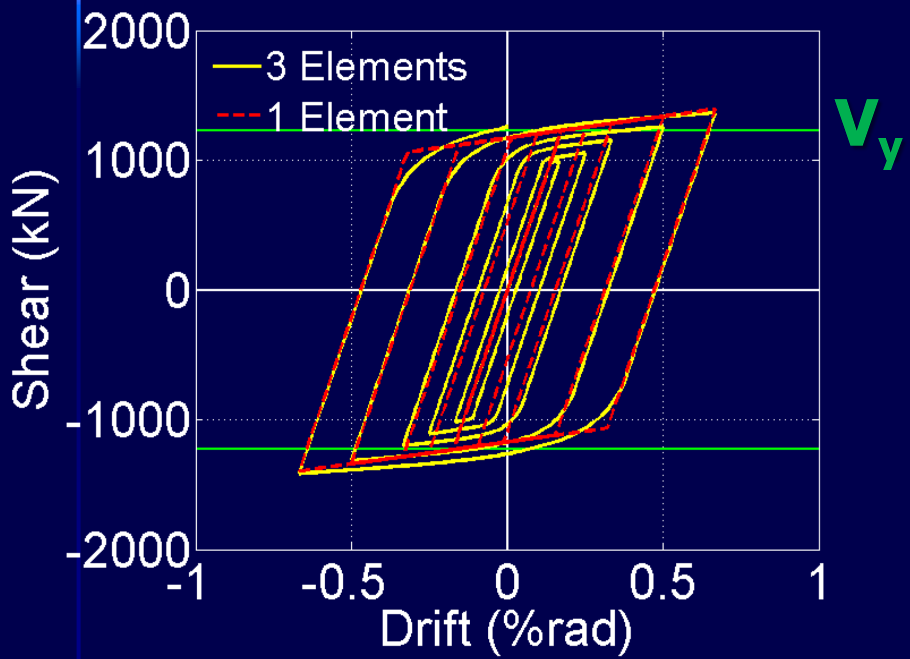
EJ: H900x250x26x30



$$\alpha = \frac{1200}{2600} = 0.46$$

α	Q_A	Q_{VY}	Q_{VZ}	Q_{MY}	Q_{MZ}
0.46	1.64	1.65	1.00	1.26	1.01

Stiffness between 1 & 3 Elements



3 element: hardening material

1 element: bilinear material

	PISA3D	
	3 Elements	1 Element
Elastic Stiffness (kN/mm)	196	196

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Summary

- 試驗結果證實SPD容量設計法可行，且加勁單元能延緩挫屈。
- 不配置橫向加勁板，SPD-2L1T與SPD-2L0T的韌性容量相差不大。
- 由簡化單根模型推導的等效斷面，可方便工程師使用。
- PISA3D與Abaqus模擬SPD之遲滯行為具有不錯的準確性，且Abaqus能模擬挫屈之時機。

↓

SPD-MRF之耐震設計與分析

簡報大綱

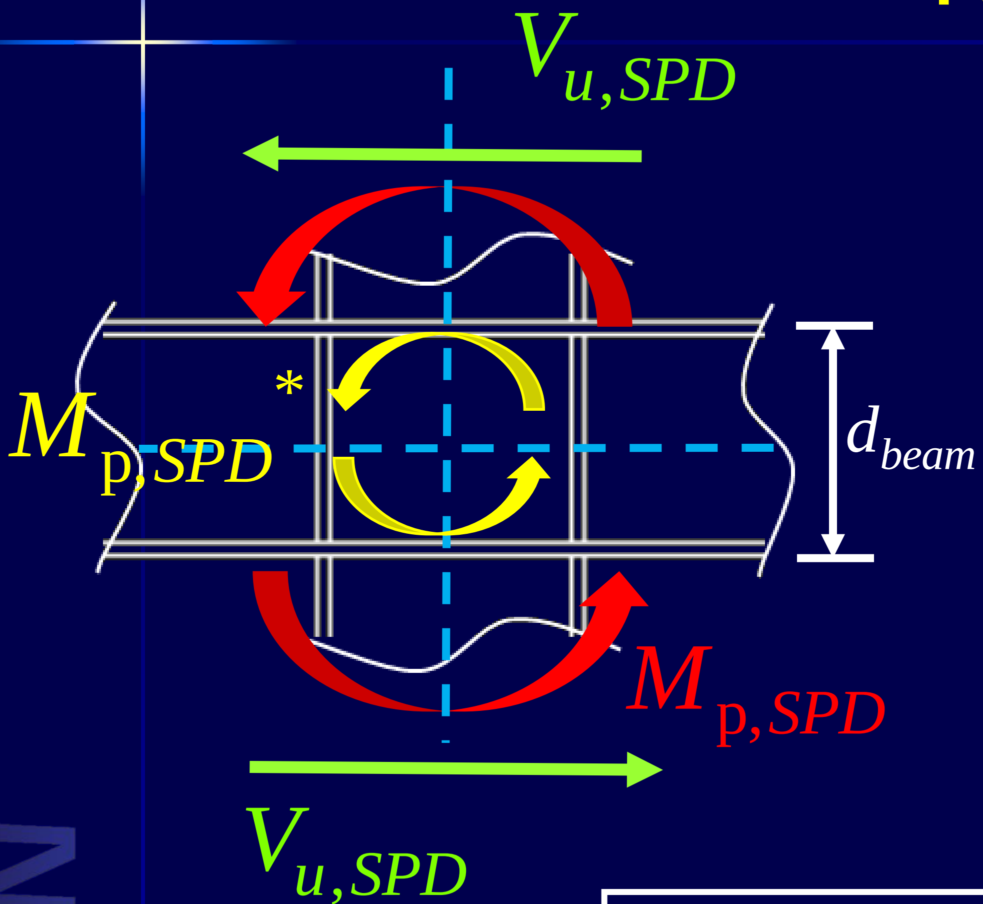
- SPD試體設計
- 試驗配置
- 試驗結果
- 試驗與分析結果比較
- SPD模型簡化方法
- SPD邊界梁設計方法
- 6層樓SPD-MRF
- 6-Story SPD-MRF分析結果
- 結論

SPD

SPD-MRF

NCREE NTU

計算 $M_{p,SPD}^{*}$



$$M_{p,SPD} = V_{u,SPD}\frac{L_{SPD}}{2} = \omega R_y V_y \frac{L_{SPD}}{2}$$

ω → 1.1

$$M_{p,SPD}^{*} = M_{p,SPD} + V_{u,SPD} \times \frac{d_{beam}}{2}$$

梁-SPD交會區剪力需求

Shear

$$\phi V_n \geq V_{pz}$$

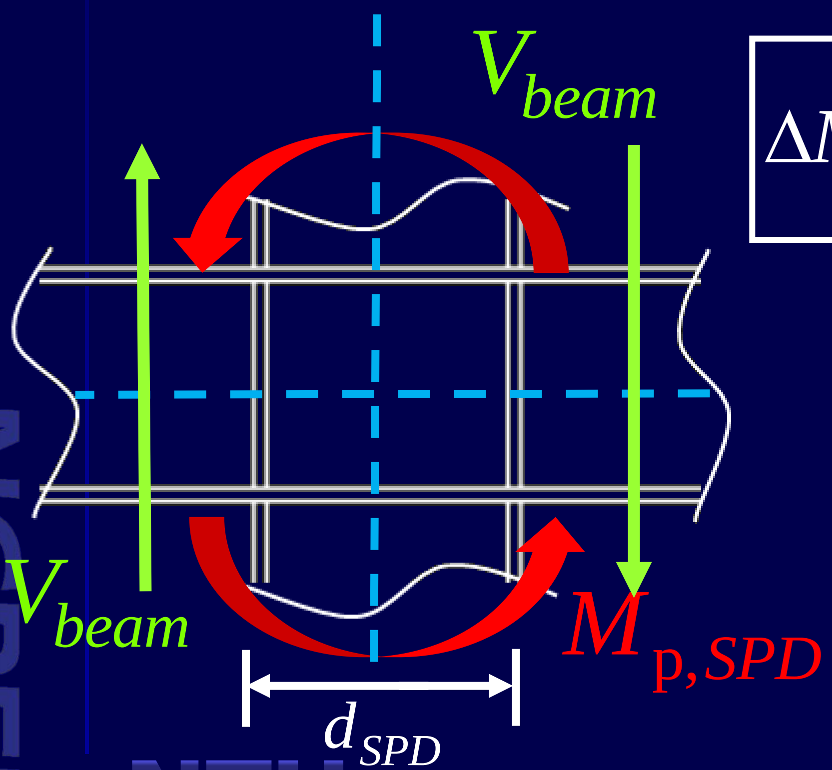
$$V_{pz} = \frac{\Delta M}{(d_{SPD} - t_f)} - V_{beam}$$

$$\Delta M = \sum M_{P,SPD} = \sum \omega R_y V_y \frac{L_{SPD}}{2}$$

1.1

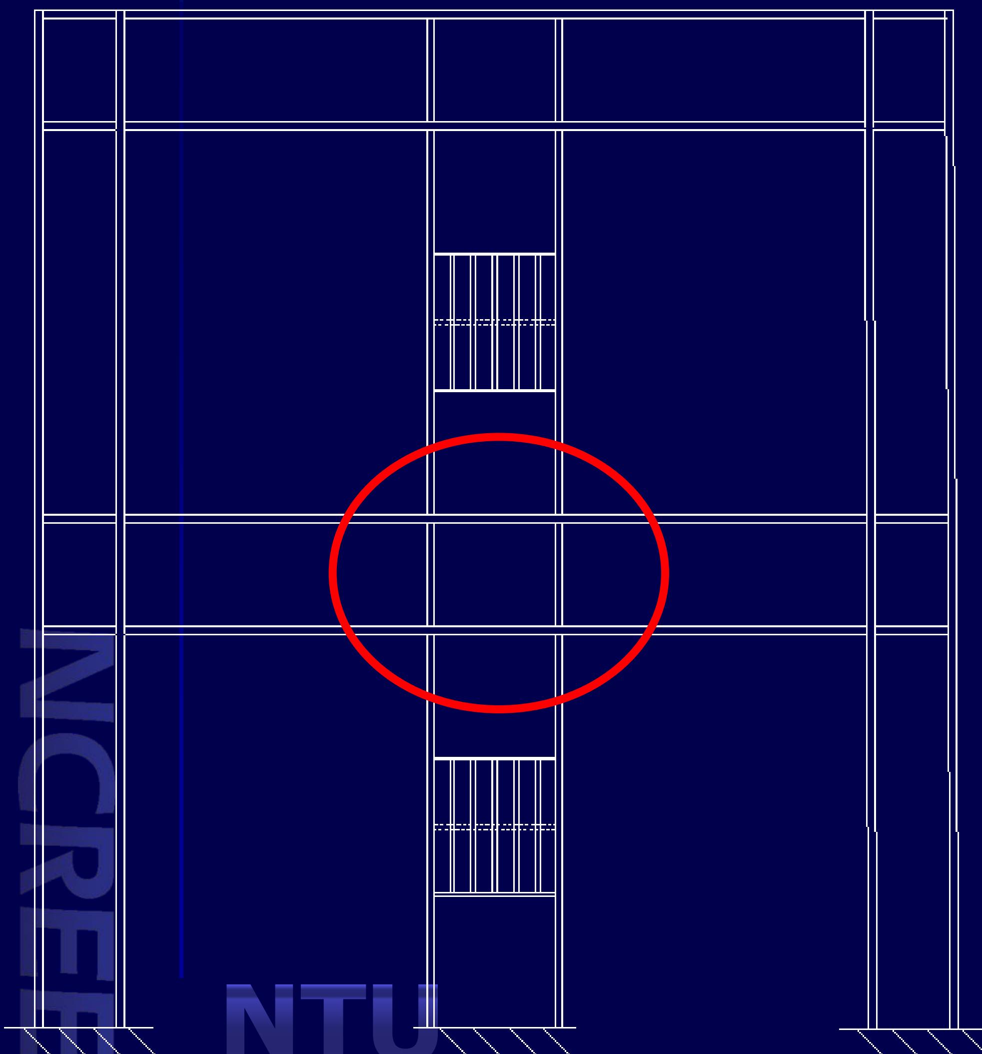
$$V_n = 0.6 F_y d_b t_{wb}$$

$$\phi = 1.0$$

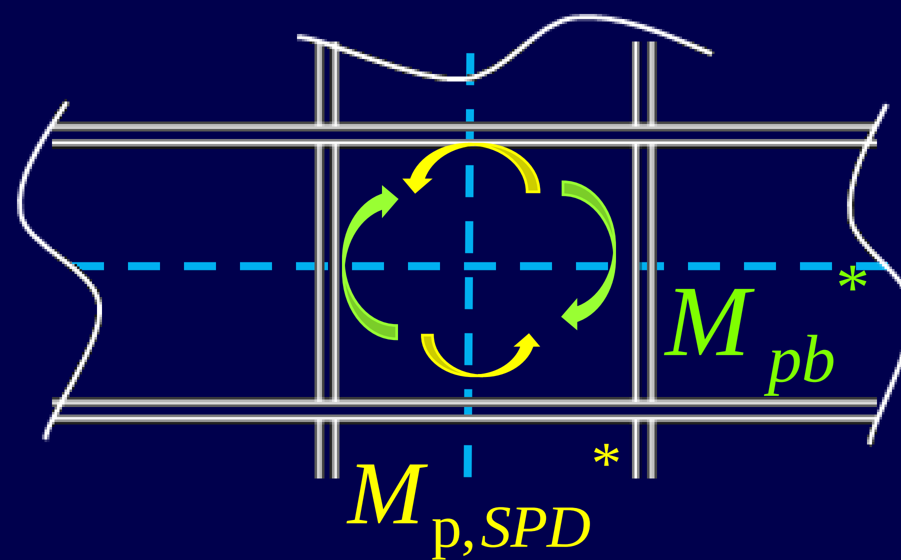


邊界梁與SPD彎矩比例

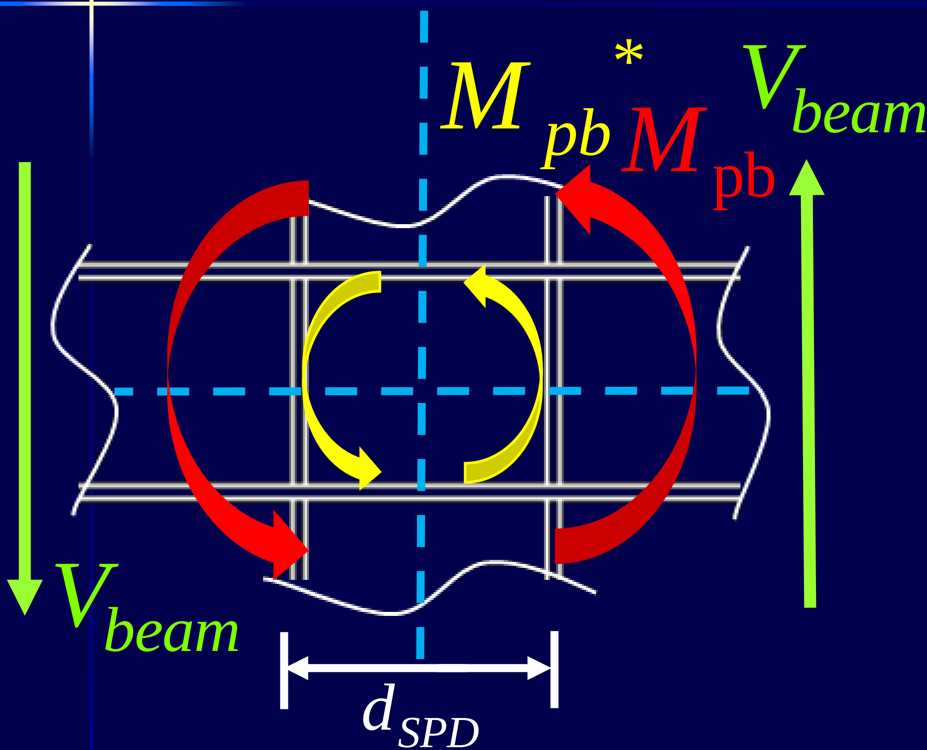
SPDs are separated by beam



$$\frac{M_{pb}^{*}}{M_{p,SPD}^{*}} \geq 1.0$$



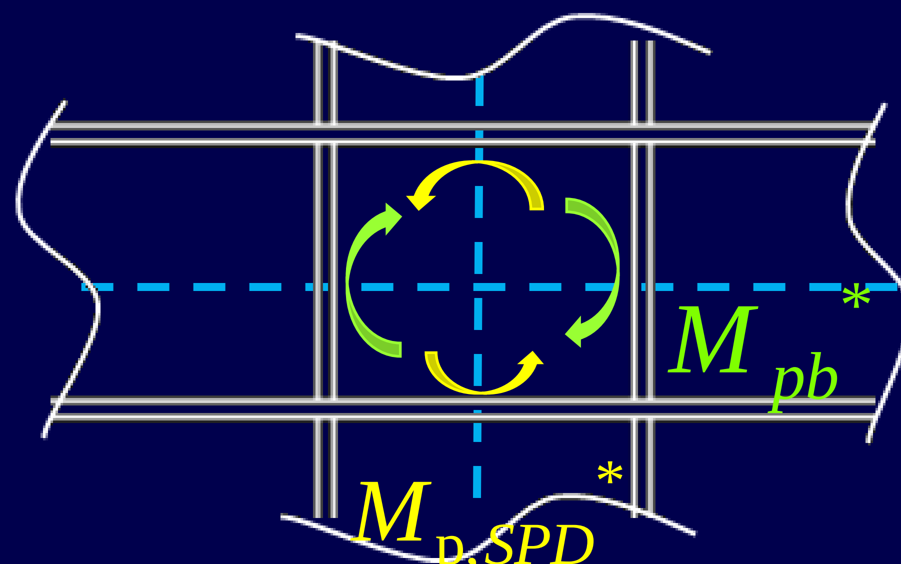
計算 M_{pb}^{*}



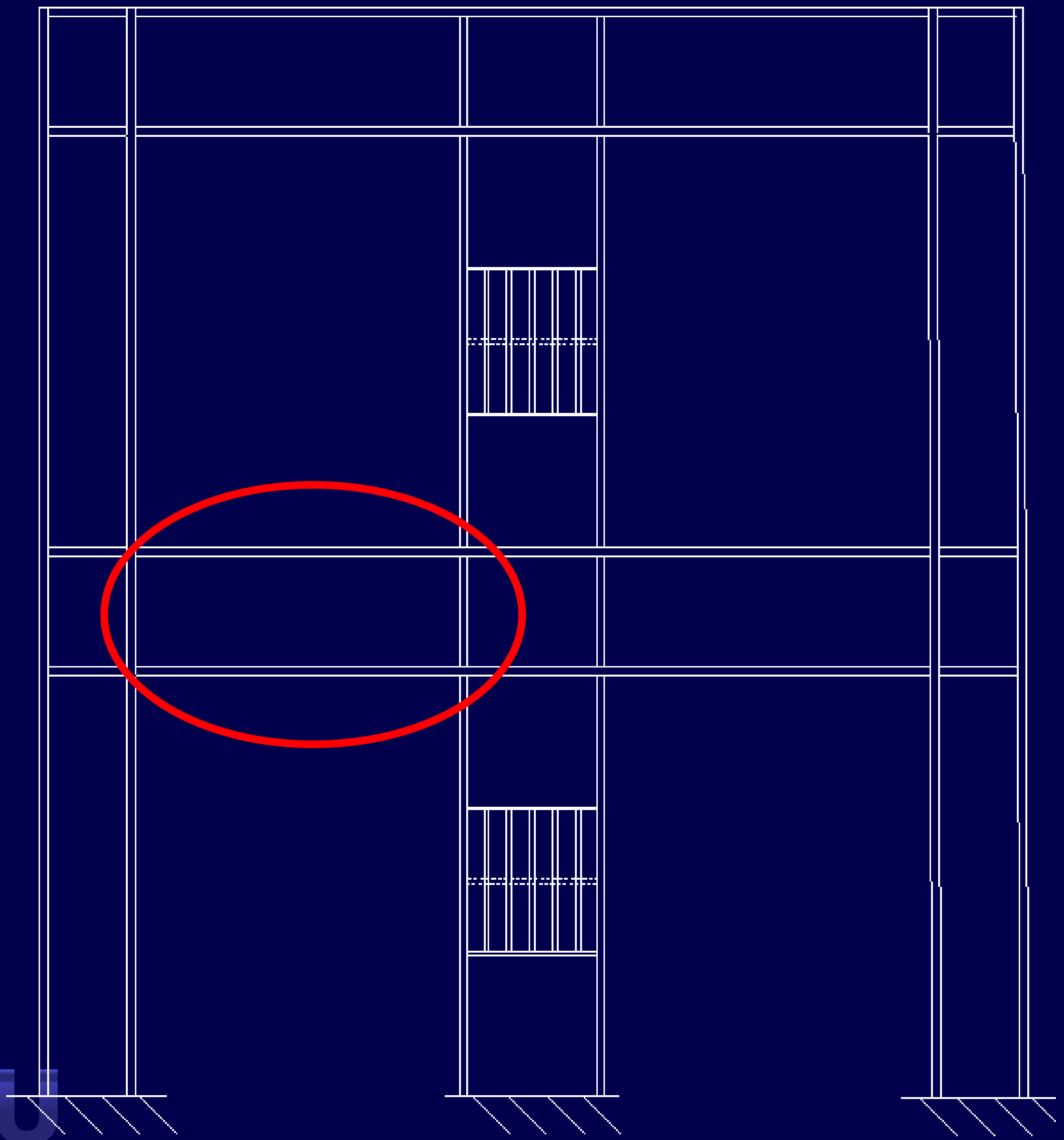
$$M_{pb} = Z_b F_y$$

$$M_{pb}^{*} = M_{pb} + V_{beam} \times \frac{d_{SPD}}{2}$$

$$\frac{M_{pb}^{*}}{M_{p,SPD}^{*}} \geq 1.0$$



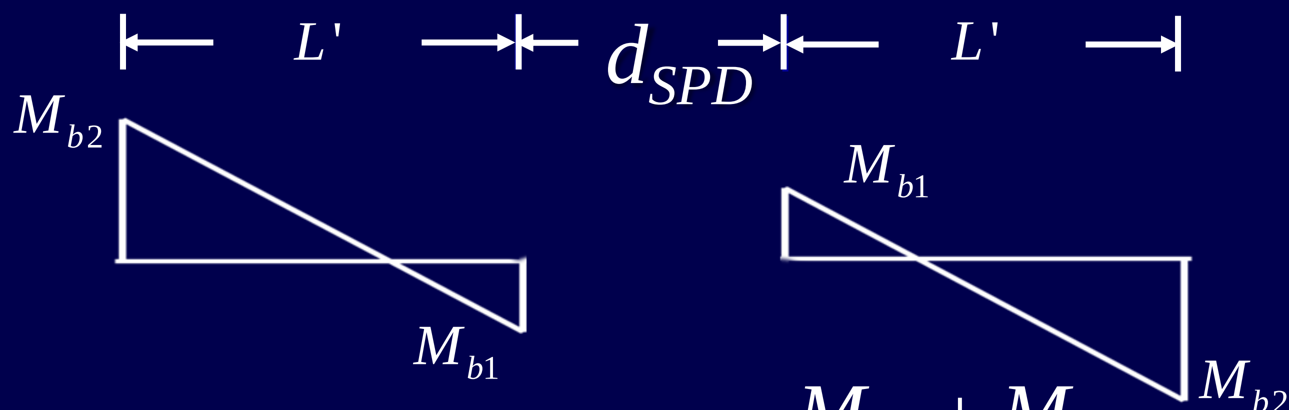
梁之剪力容量



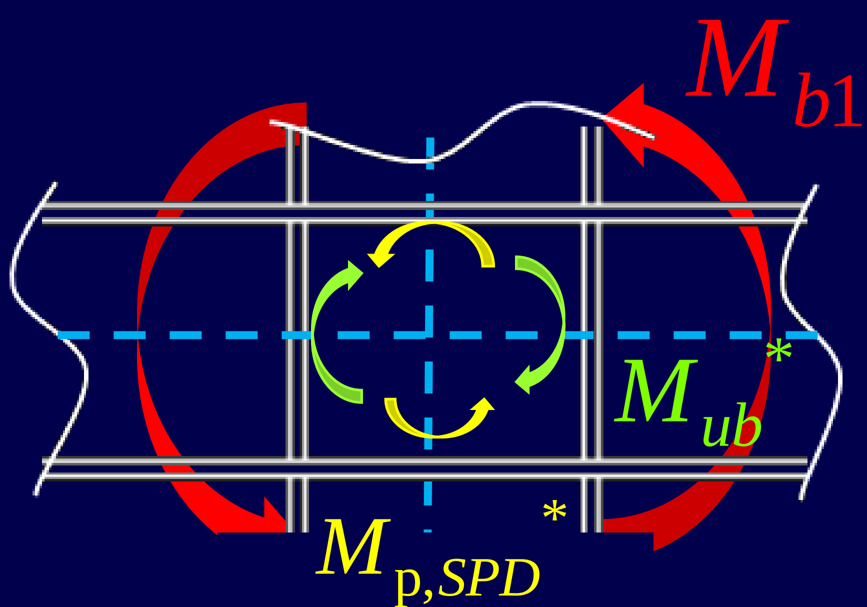
梁之剪力容量

$$\phi V_{nb} \geq V_{ub}$$

$\phi = 0.9$



$$V_{ub} = \frac{M_{b1} + M_{b2}}{L'}$$



$$M_{ub}^{*} = M_{p,SPD}^{*}$$

$$M_{b1} = M_{p,SPD}^{*} - V_{beam} \times \frac{d_{SPD}}{2}$$

$$M_{b2} = \omega Z_b F_y$$

ω → 1.1

簡報大綱

- SPD試體設計
- 試驗配置
- 試驗結果
- 試驗與分析結果比較
- SPD模型簡化方法

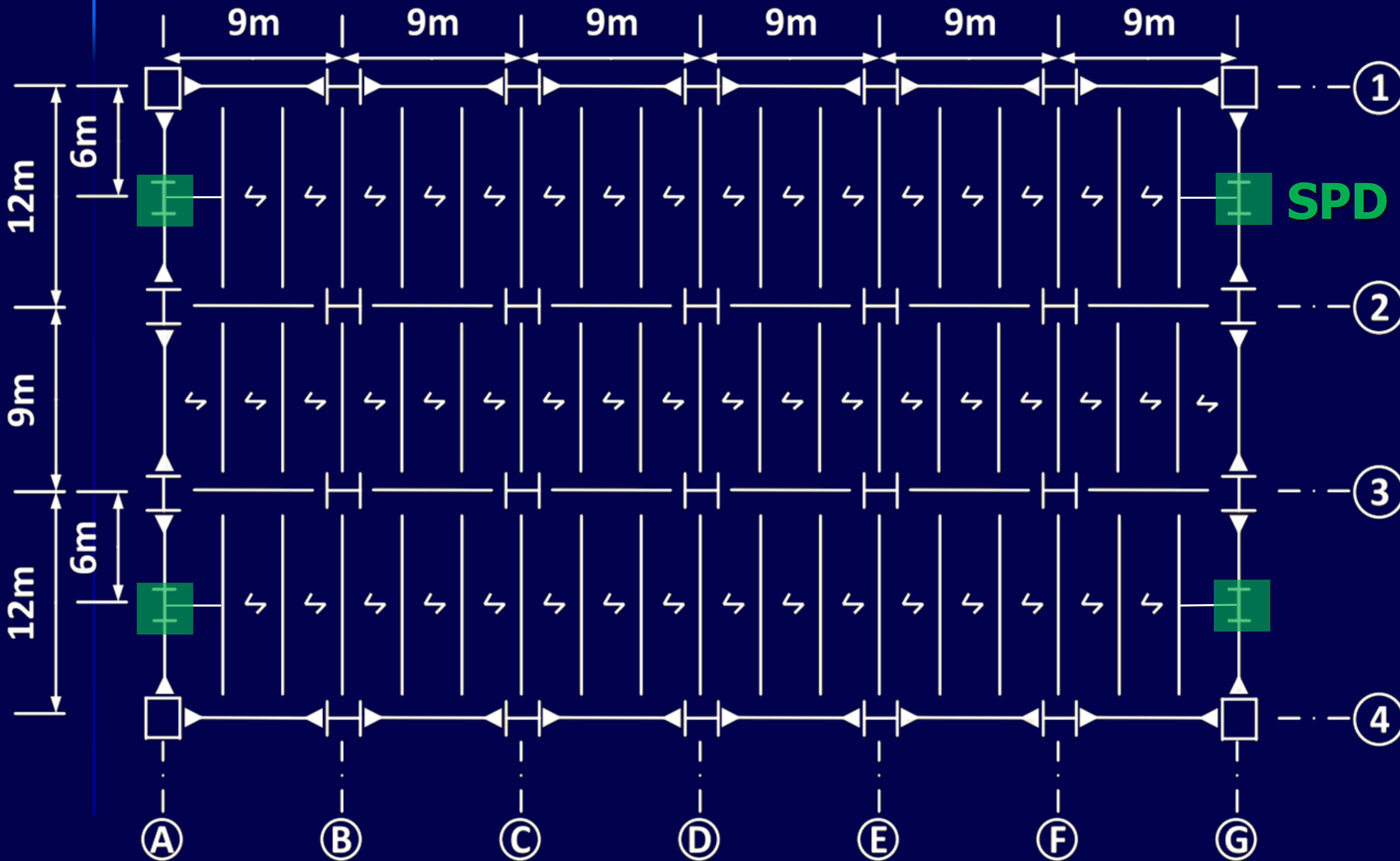
SPD

- SPD邊界梁設計方法
- 6層樓SPD-MRF
- 6-Story SPD-MRF分析結果

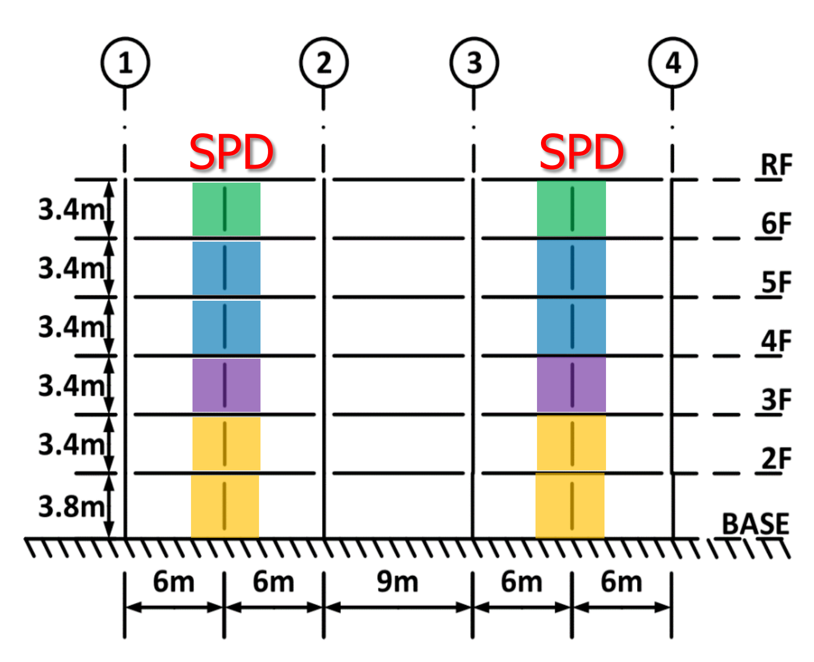
SPD-MRF

- 結論

樓層平面圖



樓層立面資訊

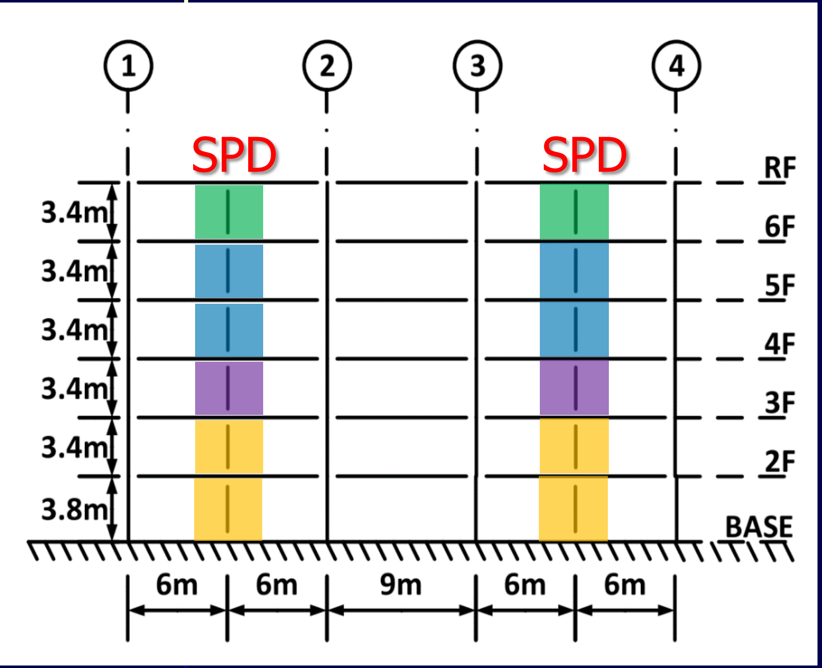


SPD	Core Section	Core Height
6 Story	H600x250x4x16	1200
4~5	H600x250x8x30	1800
3	H900x250x6x22	900
1~2	H900x250x7x30	1200

Story	Corner Column	Column	Beam
6	B400x400x37	H500x300x25x40	H600x300x14x23
3~5	B450x450x37	H500x450x24x46	H800x300x10x22
1~2	B500x500x40	H600x400x25x50	H800x300x14x26

Unit: mm

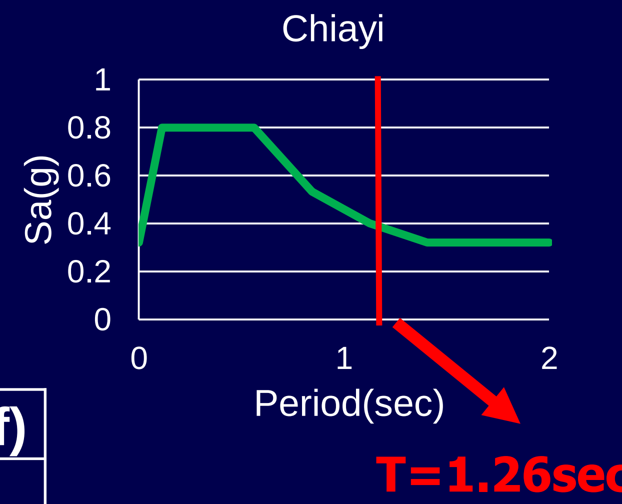
梁-SPD交會區設計結果



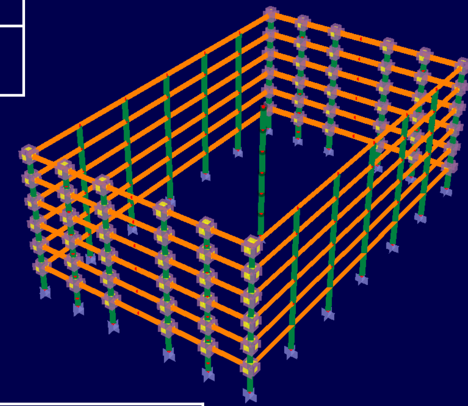
Story	Thickness (mm)		Capacity (kN)	Demand (kN)	DCR
	Doubler Plates	Web			
6	3	14	1998	2099	0.95
5	28	10	6148	6256	0.98
4	40	10	8198	8232	1.00
3	24	10	5502	5598	0.98
2	25	14	6374	6421	0.99
1	31	14	7268	7409	0.98

靜力分析方法與設計結果

- Building site: 嘉義市東區
- I(用途係數): 1.0
- R=4.8 for SPD-MRF



Floor	Area(m^2)	Dead Load(tf/m^2)	Weight(tf)
2~6F	1782	0.80	1426
RF	1782	0.60	1069

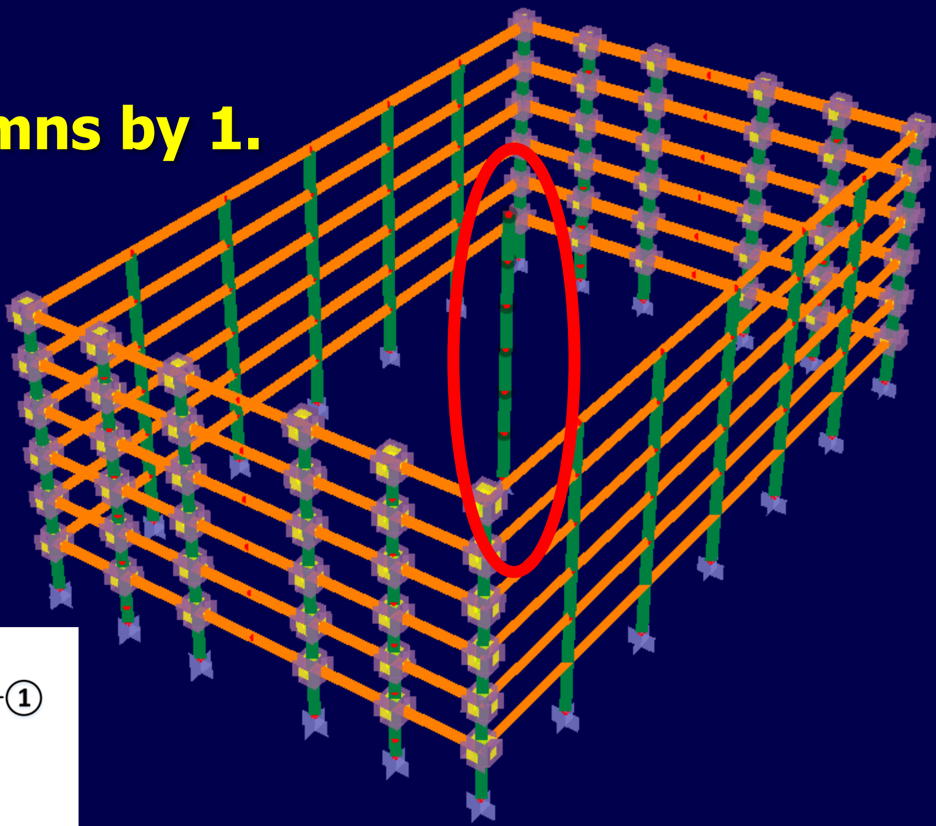


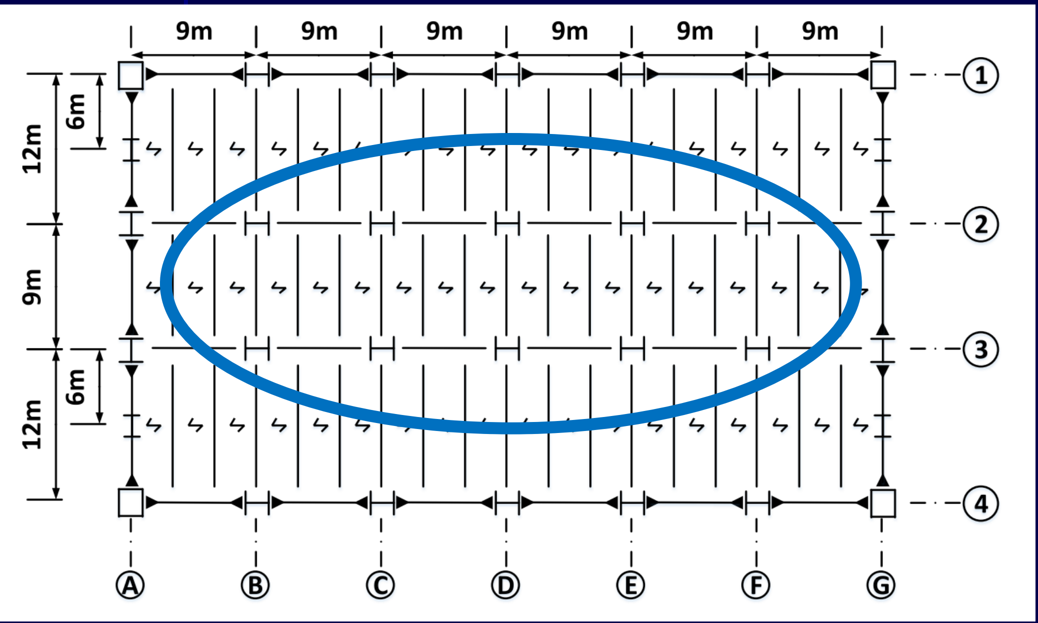
Periods (ETABS)	
Longitudinal	1.30sec
Transverse	1.26sec

Design Force	
Longitudinal	0.092W=754tf controlled by 中小度地震
Transverse	0.092W=754tf controlled by 中小度地震

Leaning Column

In order to reduce D.O.F.
Simulate 10 leaning columns by 1.



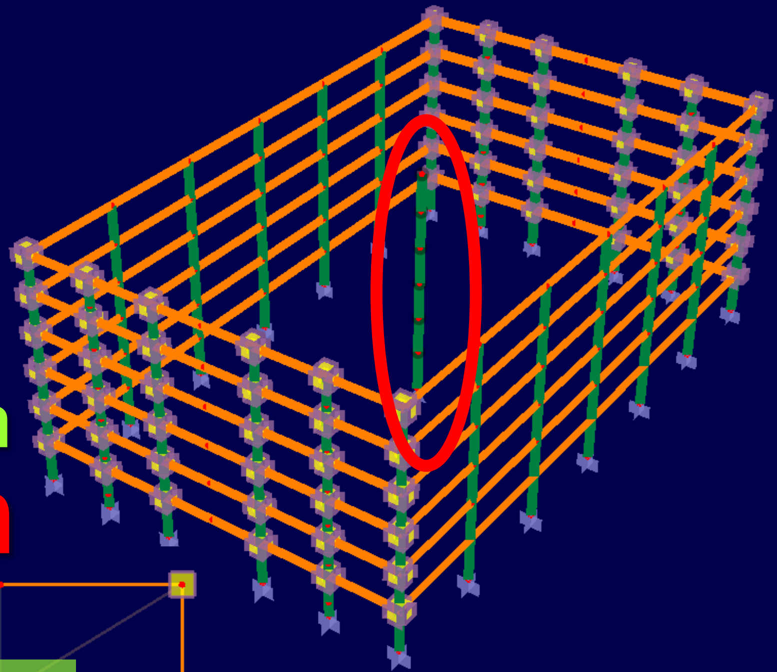


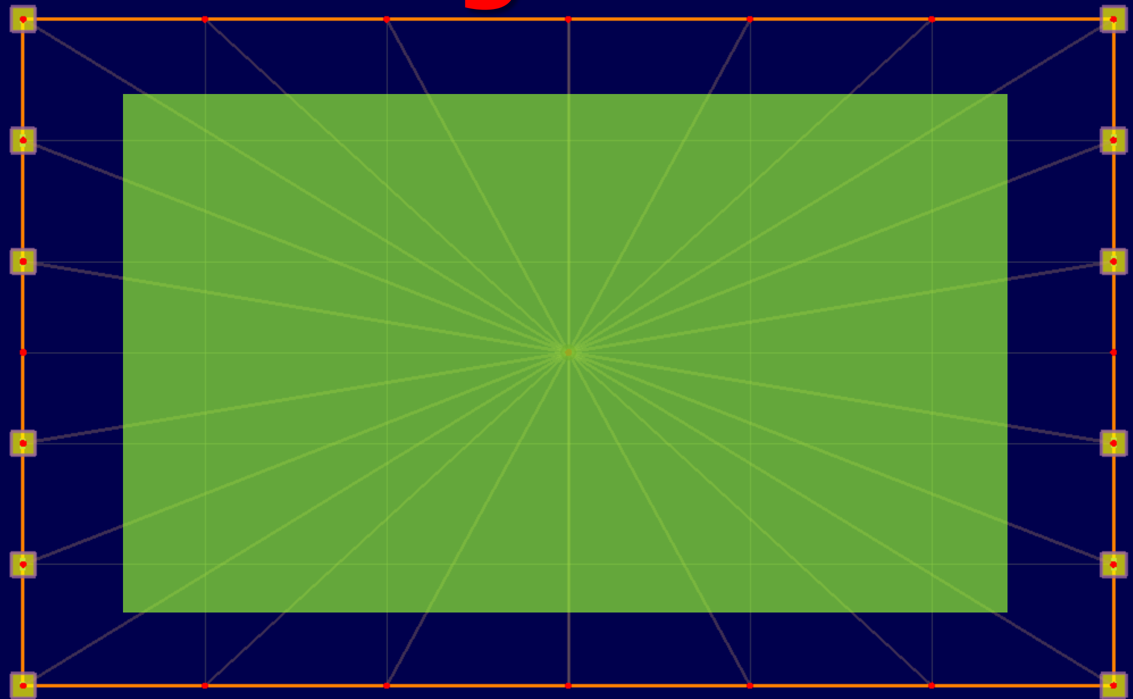
Gravity Loads

Use 1.2DL+0.5LL

Load from tributary area

Leaning column





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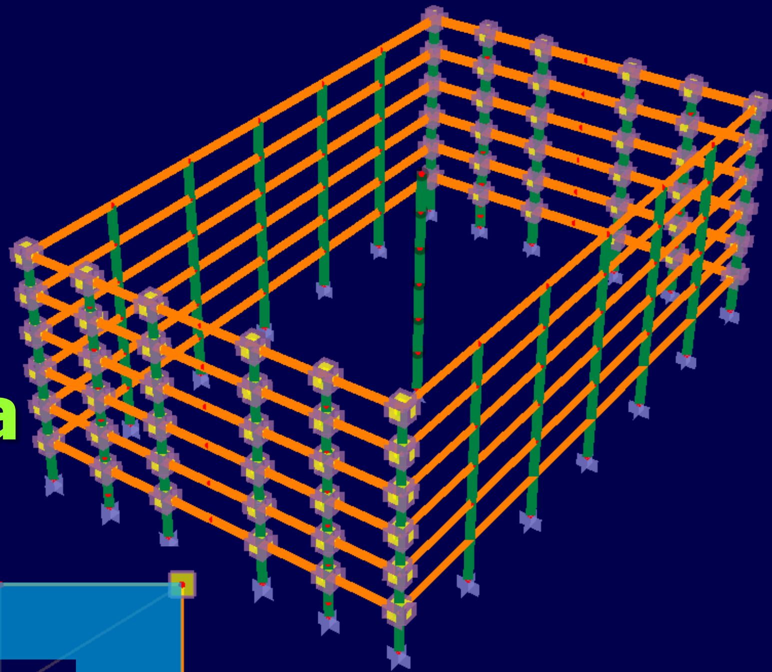
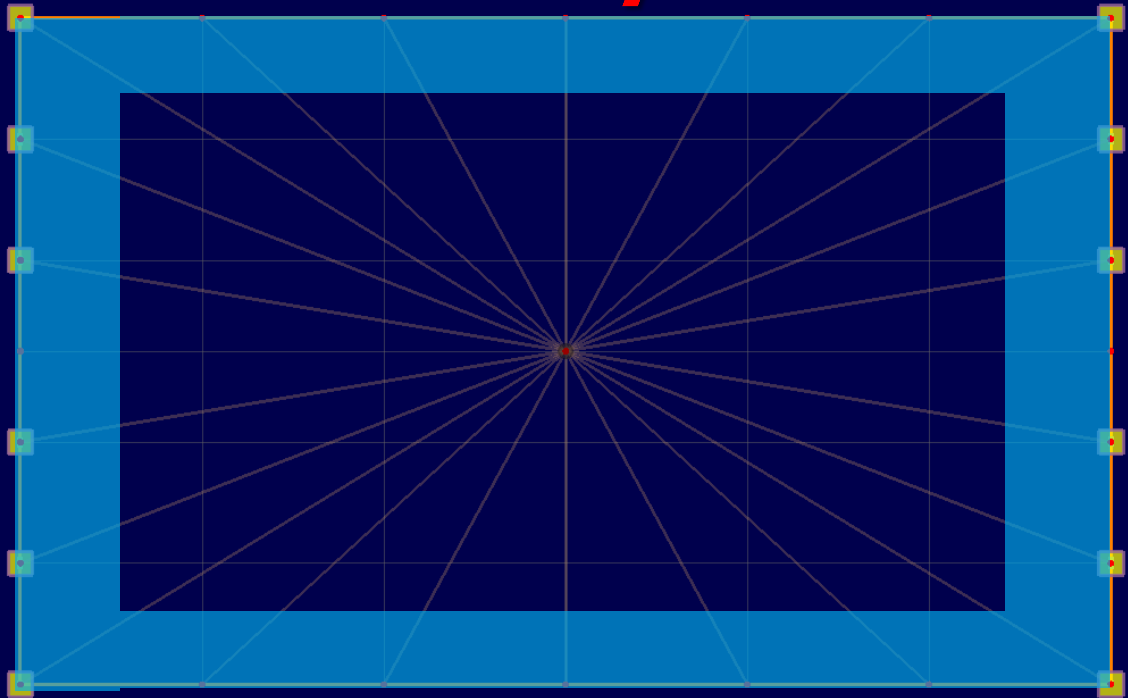
Gravity Loads

Use 1.2DL+0.5LL

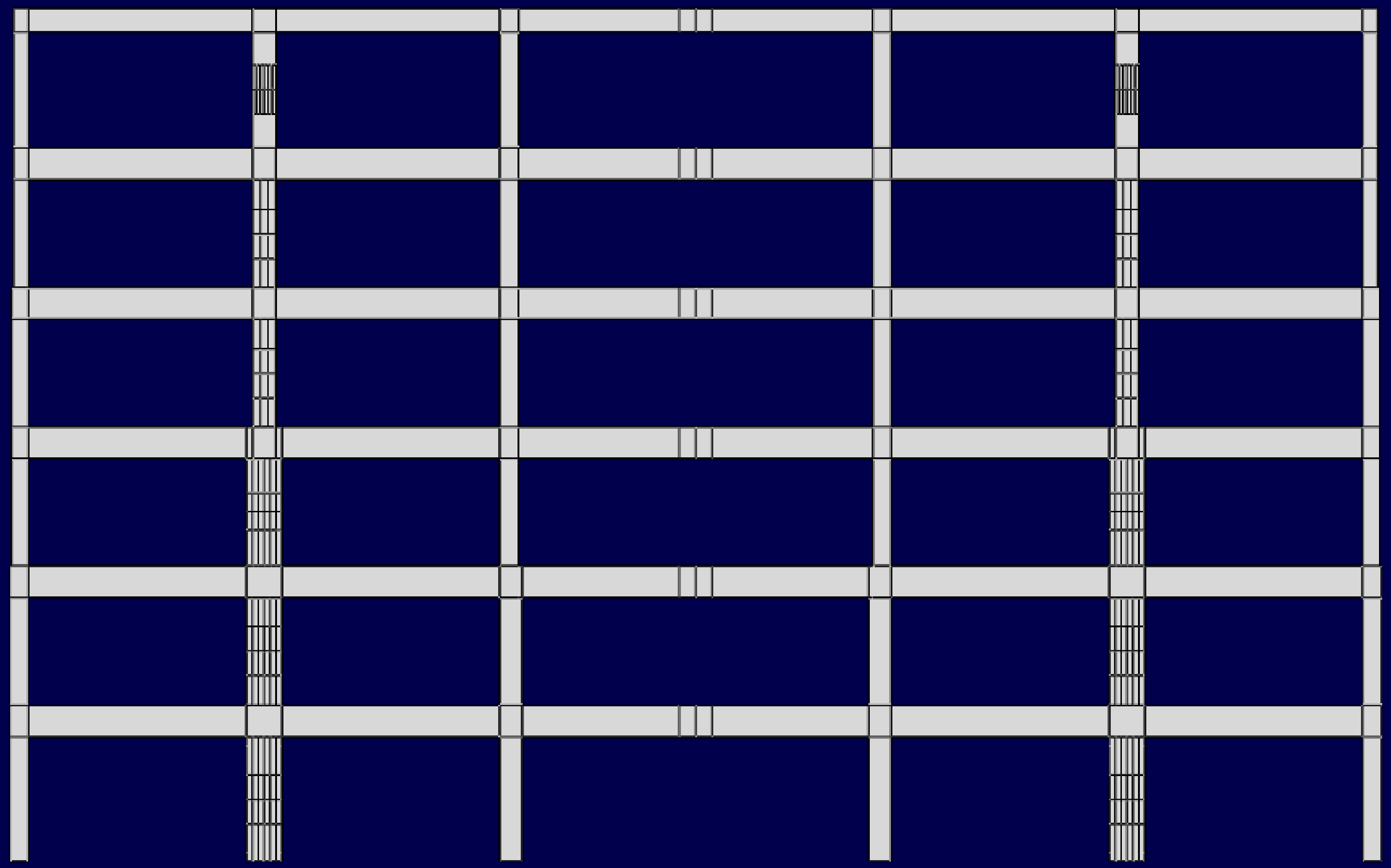
Load from tributary area

SPD-MRF System

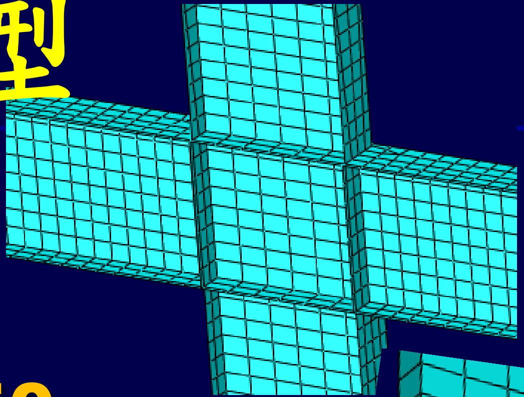




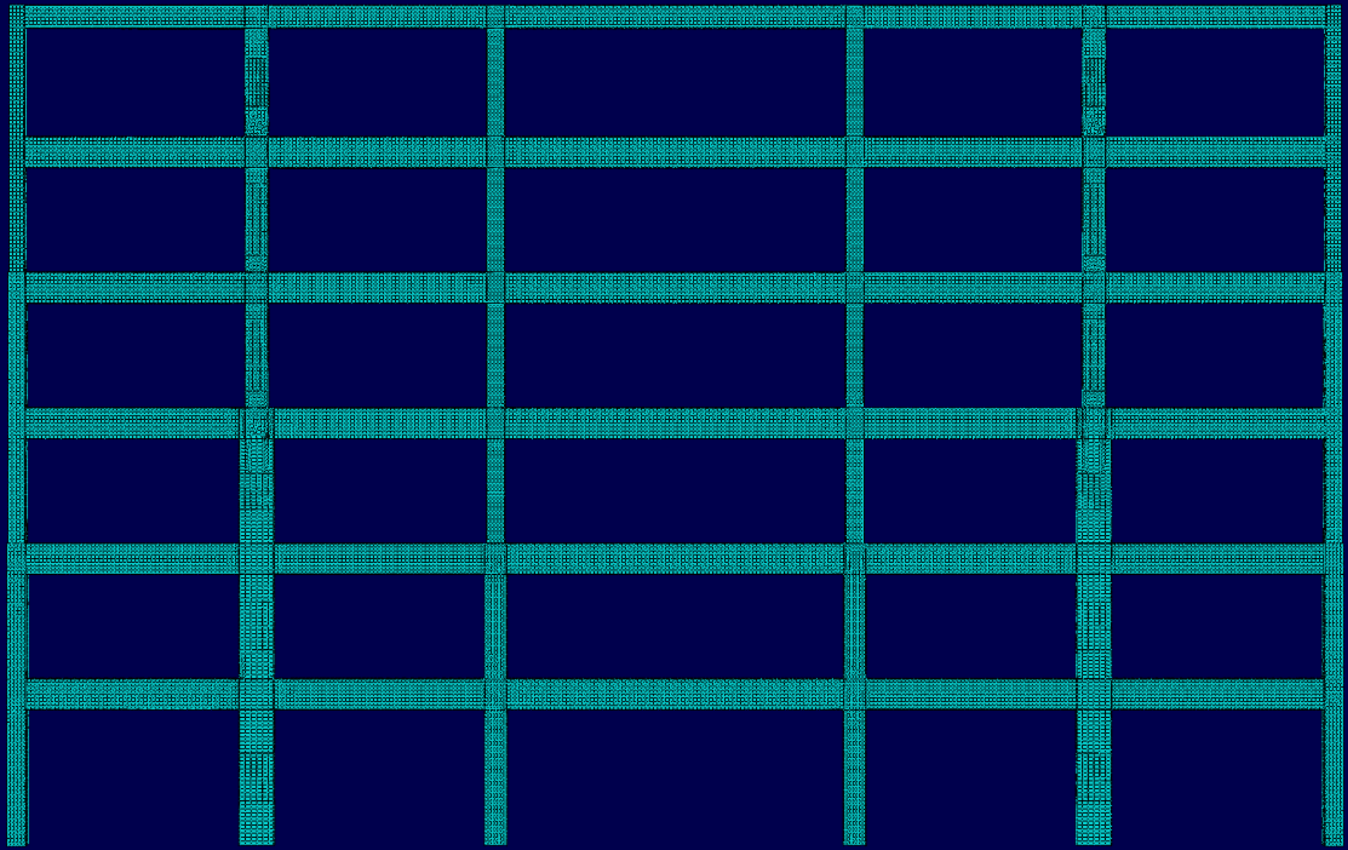
Abaqus分析模型

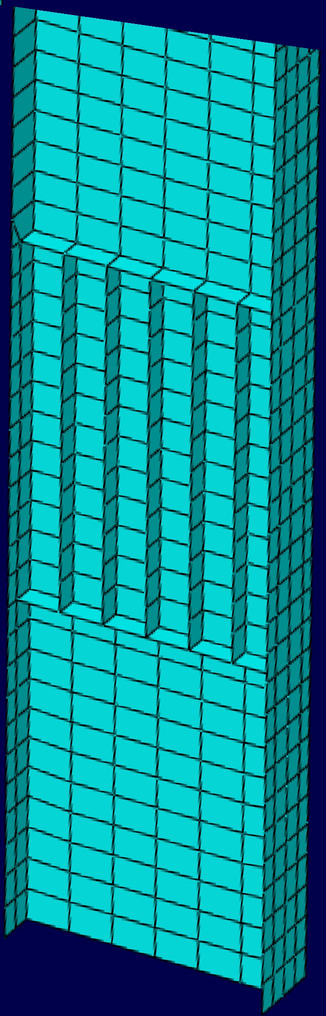


Abaqus分析模型



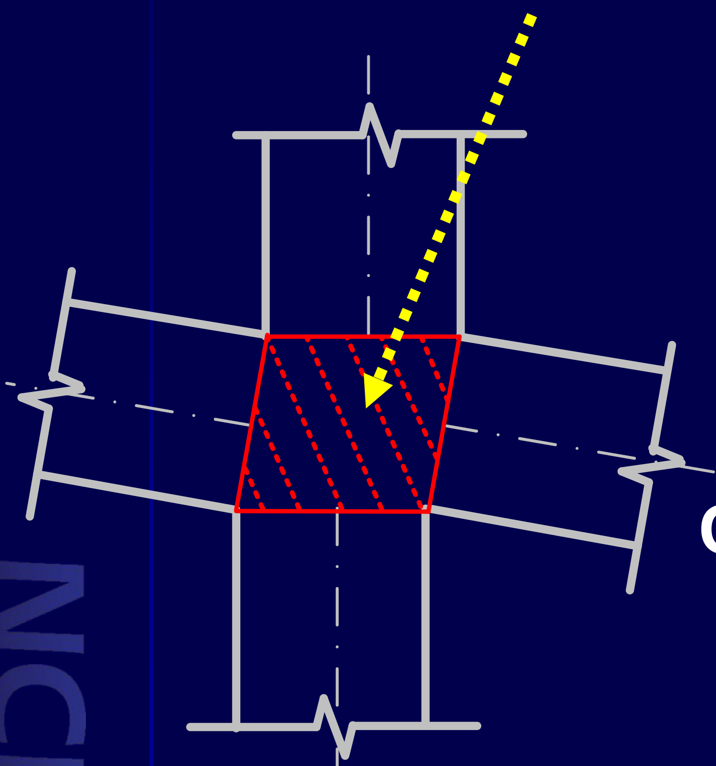
Shell Elements
Mesh Size: 100mm
Number of Elements: 58,760

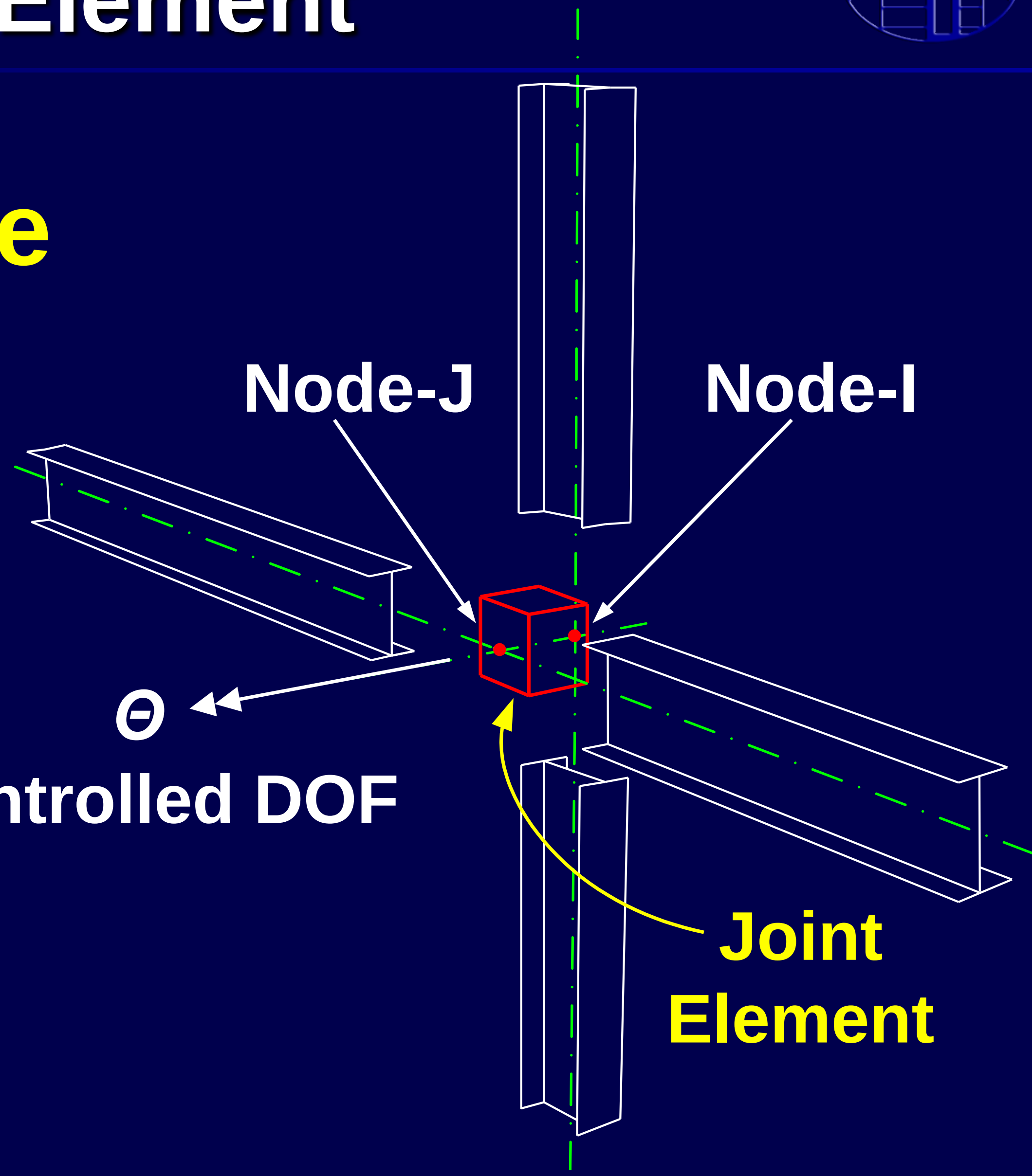




PISA3D Joint Element

- **Panel Zone**

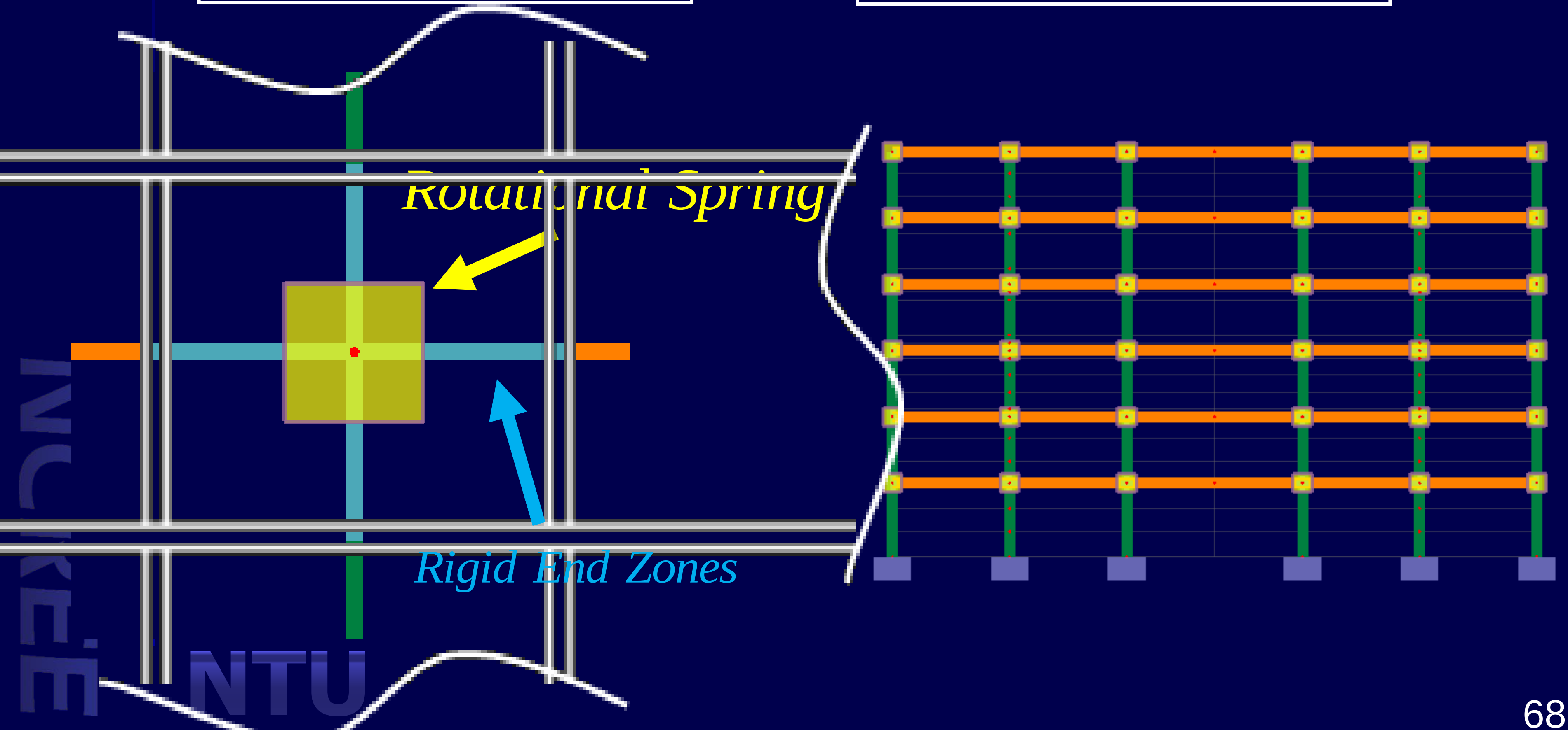




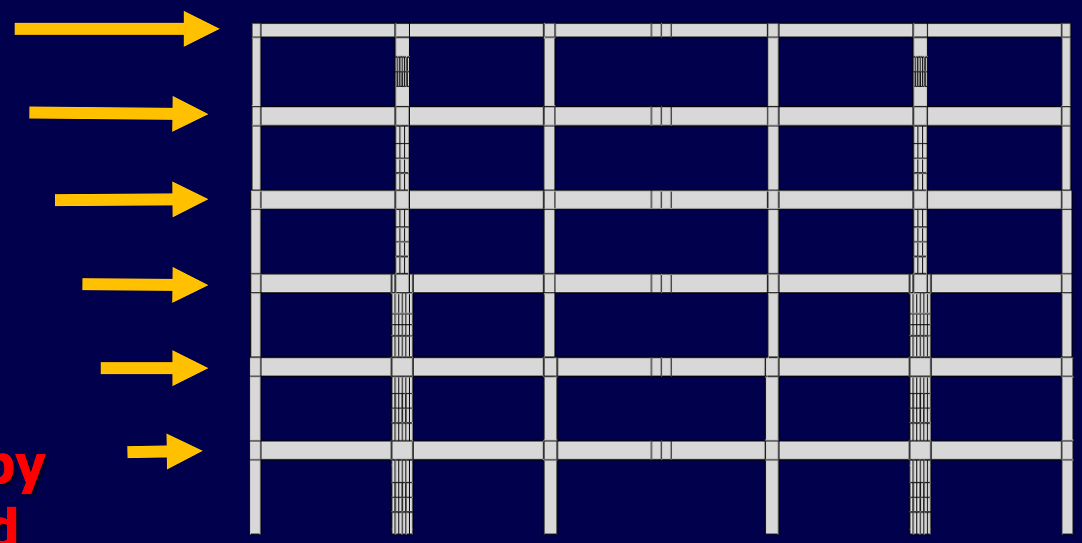
PISA3D梁-SPD交會區設定

$$K_e = \frac{G d_b d_{SPD} t_{pz}}{1 - \frac{d_{SPD}}{L_b}}$$

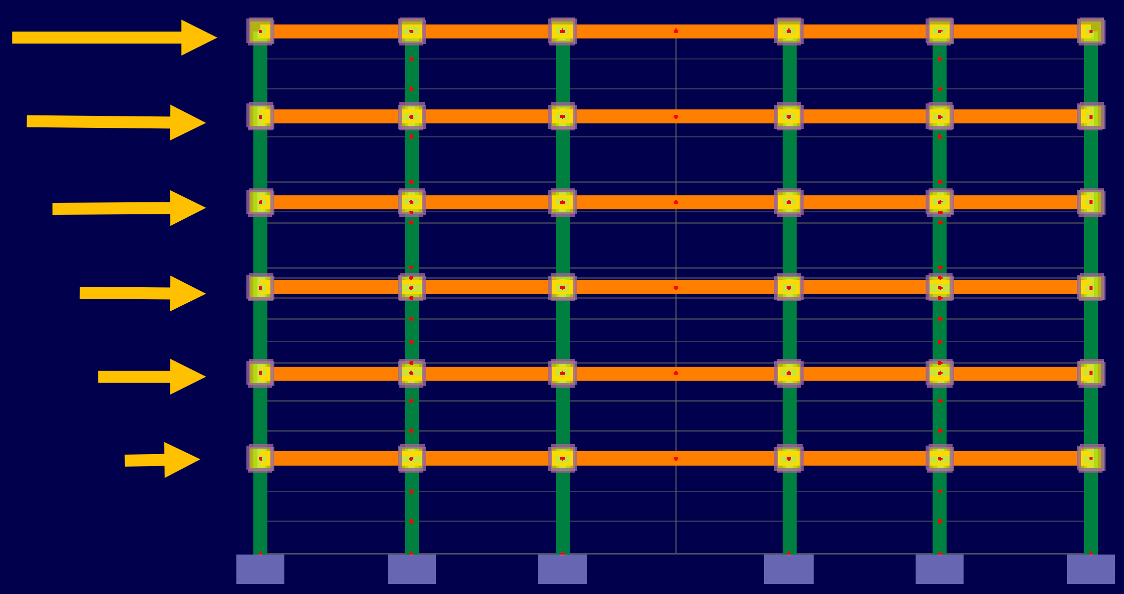
$$M_y = \frac{\tau_y d_b d_{SPD} t_{PZ}}{1 - \frac{d_{SPD}}{L_b}}$$



反覆載重分析加載方式



Design Force Patterns by static analysis method



PISA3D分析模型

BeamColumn Element

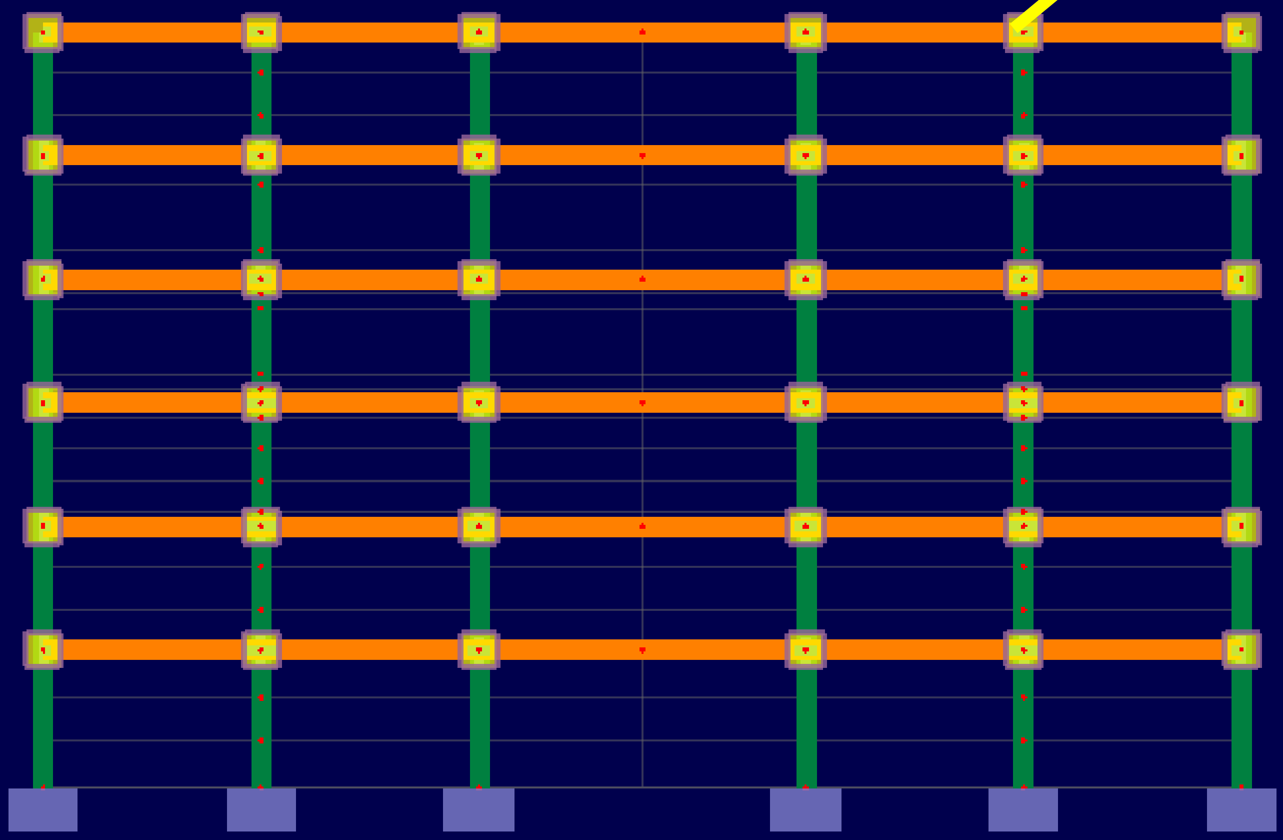
Center line to center line

3 elements for each SPD

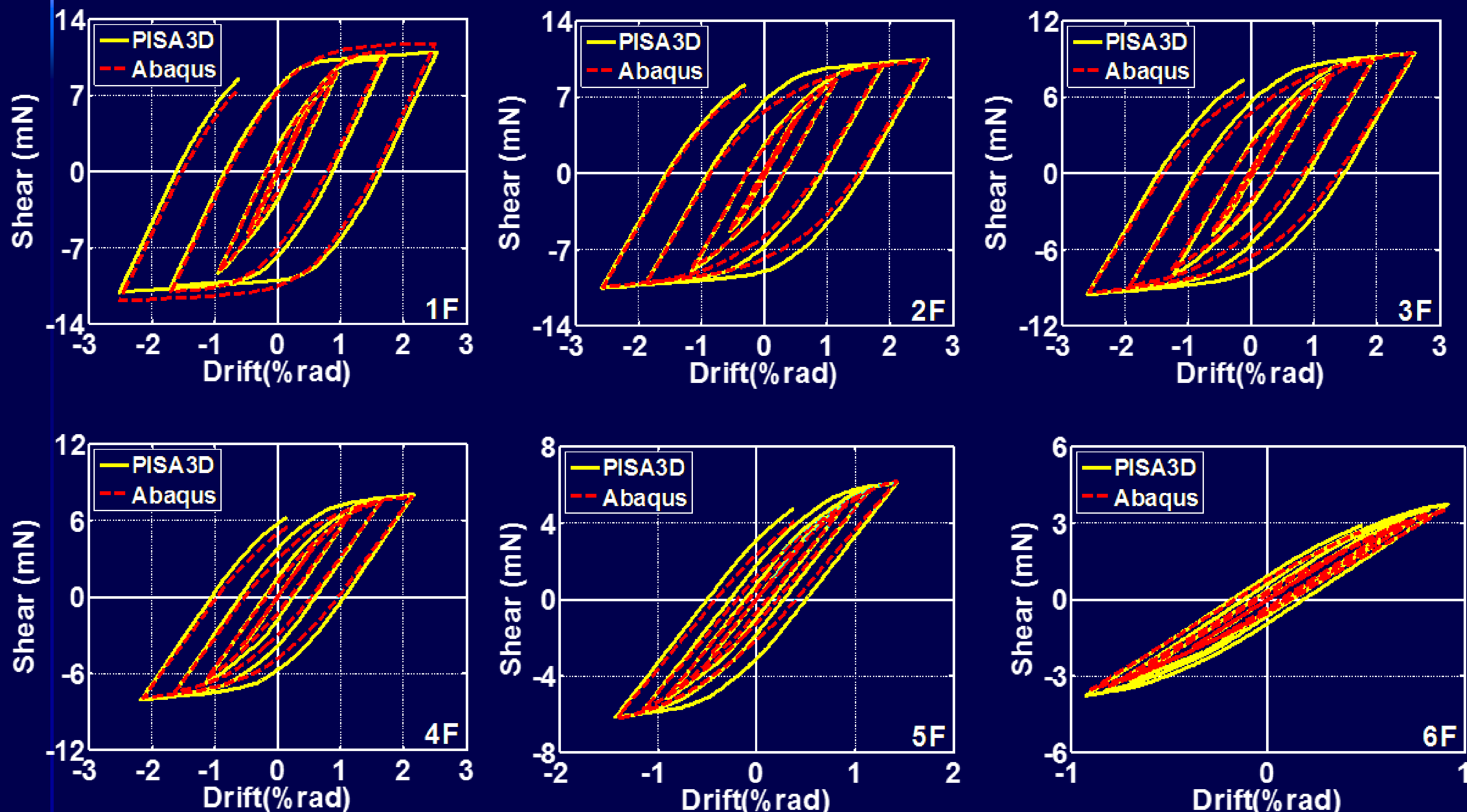
$$\xi = 2\%$$

Joint Element &

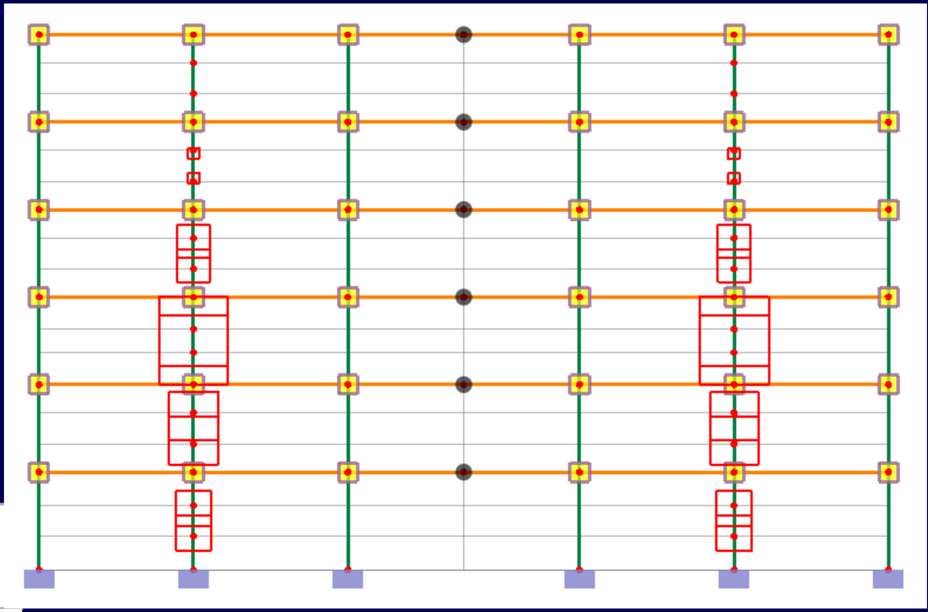
Rigid End Zones



樓層剪力 VS. 層間側位移

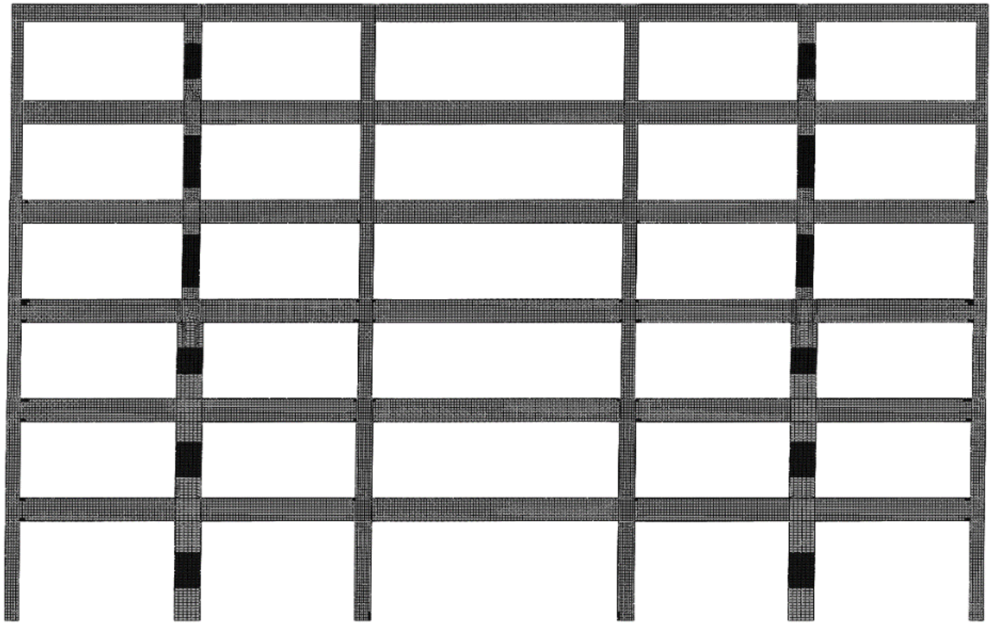


Roof Drift at 1%



Abaqus Model

Black: Yield Area

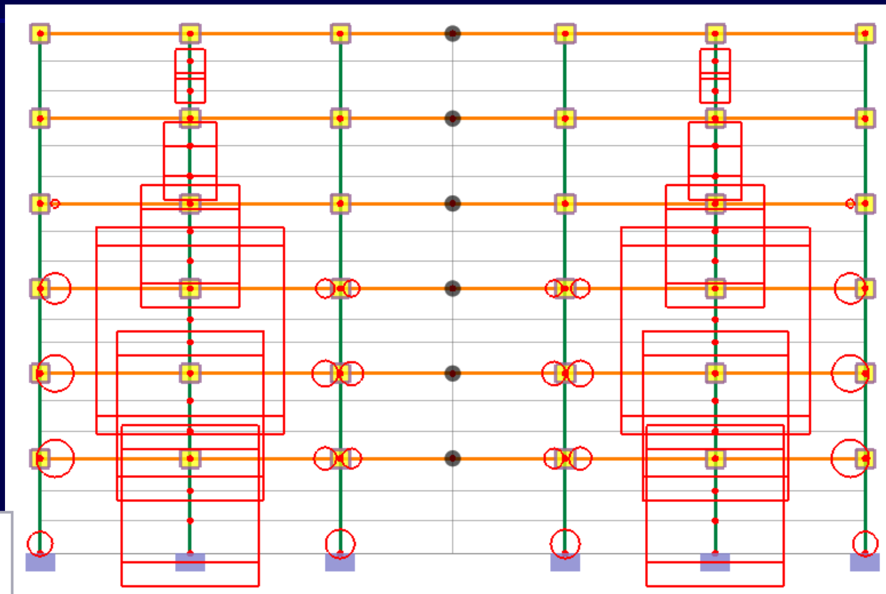


PISA3D Model

□ : Shear Hinge

○ : Moment Hinge

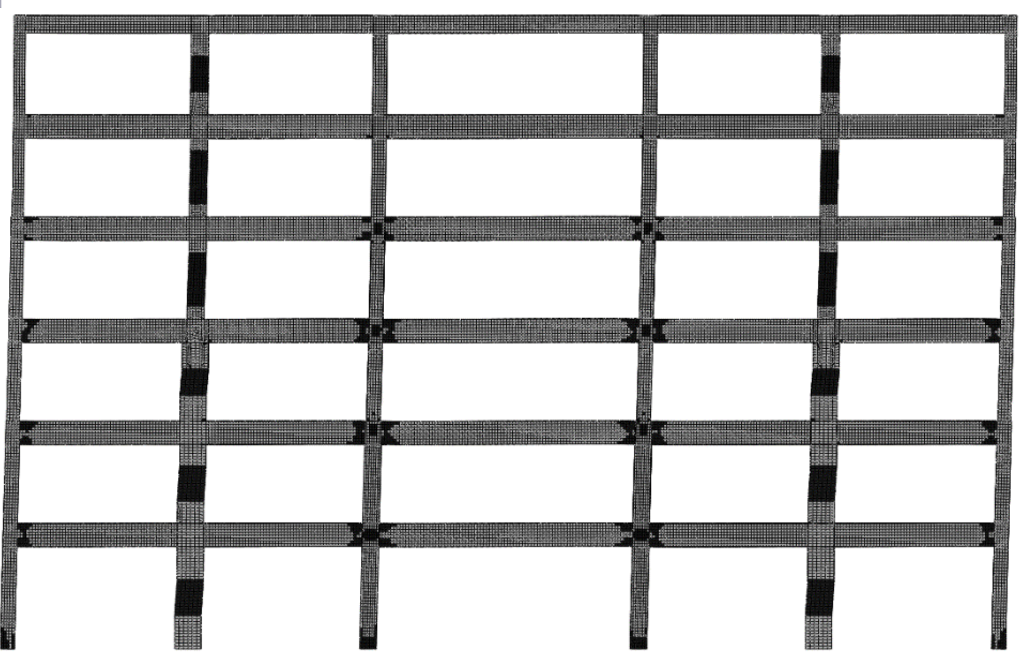
Roof Drift at 2%



PISA3D Model

Abaqus Model

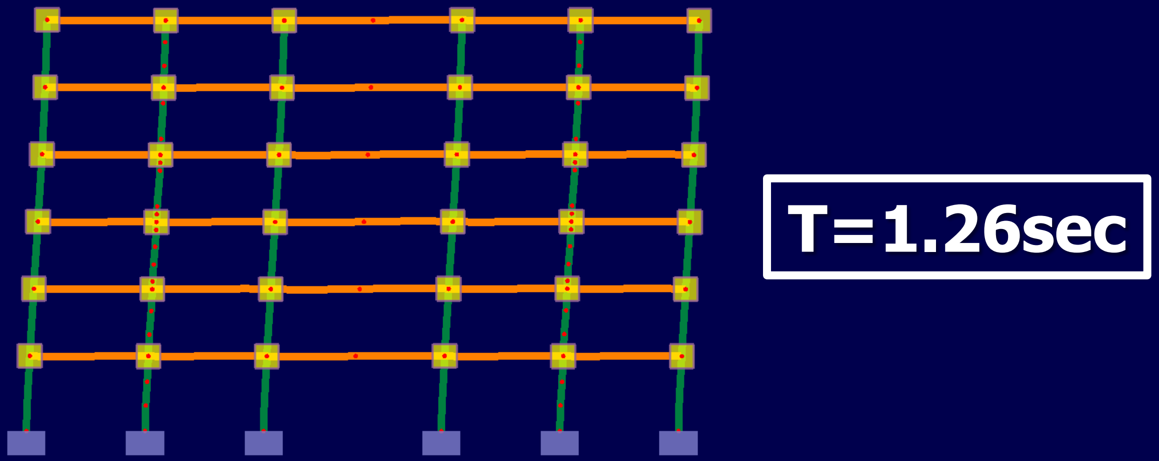
Black: Yield Area

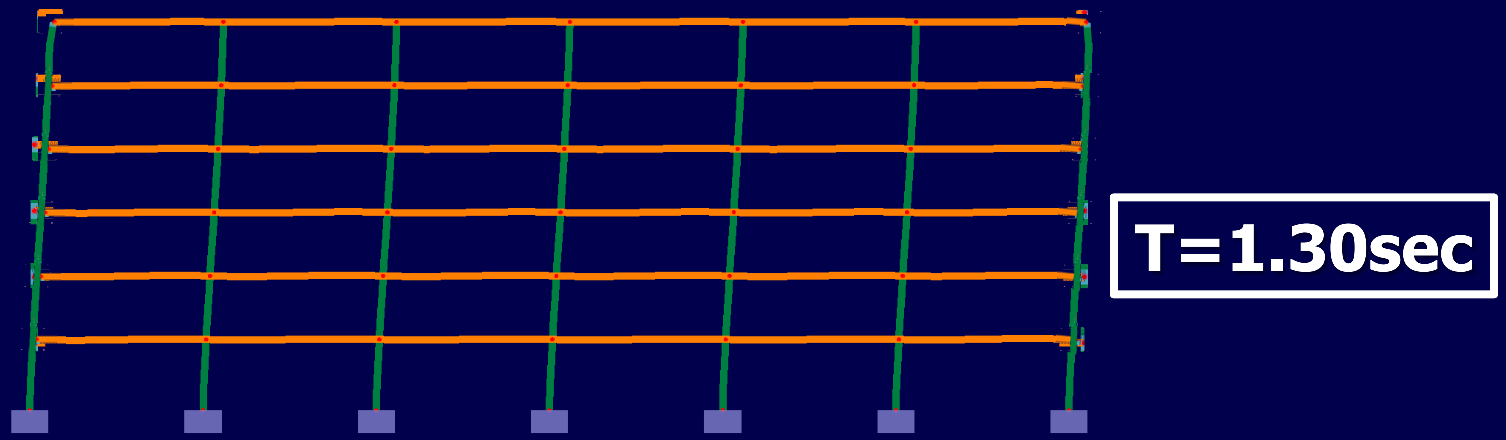


□ **: Shear Hinge**

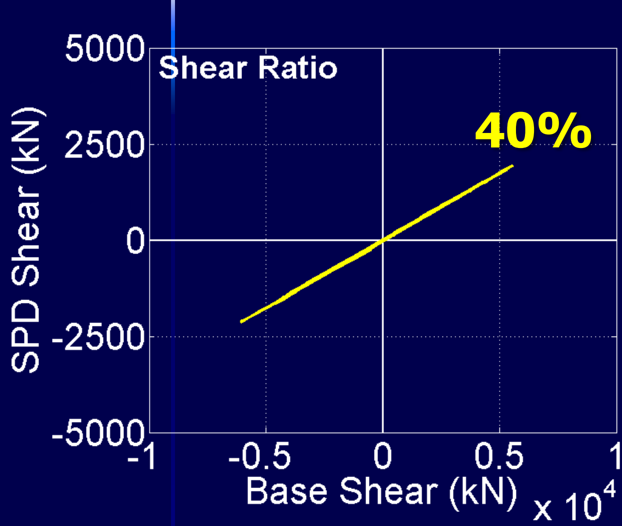
○ **: Moment Hinge**

Mode Shapes

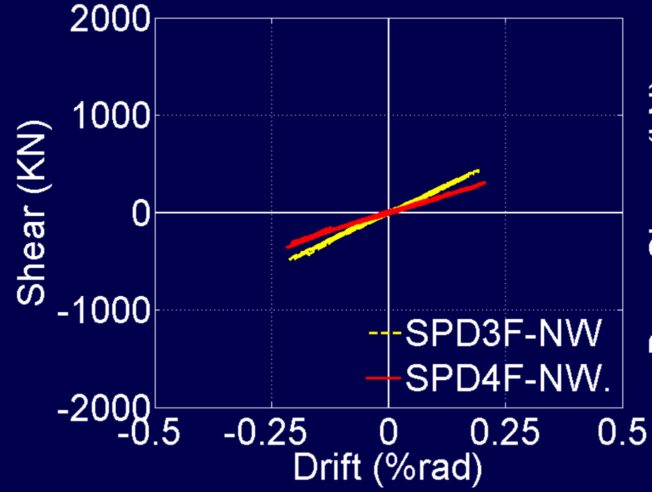




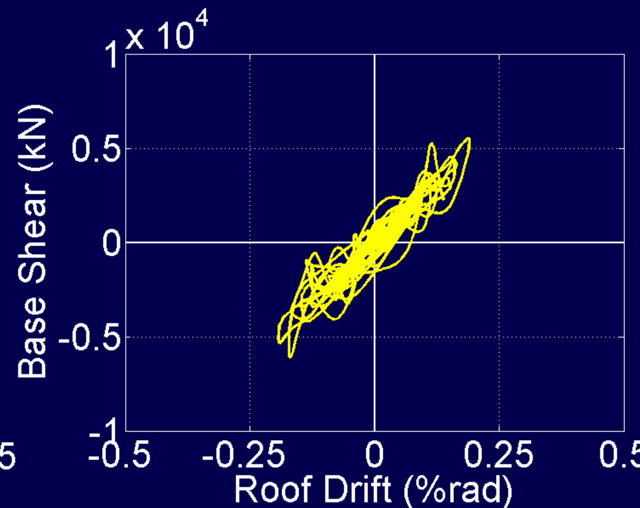
SLE → **DBE** → **MCE**



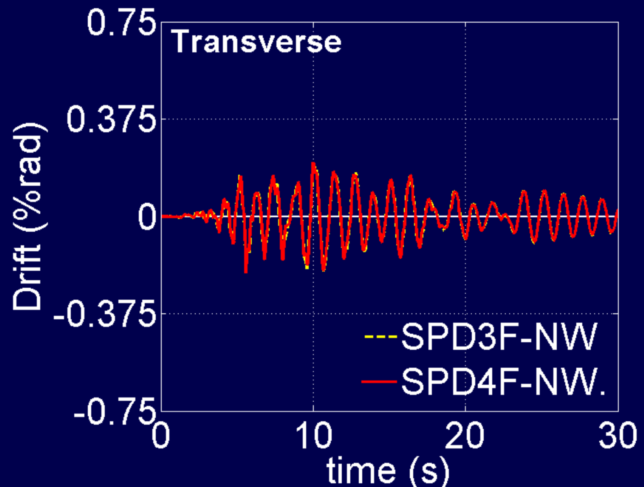
Shear Ratio
40%
SPD Shear (kN)
5000
2500
0
-2500
-5000
-1
-0.5
0
0.5
1
Base Shear (kN)
x 10^4



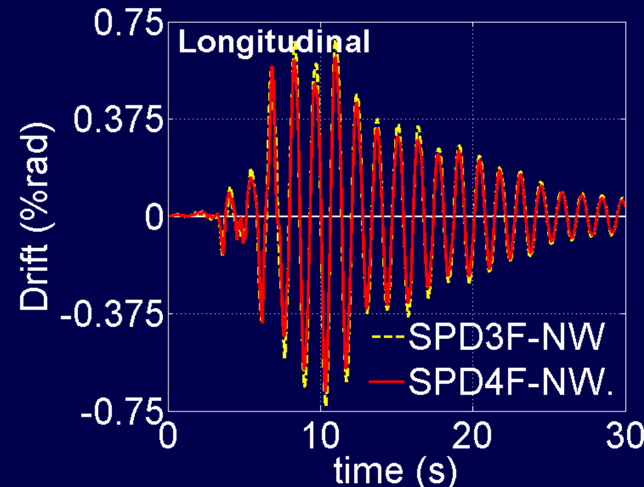
Shear (KN)
2000
1000
0
-1000
-2000
-0.5
-0.25
0
0.25
0.5
Drift (%rad)
SPD3F-NW
SPD4F-NW.



x 10^4
Base Shear (kN)
1
0.5
0
-0.5
-1
-0.5
-0.25
0
0.25
0.5
Roof Drift (%rad)

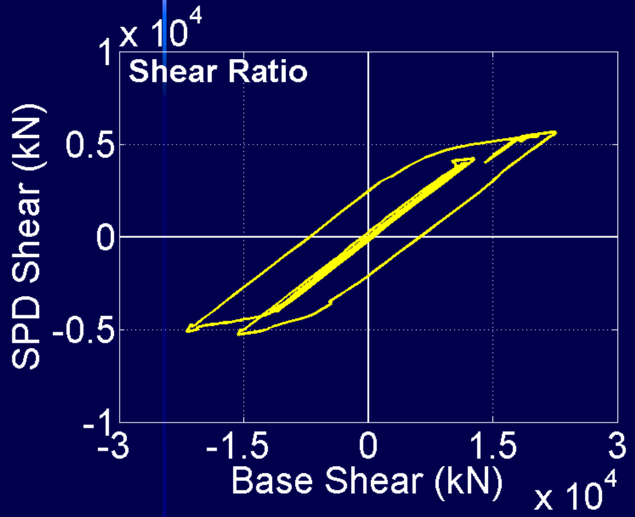


Transverse
Drift (%rad)
0.75
0.375
0
-0.375
-0.75
0
10
20
30
time (s)
SPD3F-NW
SPD4F-NW.

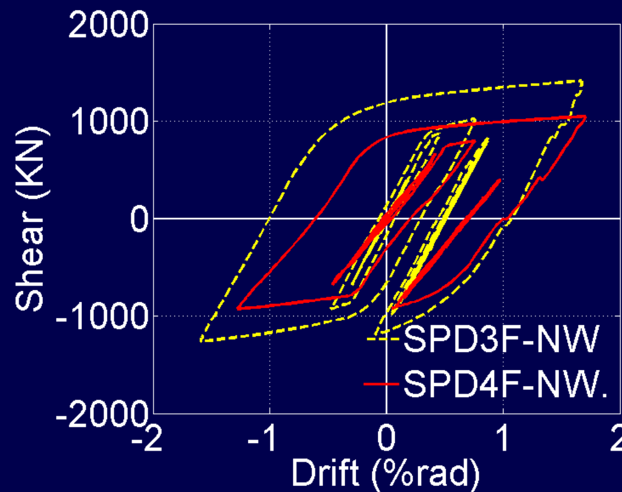


Longitudinal
Drift (%rad)
0.75
0.375
0
-0.375
-0.75
0
10
20
30
time (s)
SPD3F-NW
SPD4F-NW.

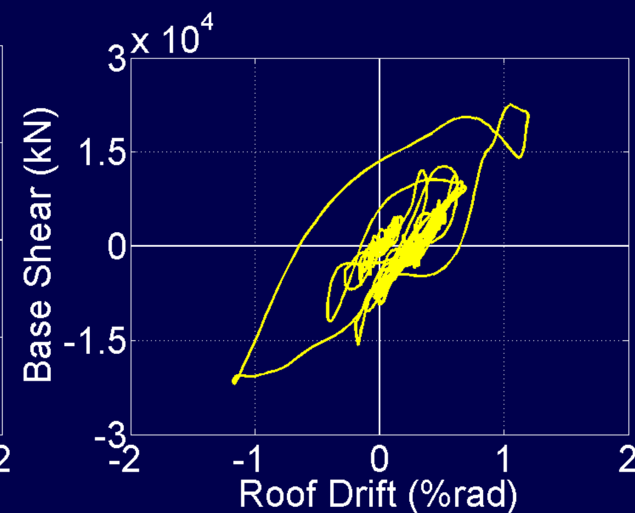
SLE → DBE → MCE



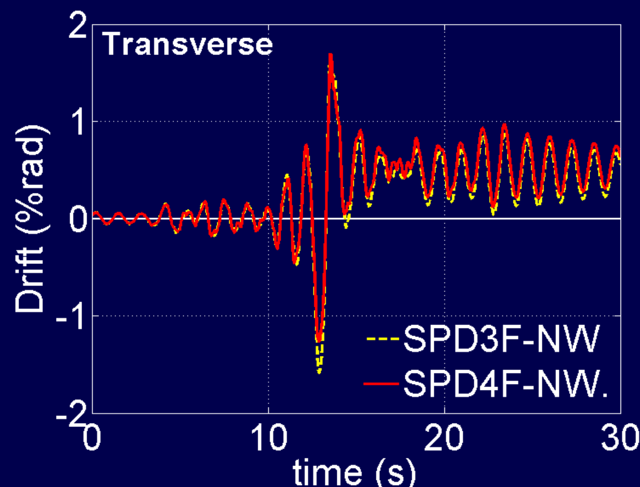
Shear Ratio
SPD Shear (kN)
Base Shear (kN)
x 10^4
x 10^4



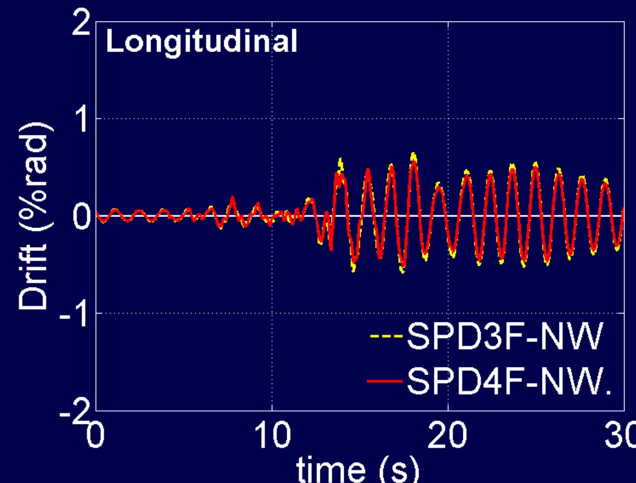
Shear (KN)
Drift (%rad)
SPD3F-NW
SPD4F-NW.



Base Shear (kN)
Roof Drift (%rad)
x 10^4

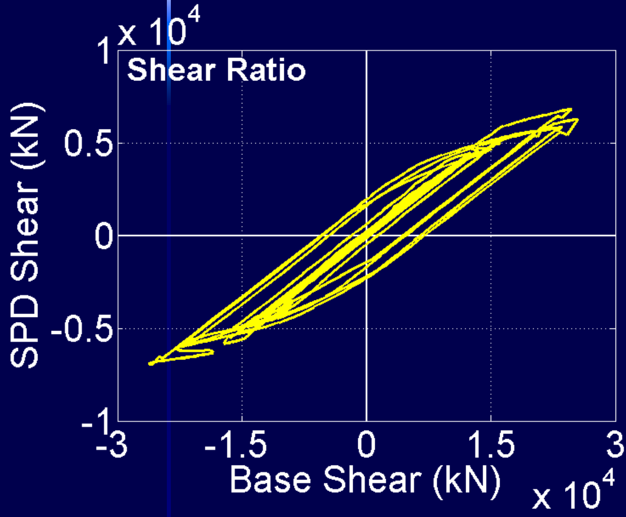


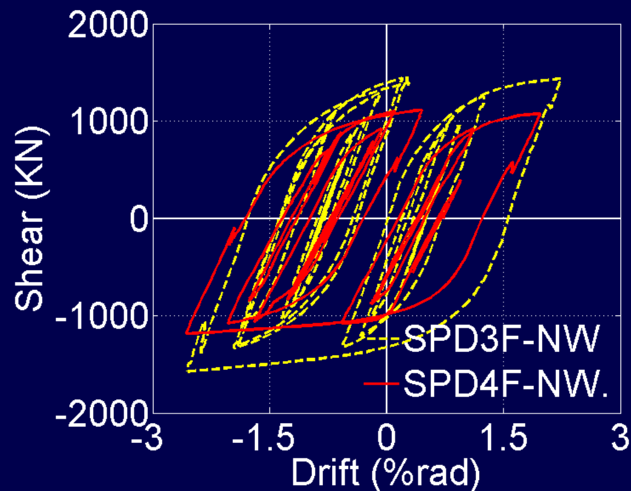
Transverse
Drift (%rad)
time (s)
SPD3F-NW
SPD4F-NW.

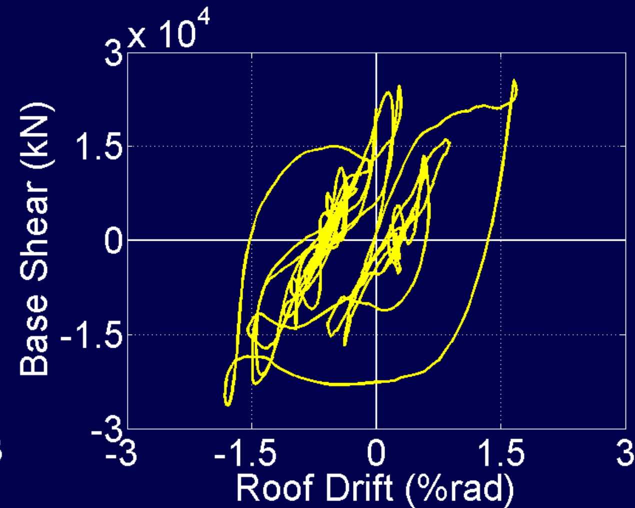


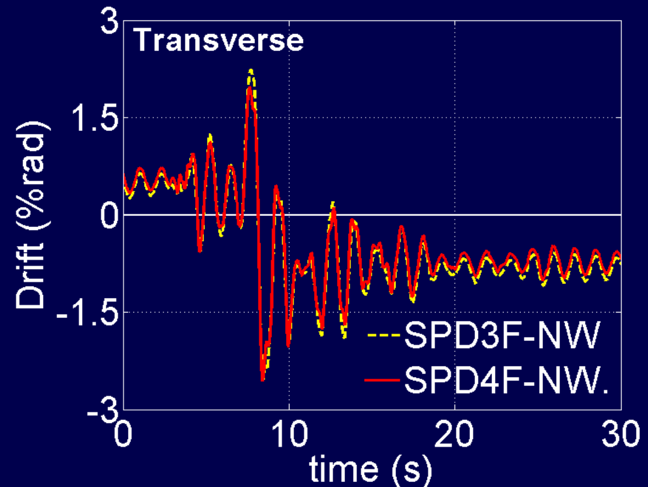
Longitudinal
Drift (%rad)
time (s)
SPD3F-NW
SPD4F-NW.

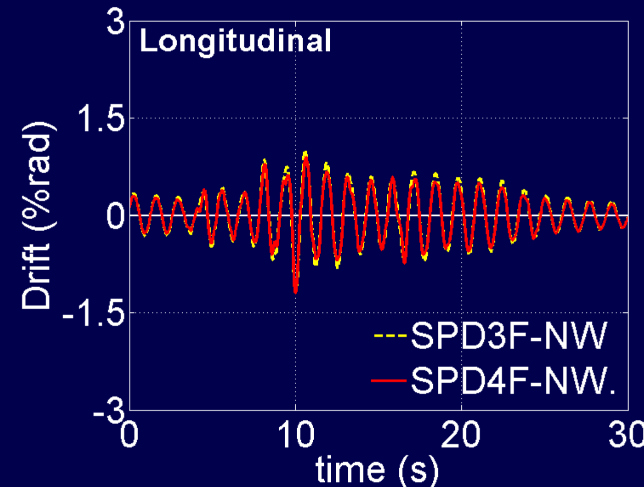
SLE → **DBE** → **MCE**











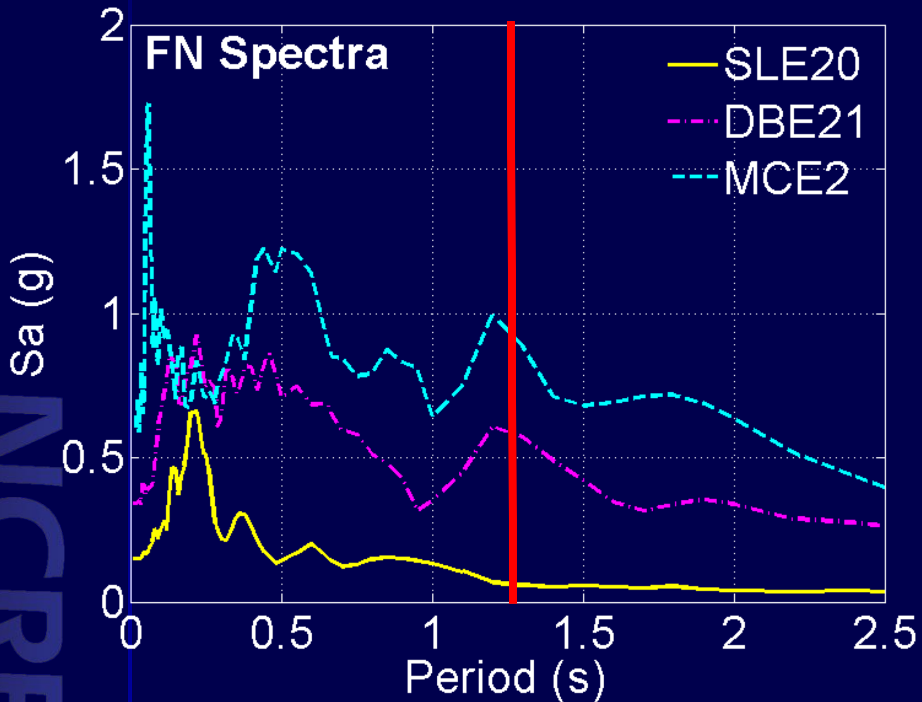
NCREE

NTU

Scaled Spectrum

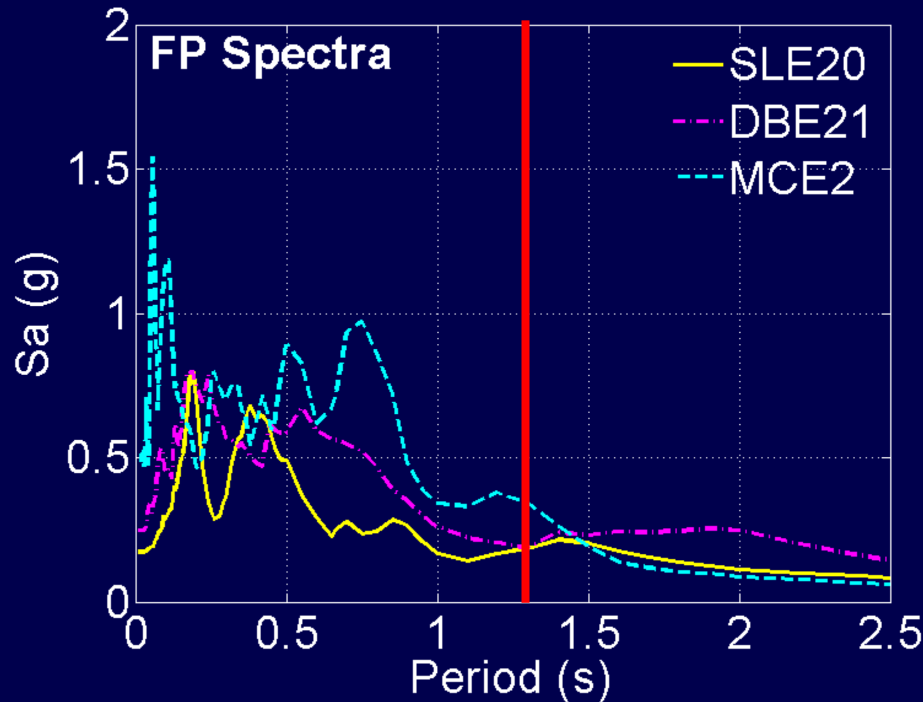
Transverse Direction

T=1.26s

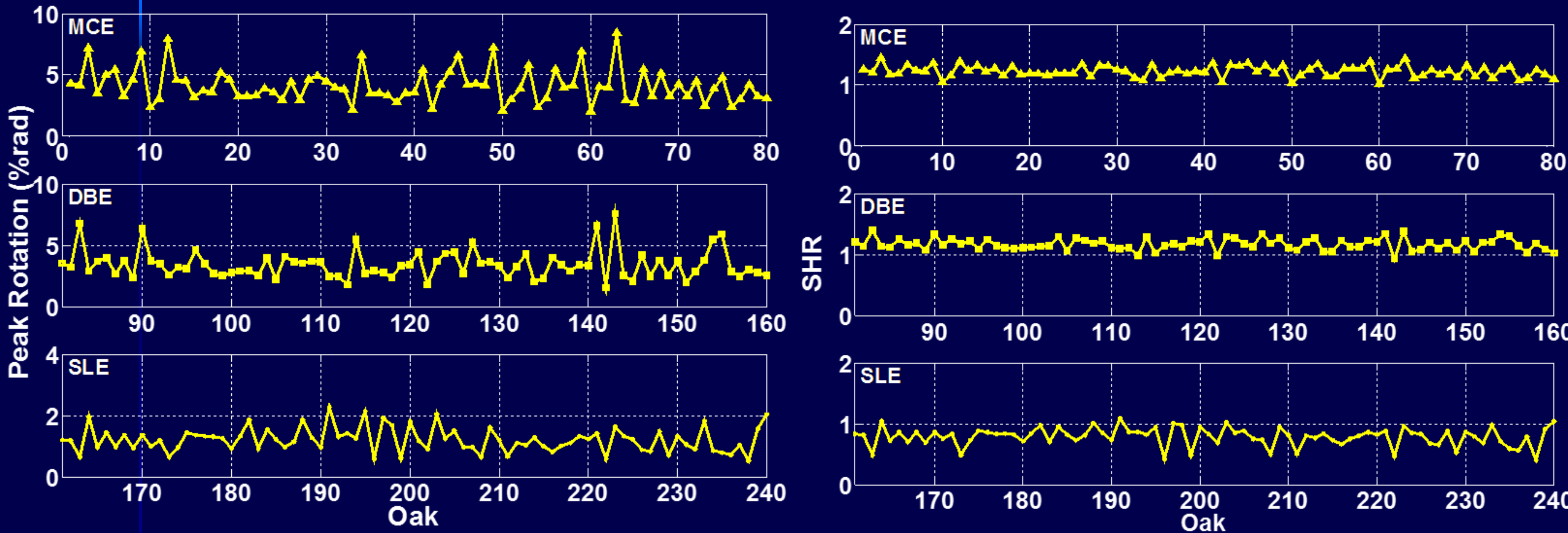


Longitudinal Direction

T=1.30s



Maximum Responses



SPD	240 EQs			Tests	
	Mean+SD			SPD-2L1T	SPD-2L0T
	MCE	DBE	SLE		
Core Rotation (%rad)	5.5	4.6	1.6	11.1	11.3
Strain Hardening	1.3	1.3	0.8	1.5	1.5

結論

- 塑鉸發生之位置與順序證實SPD-MRF之邊界梁容量設計法可行。
- PISA3D與Abaqus對6層樓SPD-MRF之模擬結果相近，代表PISA3D模型以旋轉彈簧與剛域模擬Panel Zone具準確性。
- 由240非線性動力分析結果與試驗比較，證實SPD具有足夠的耐震容量。