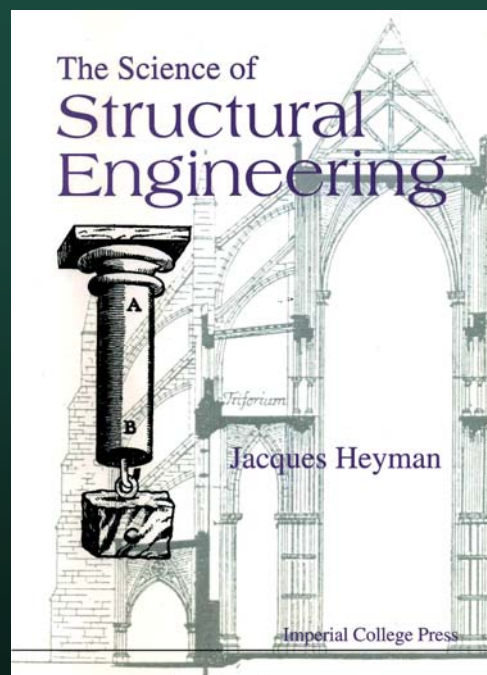


結構工学的科学
--塑性変形与鋼結構建築之耐震設計--

和田 章

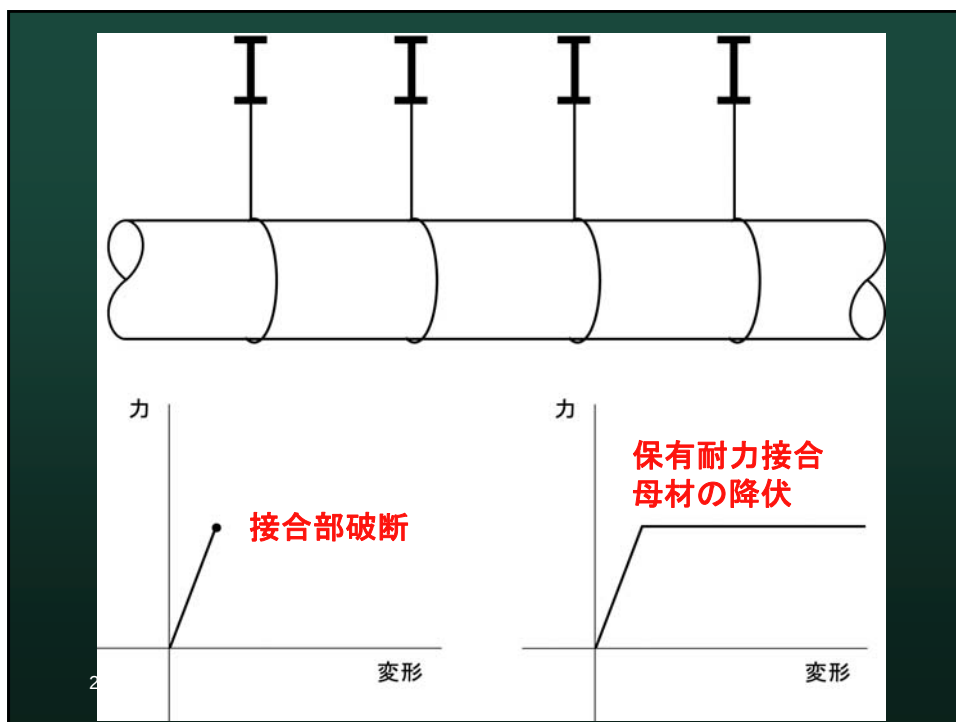
東京工業大学教授



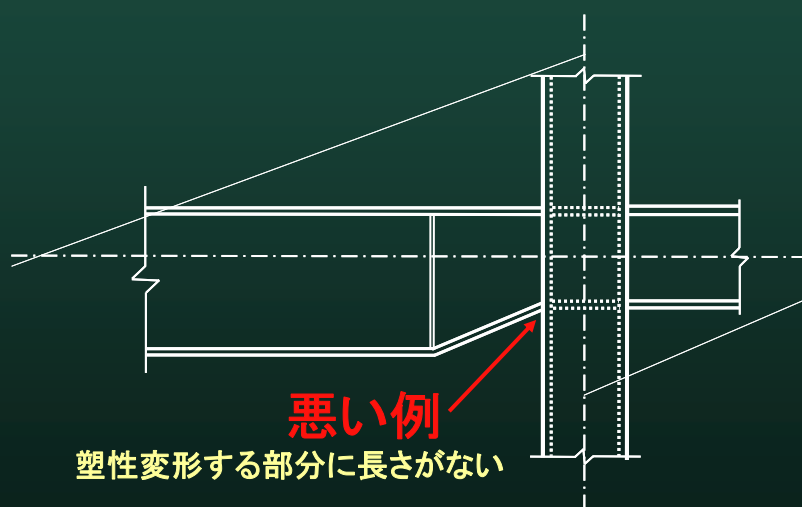


不安定	静定	不静定
(a)	(b)	(c)

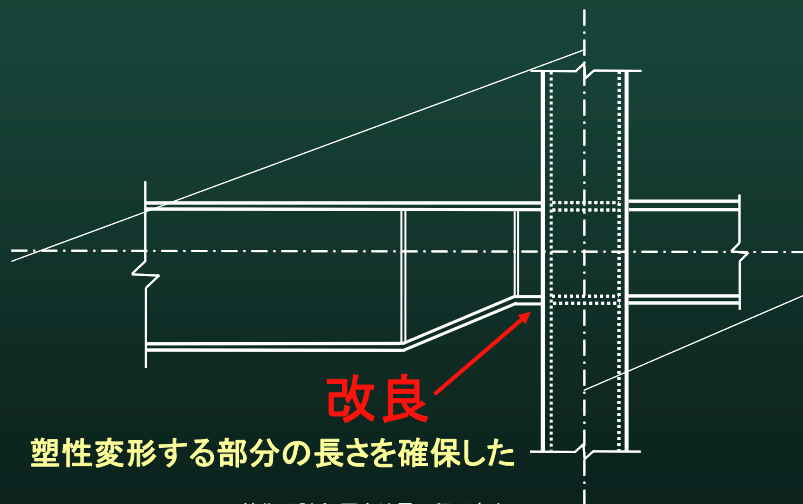
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4



部材形状と塑性変形能力



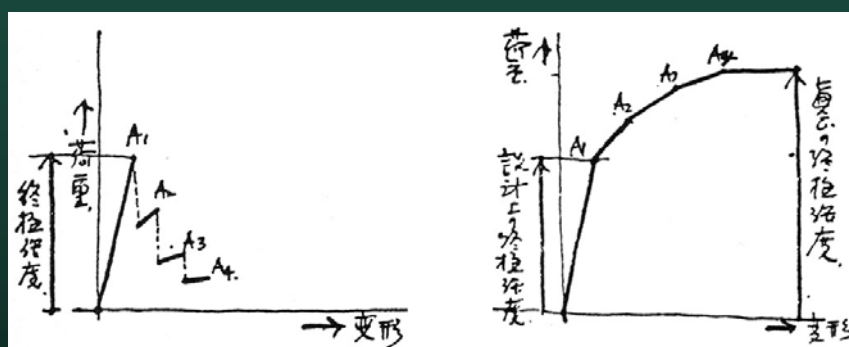
部材形状と塑性変形能力



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(a) 脆き材料による場合

(b) 靱き材料による場合

棚橋 諒「材料の靱性による構造物の耐震的終極強度の高められに関して」1934年 より

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構造設計と塑性変形能力

- 構造材料・部材の適度な塑性変形能力
- 構造物の崩壊荷重を説明する3定理
 - 下界の定理
 - 上界の定理
 - 唯一性の定理

下界の定理

- 「もし、ある外荷重と釣合い、構造物中のどの部分でも降伏条件を満足している応力分布が求められれば、その荷重は真の崩壊時の荷重に等しいか小さい」

上界の定理

- 「もし、ある想定した崩壊メカニズムにおいて、ある荷重によりなされた外部仕事
が、塑性ヒンジでなされる内部仕事に等
しい場合、その荷重は真の崩壊時の荷
重に等しいか大きい」

解の唯一性定理

- 「もし、ある荷重に釣合い、メカニズム形
成および降伏条件の3つの条件を満足
する応力分布を求めることができれば、
その荷重は真の崩壊時の荷重である」

構造物内部の応力状態は 解析とは異なる (1/2)

- ・ 建築構造物の建設過程
(型枠、支保工)
- ・ 建築構造物は無重力の宇宙で作って地球へ
運ぶのではない
- ・ 逆打ち工法、鋼心柱、地下外壁

構造物内部の応力状態は 解析とは異なる (2/2)

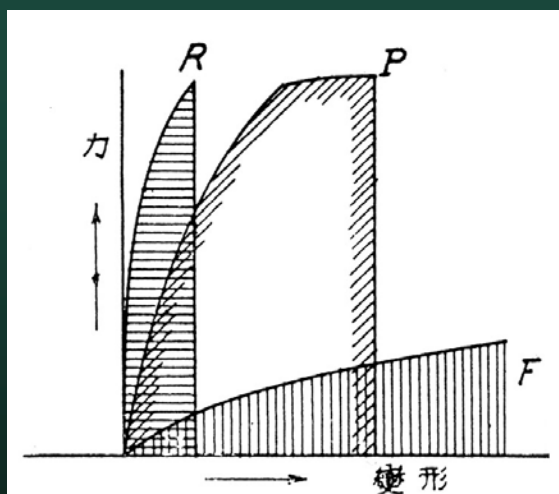
- ・ 鋼構造の形鋼の残留応力
- ・ 溶接構造の残留応力
- ・ 施工順序による影響
- ・ 柱の伸縮の影響

上界の定理

- 真の崩壊荷重は、ありうるすべての崩壊形を仮定した計算を行い、得られた値の最小値である。
- 構造設計はいかに色々な場合を考え抜くかということである。考え抜くことこそ、常に忘れてはならない。

変形能力の2つの役割

- 不静定構造を成立たせるために
必要な変形能力
 - ・各個撃破を防止、
 - ・部材耐力の総和が抵抗力、
 - ・下界の定理を成立させるため
- 地震時のエネルギー吸収を
高めるための変形能力



棚橋 諒「材料の靱性による構造物の耐震的終極強度の高められに関して」1934年 より

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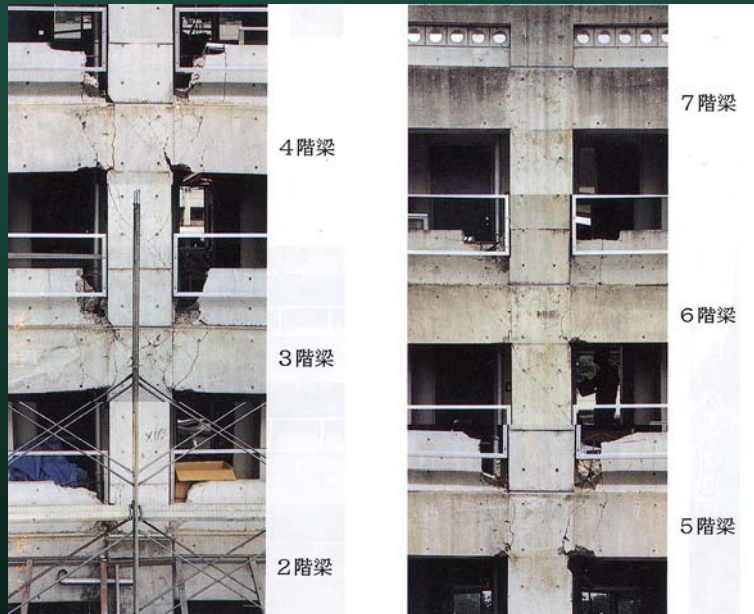


神戸地震の被害
鉄筋コンクリート構造(7階建)

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社新井組 1995年技術研究報20
裏より



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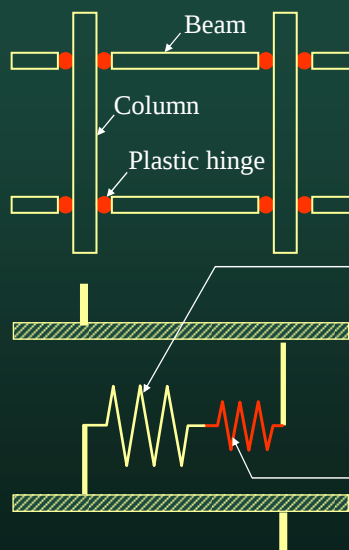
26

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梁降伏形ラーメンの仕組み



Total deformation
of the structure :

$$\Delta = \Delta_e + \Delta_p$$

Δ_e = Elastic deformation of
beams and columns. This
could be over 1/200 inter
story deformation angle.
Elastic deformation is too
large itself.

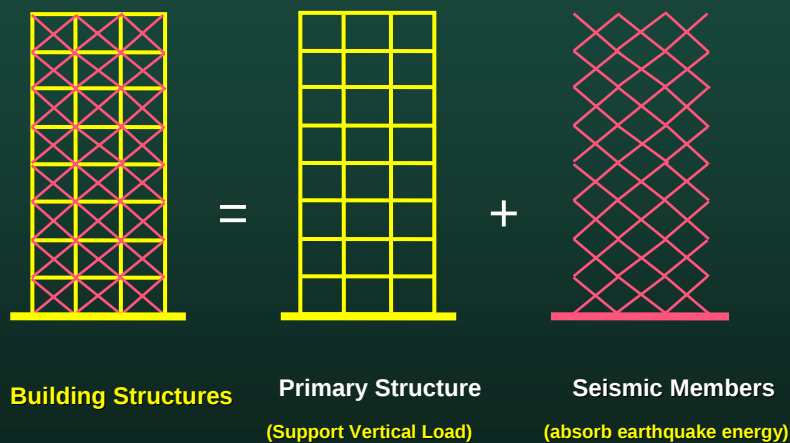
Δ_p = Plastic deformation of
caused by yield hinges

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損傷制御構造の仕組

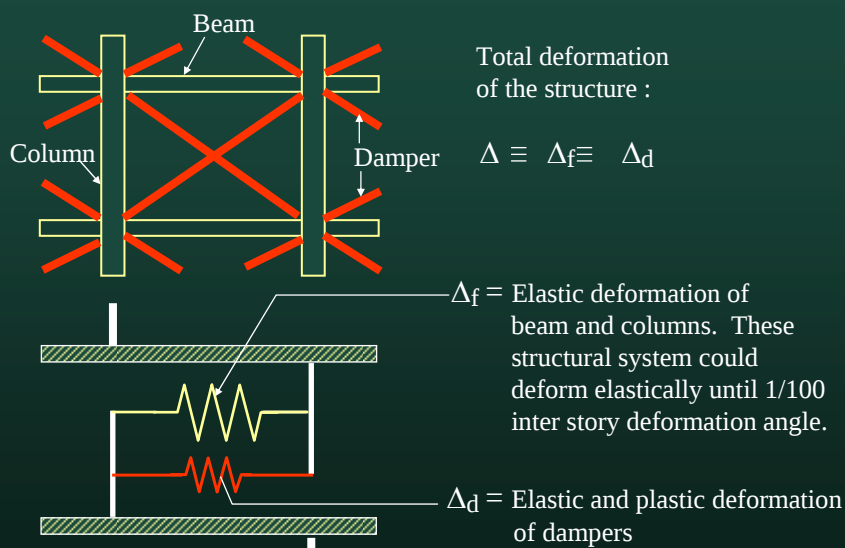


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損傷制御骨組の仕組み

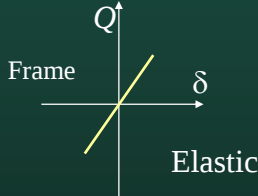
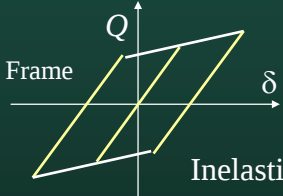
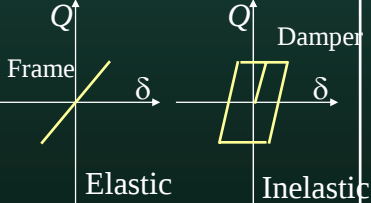
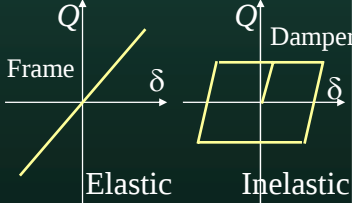


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Old concept and New concept

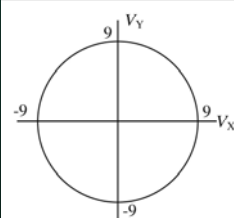
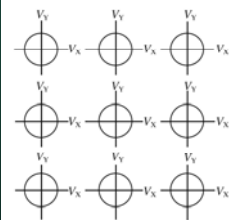
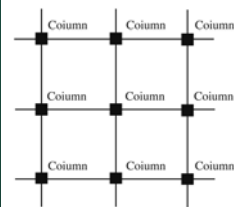
	Small/medium earthquake	Large earthquake
Old		
New		

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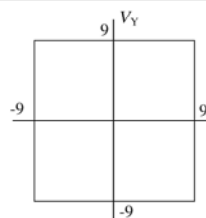
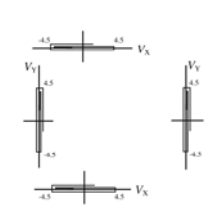
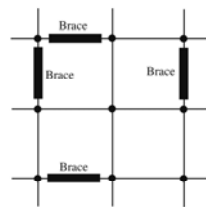
31

Weak Columns Structure



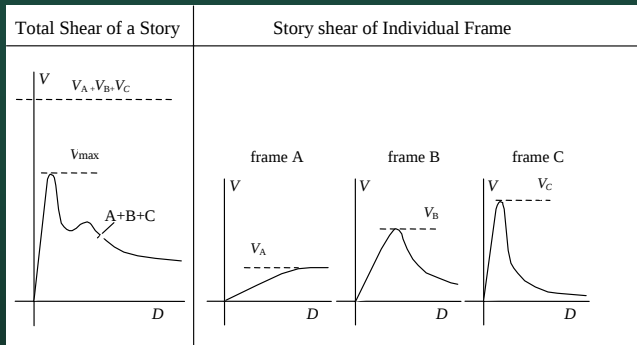
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Braced Structure

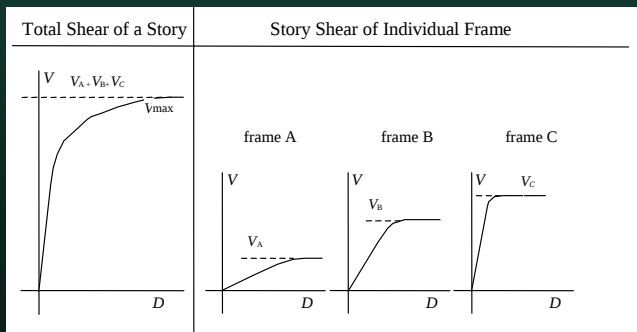


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Brittle



Ductile



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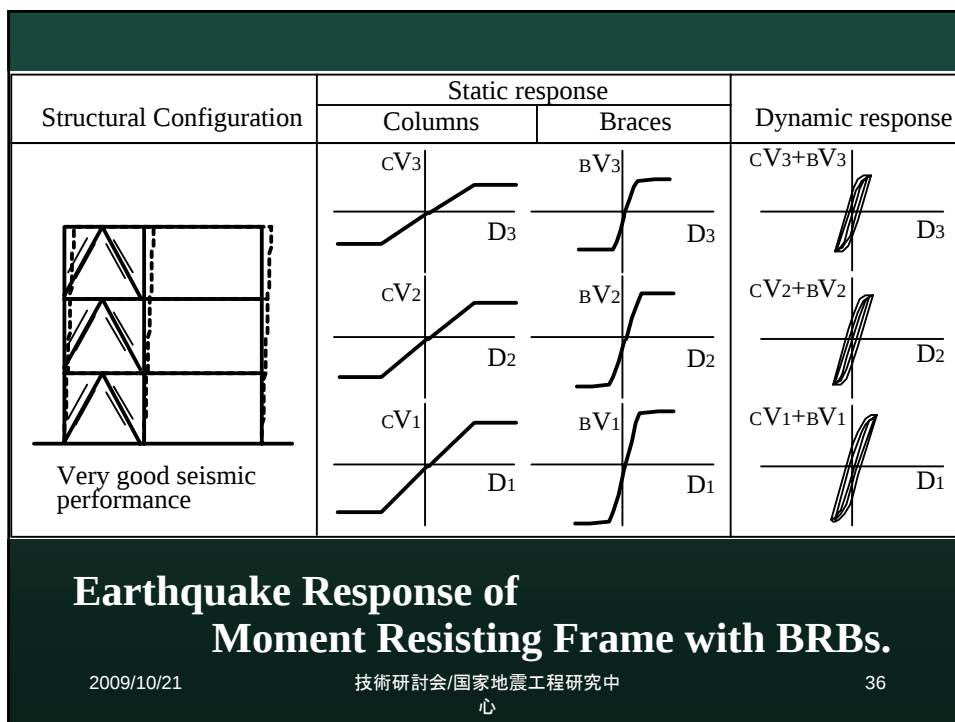
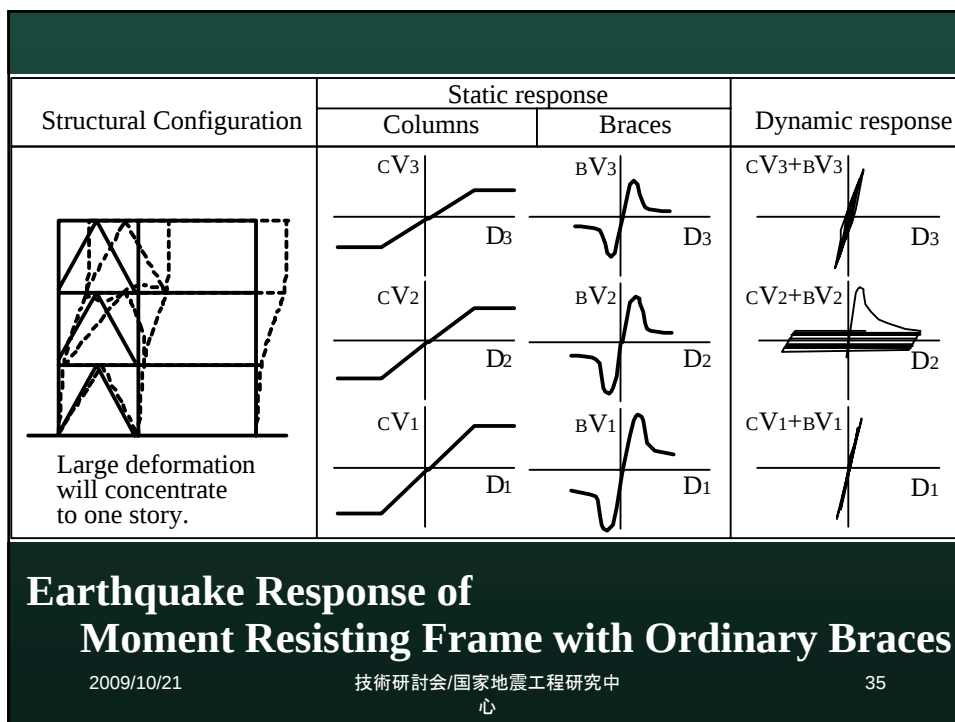
Structural Configuration	Static response		Dynamic response
	Columns		
Too large deformation will occur.			
		No Brace	

Earthquake Response of Moment Resisting Frame

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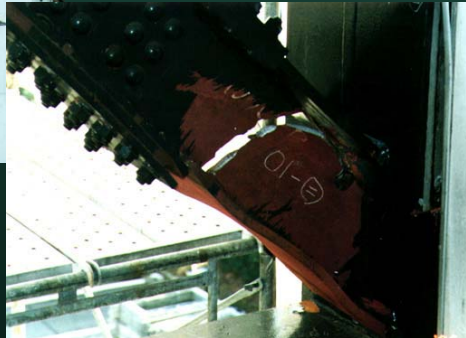


Concentrically-Braced Frames in Earthquakes

Kobe, January 17, 1995



Photos: EQE International



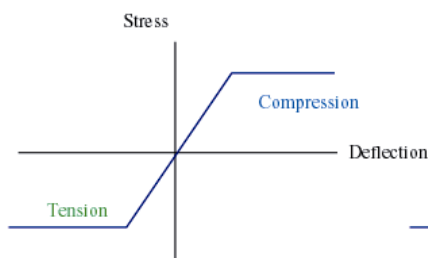
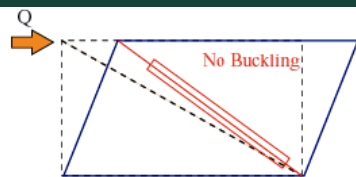
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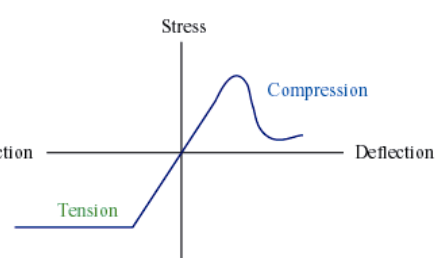
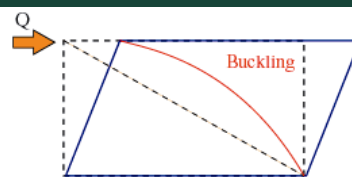
Concept of Buckling Restrained Braces

Unbonded Brace



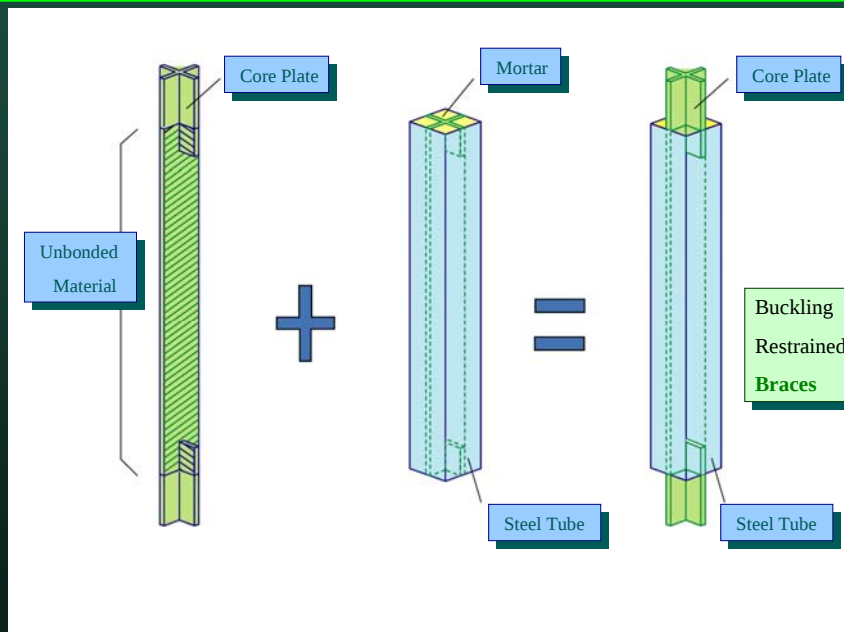
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Conventional Brace



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Components of Buckling Restrained Braces



Steel Members subjected to Compressive Force

- Axial strength of cross section is $A\sigma_{yield}$.
- Maintain stability of member by EI .
-
- In the case of ordinary compression member, there is interaction between $A\sigma_{yield}$ and EI .
- We can decouple between $A\sigma_{yield}$ and EI , in the case of UBB using **Core** and **Jacket**.

Basic Equation for BRBs

$$N_E / N_y > \alpha$$

in which

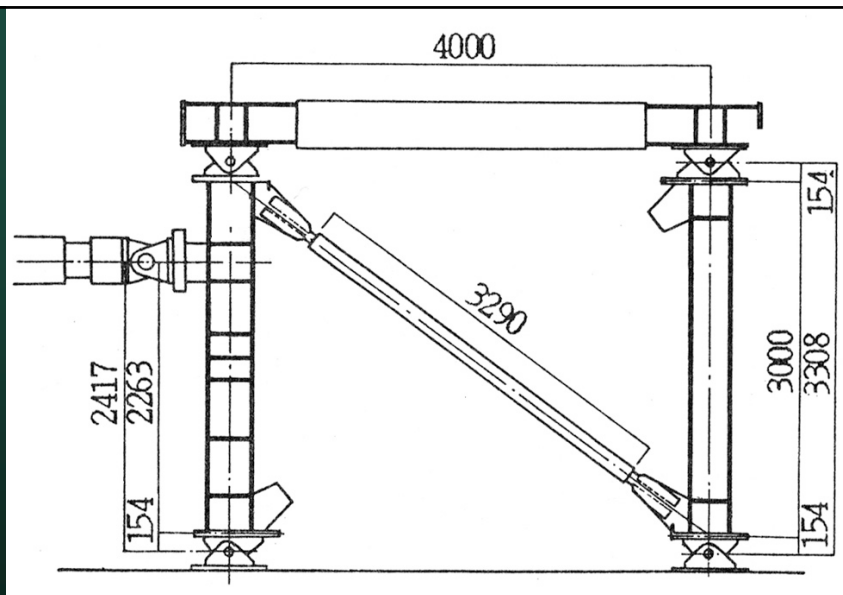
$$N_E = \pi^2 EI / l^2$$

$$N_y = A \sigma_{\text{yield}}$$

we use $\alpha = 1.5$



**First Tests of BRBs in Japan were carried out
in 1987 at Kanagawa University**

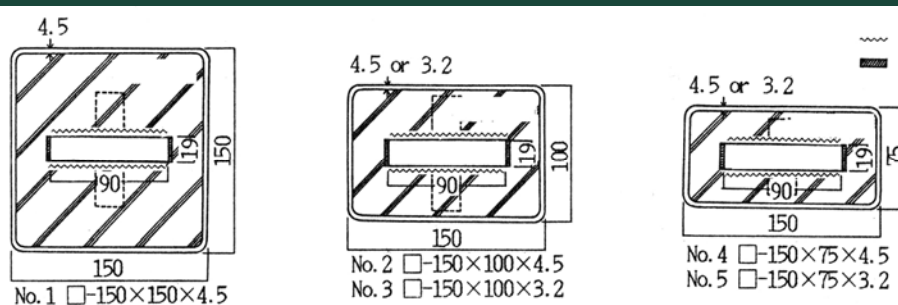


Loading apparatus

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$$P_E/P_y = 3.53$$

$$P_E/P_y = 1.39$$

$$P_E/P_y = 1.03$$

$$P_E/P_y = 0.72$$

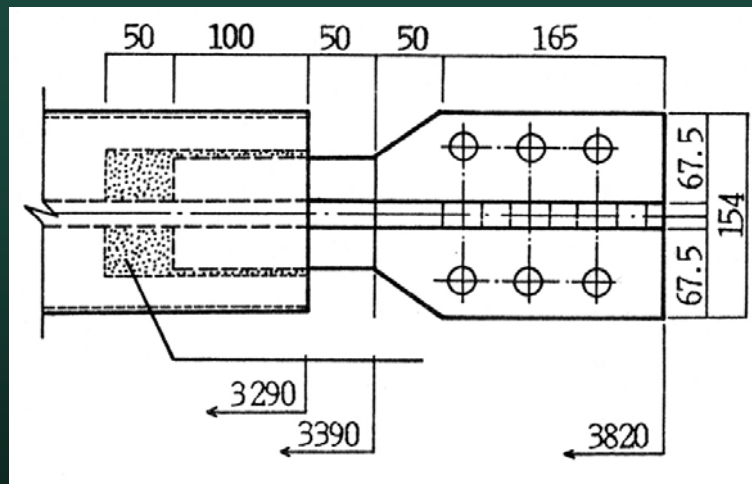
$$P_E/P_y = 0.55$$

Section of 5 Specimens

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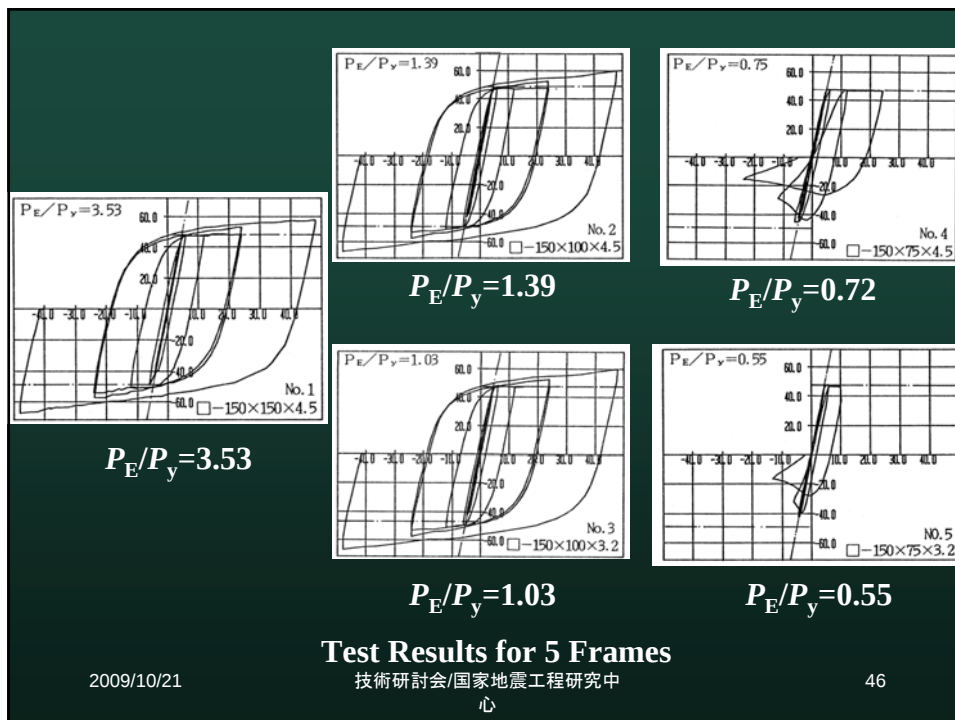


Joint Detail around the End of Brace

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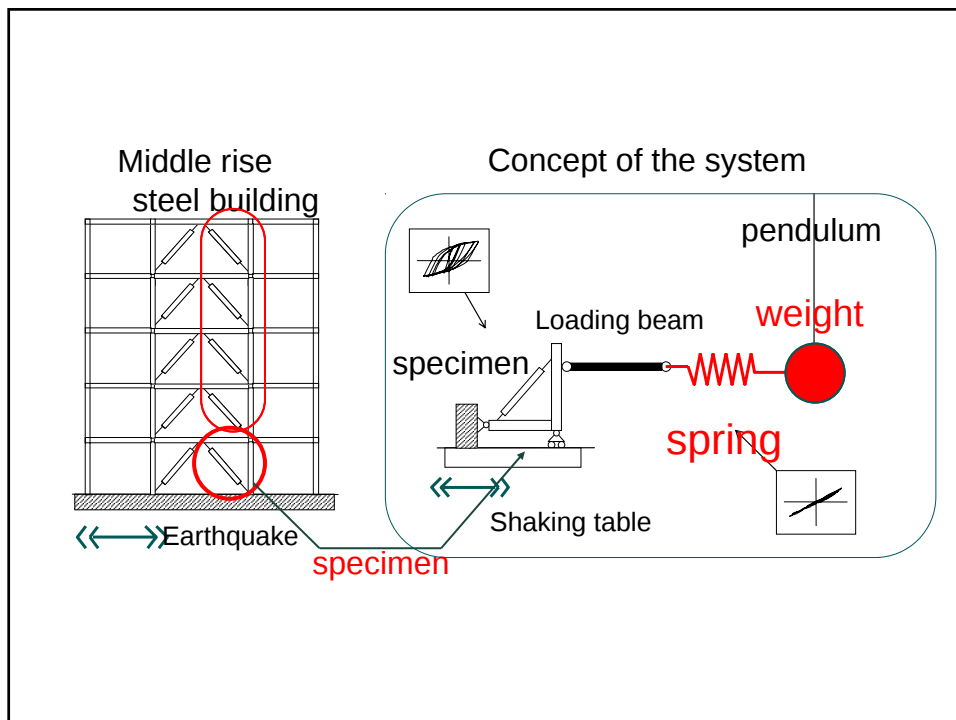
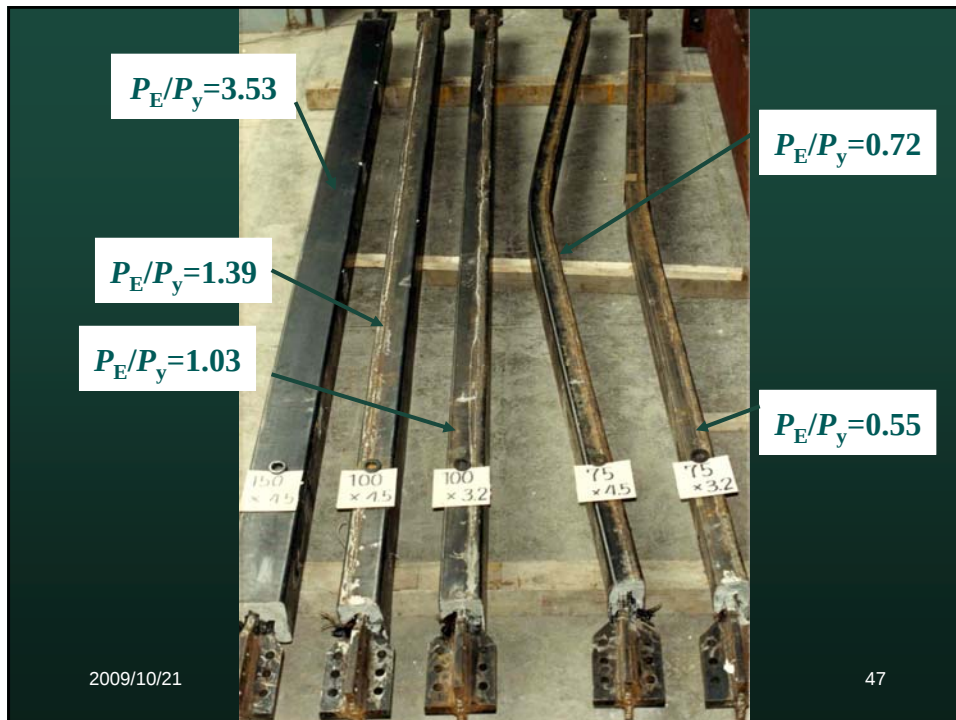
45

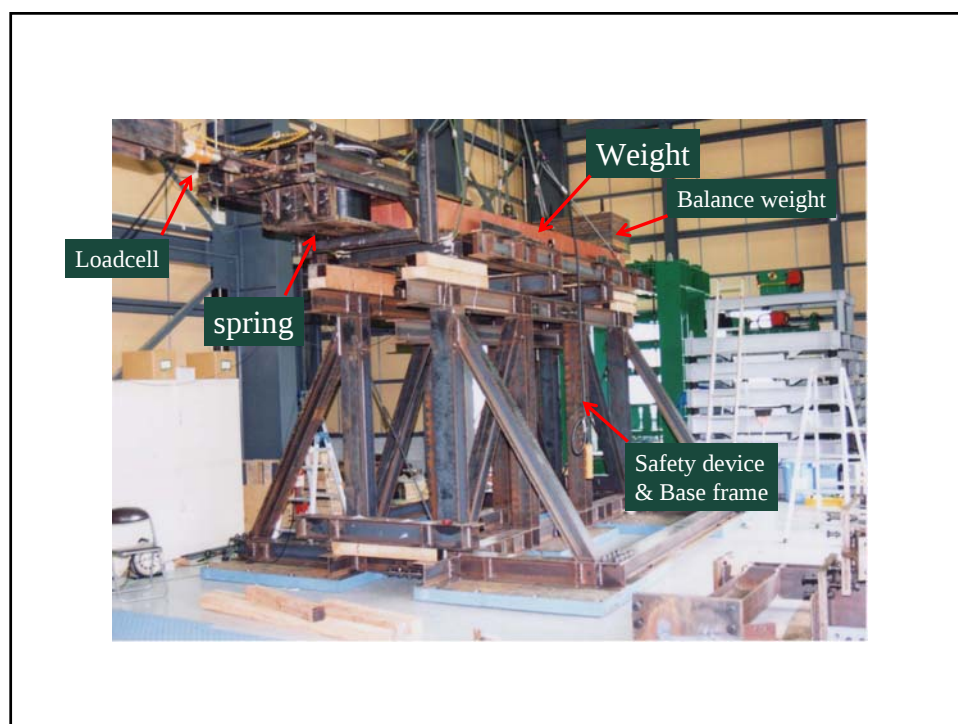
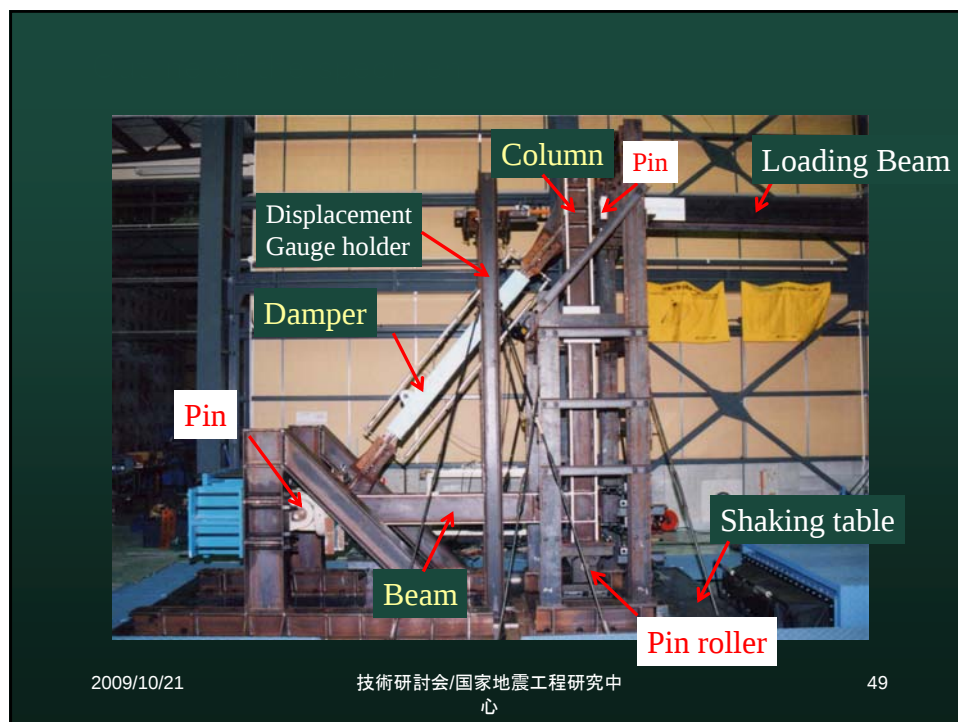


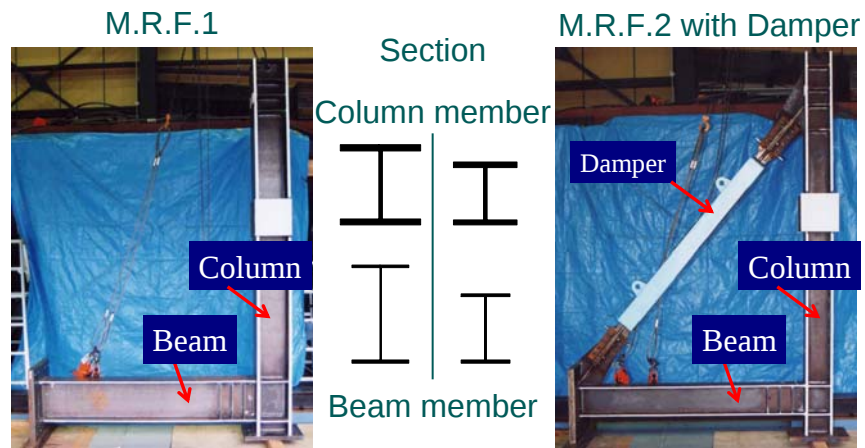
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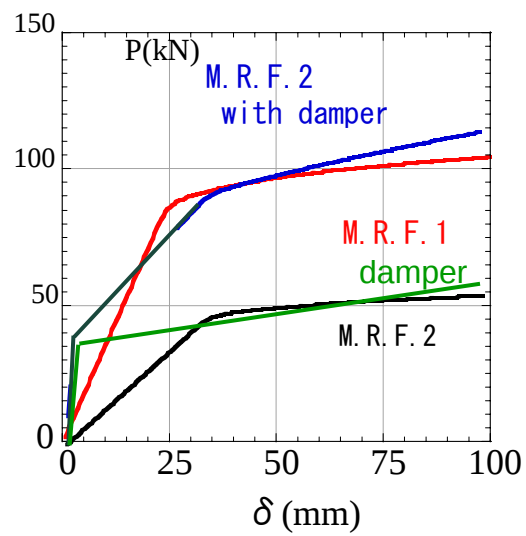




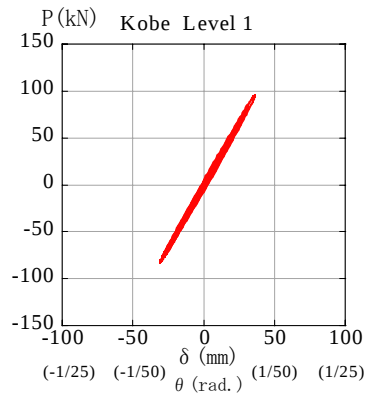


Weight ratio of M.R.F.

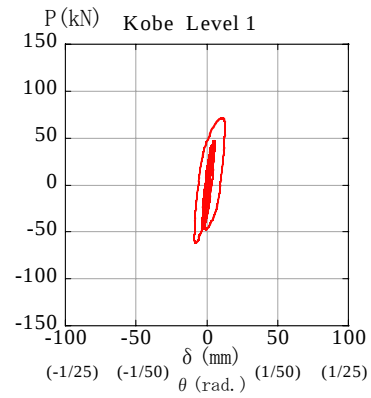
1.00 : 0.80



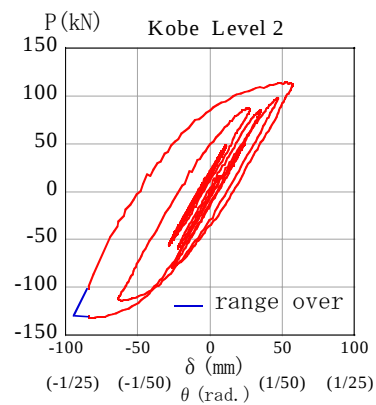
M.R.F.1



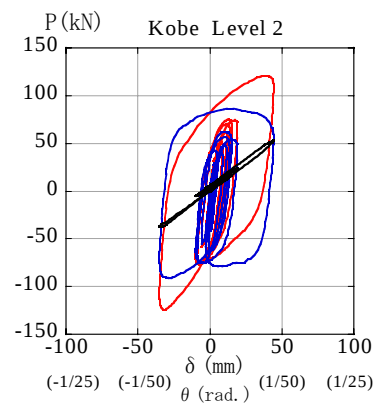
M.R.F.2 with damper



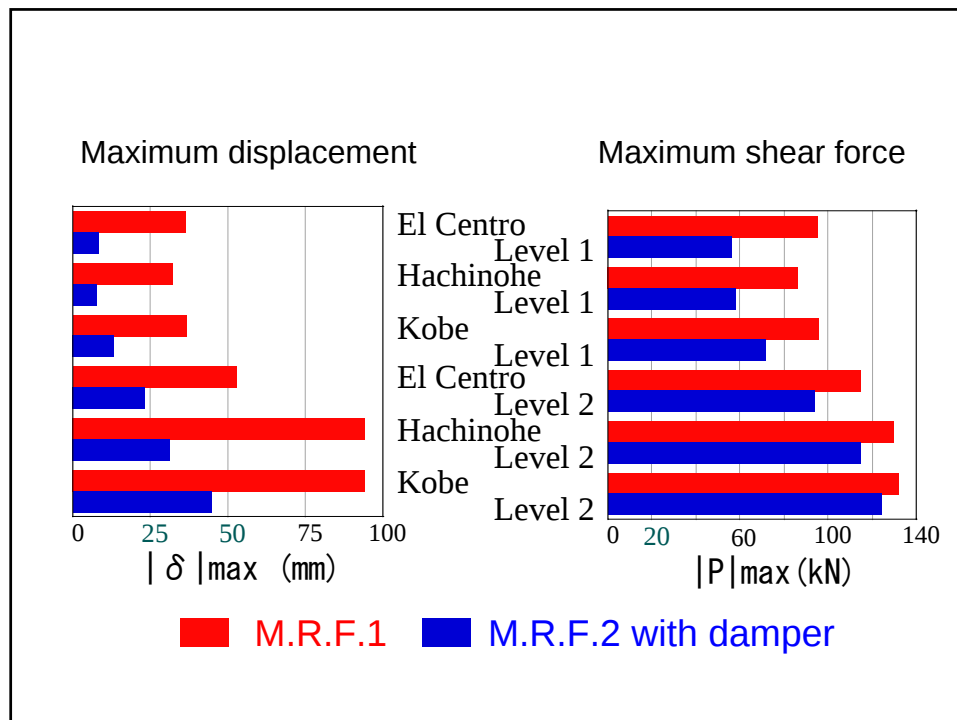
M.R.F.1

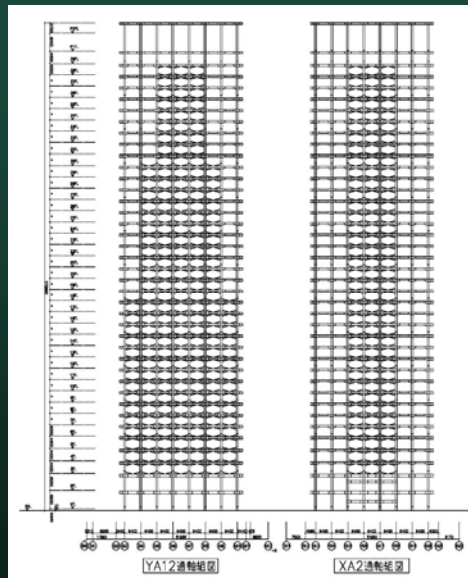
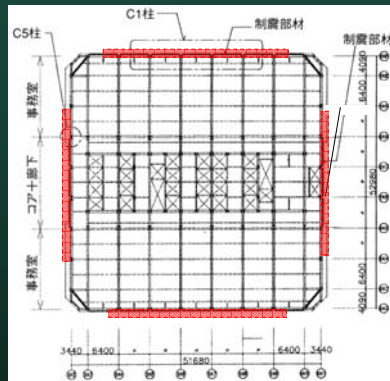


M.R.F.2 with damper



— Specimen — damper — M. R. F. 2



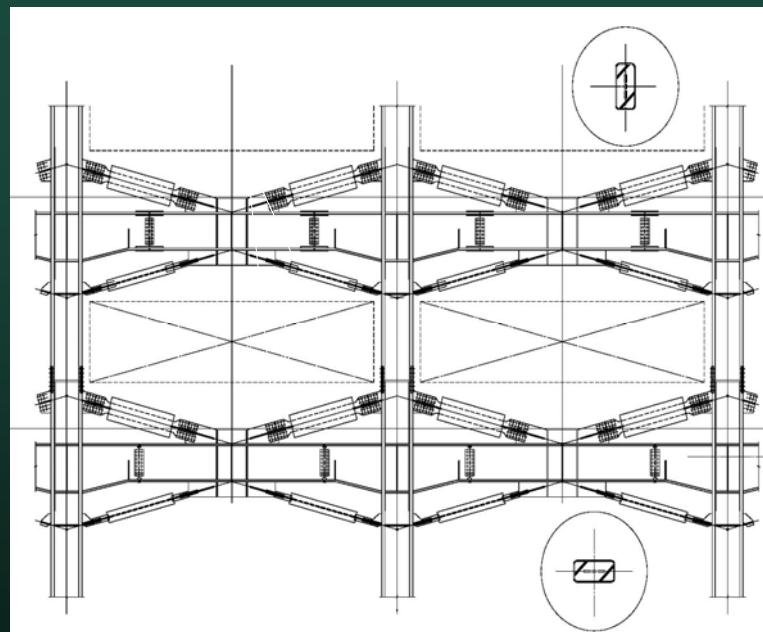


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4.0m



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Osaka International Convention Center



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Courtesy: Arup

技术研讨会/国家地震工程研究中心

- “Damage tolerant” structural design concept
- \$420 million project, completed in 1999
- 721,000 s.f., 13 + 2 stories, h = 300 ft.
- 370 braces, largest 73 ft, $P_y = 1200$ kips

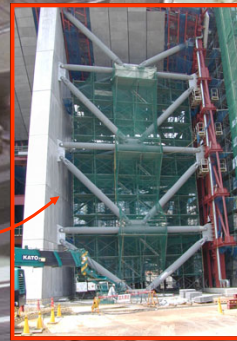


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Toyota Stadium

Unbonded
Braces





Nippon TV Headquarters, Tokyo

- 1.4 million s.f., 32 + 2 stories, $h = 630$ ft.
- 68 braces, $L = 60$ ft, $P_y = 2400$ kips



**We, structural engineers,
have a mission to make
the world better place to live.**

國家地震工程研究中心之 鋼結構耐震技術研究與應用 Researches on Steel Structures in NCEE



蔡克銓 Keh-Chyuan Tsai
National Center for Research on Earthquake Engineering

國家地震工程研究中心 (NCREE)
Dept. of Civil Engineering, National Taiwan University

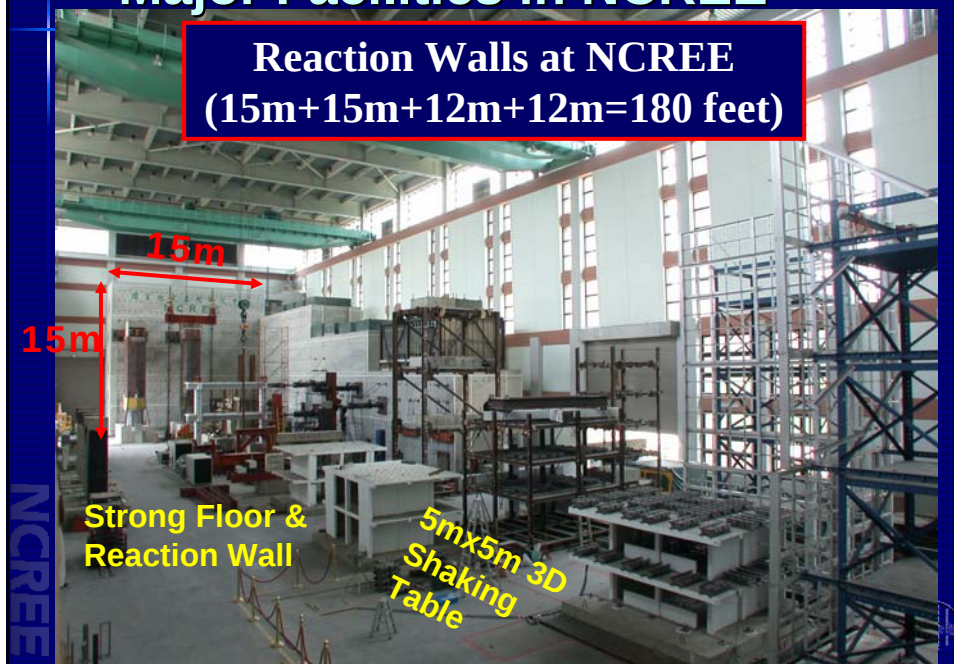
國立台灣大學土木系 (NTU)

鋼結構與隔減震技術研討會
October 9, 2009



Major Facilities in NCREE

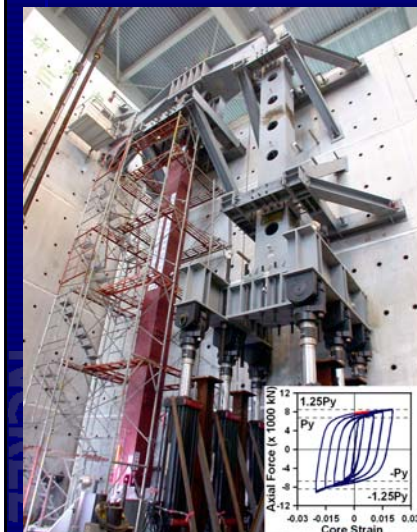
Reaction Walls at NCREE
(15m+15m+12m+12m=180 feet)

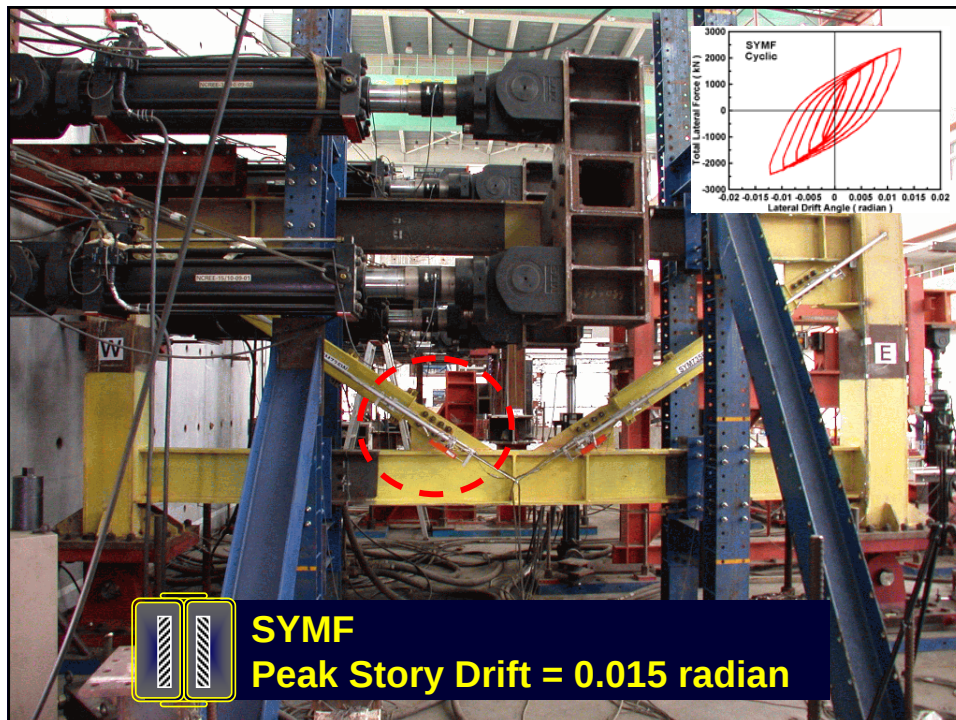


Recent research activities

Large or Full Scale Structural Testing

World Largest BRB
Ever Tested





Earthquake Engineering Simulation Examples

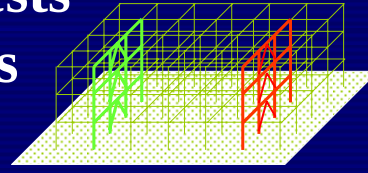
Shake Table Tests
Cyclic Loading Tests
Pseudo Dynamic Tests
Numerical Simulation Platforms
Substructure Hybrid Tests
Multi-sited Networked Hybrid Tests
Field Tests

NCREE

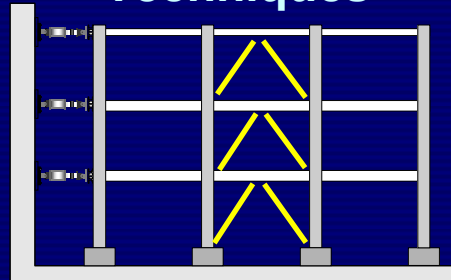
Hybrid Simulation Tests

Pseudo Dynamic Tests

$$\mathbf{Ma} + \mathbf{CV} + \mathbf{Kd} = -\mathbf{Ma}_g$$

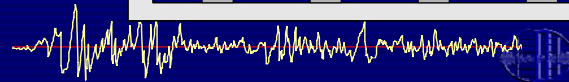


Substructuring Techniques



Ground accelerations

$\mathbf{a}_g$



Hybrid Simulation Tests

Pseudo Dynamic Tests

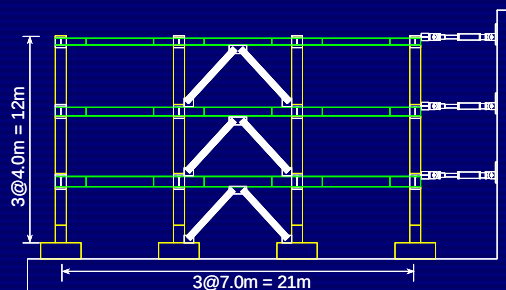
$$\mathbf{Ma} + \mathbf{CV} + \mathbf{Kd} = -\mathbf{Ma}_g$$

$\mathbf{a}_g, (\mathbf{a}, \mathbf{v}, \mathbf{d})_n$



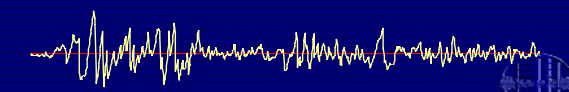
$\mathbf{d}_{n+1}$

$\mathbf{Kd}_{n+1}$



Ground accelerations

$\mathbf{a}_g$





ISEE (NCREE/Taiwan) Internet-based Simulation on Earthquake Engineering

Networked Earthquake
Engineering Simulation
NEES (NSF/USA)

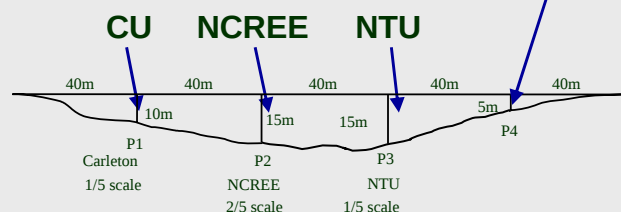
Multi-site Distributed Hybrid Simulations

Transnational Experiment

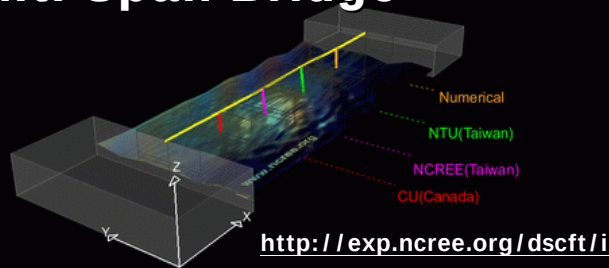
Characteristics of the experiment:

- Bi-lateral
- Transnational
- Hybrid facility systems
- Hybrid scale factors

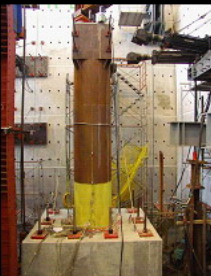
Option 1 : Numerical Simulation
Option 2 : Hunan Univ.



Distributed Hybrid Testing of a Multi-Span Bridge



CU (1/5 scale)



NCEE (2/5 scale)



NTU (1/5 scale)

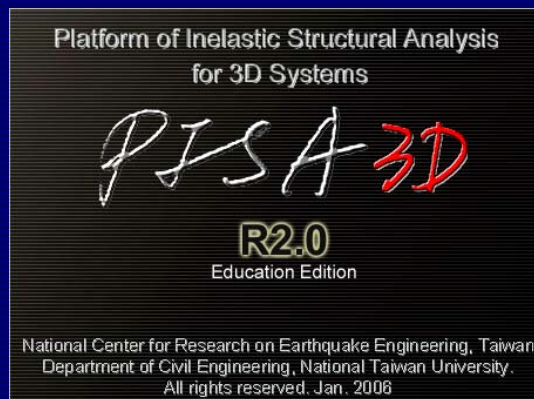
NCEE



Earthquake Simulation: Numerical Computations and Visualizations

PISA3D, VISA3D, GISA3D

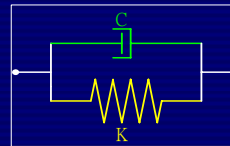
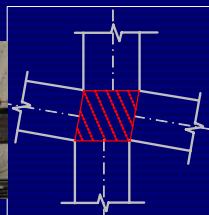
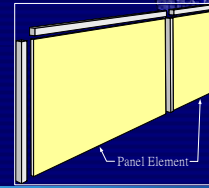
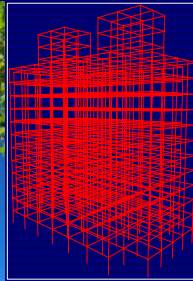
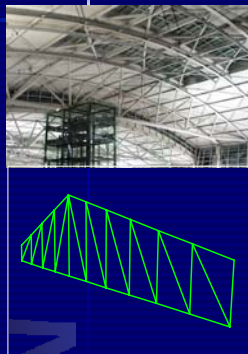
Object-oriented Nonlinear Structural
Analysis Software



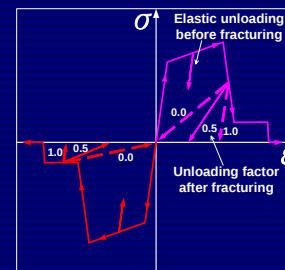
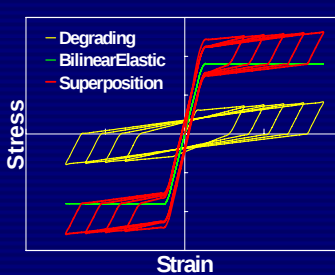
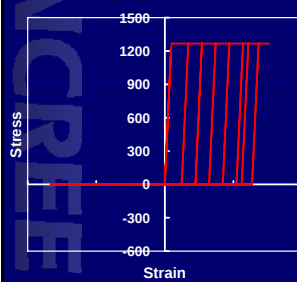
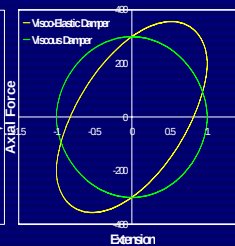
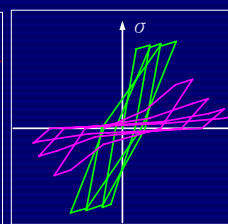
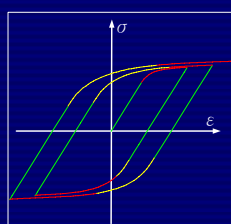
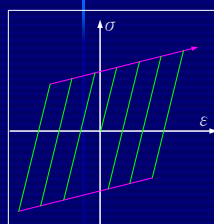
NCEE

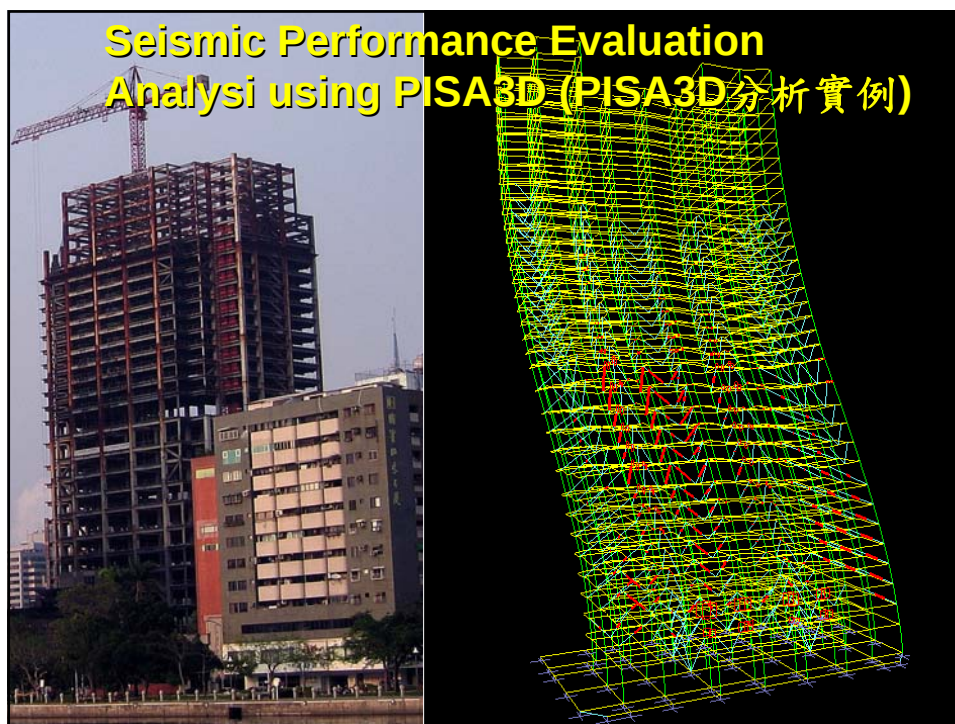
National Center for Research on Earthquake Engineering

Nonlinear Elements

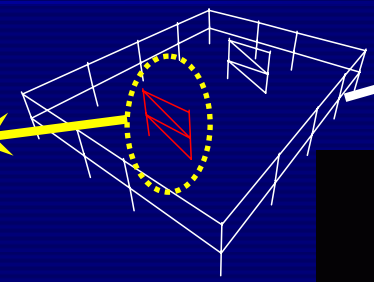


Nonlinear Materials





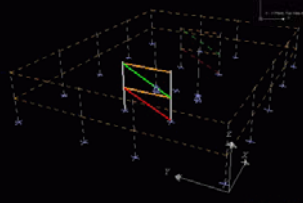
Sub-structural Pseudo Dynamic Test on the Full Scaled 2-story BRBF




<http://substructure-brbf.ncree.org>

National Center for Research on Earthquake Engineering
Test: SPDT No.1
Amp. Factor of Response = 100
Time Step = 0.00 sec.



— 1000KN

○ 0.01 rad Plastic Hinge

• Yielding in Axial Direction

$Fx1 = 0.00\text{ KN}$

$Fx2 = 0.00\text{ KN}$

$Fy1 = 0.00\text{ KN}$

$Fy2 = 0.00\text{ KN}$

GISA3D Visualization

Experimental Setup



6 External Displacement Transducers

Vertical loads applied by Post Tension Rods

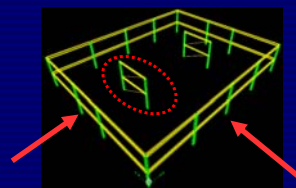
Substructure Pseudo Dynamic Tests

- Benefits

Full Scale

Cost Effective

Real Demand / Real Capacity



- Requirements

A good substructure specimen

A good analytical model

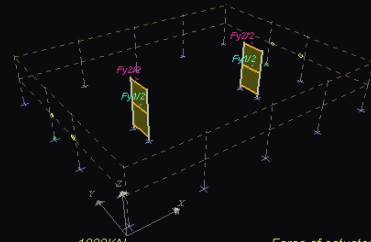
A good test bed

A good set of boundary conditions

Sub-structural Pseudo-Dynamic Tests of a Two-Story Steel Plate Shear Wall Frame



National Center for Research on Earthquake Engineering
Test: SPDT SPSW4
Amp. Factor of Response = 20
Time Step = 16.00 sec.



Force of actuators:
 $Fy2 = 212.44 \text{ KN}$
 $Fy1 = -115.33 \text{ KN}$

<http://exp.ncree.org/spsw/>

Application: Two-story X-brace and Inverted V-brace CBF

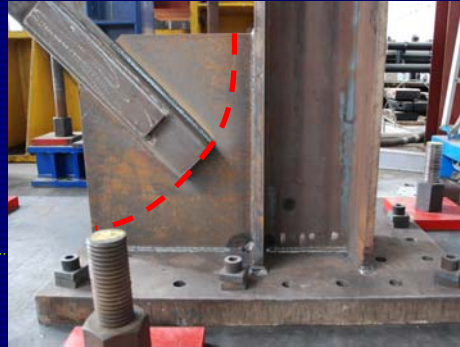
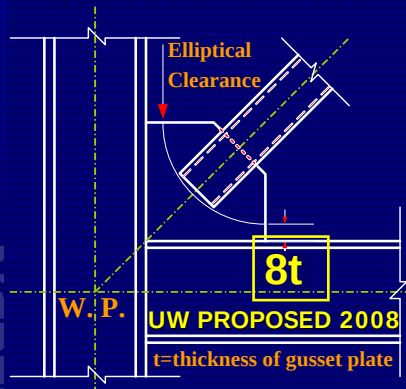


Test 1

Phase I (HSS-8t):

Braces: (A500)

Gusset plate connections:

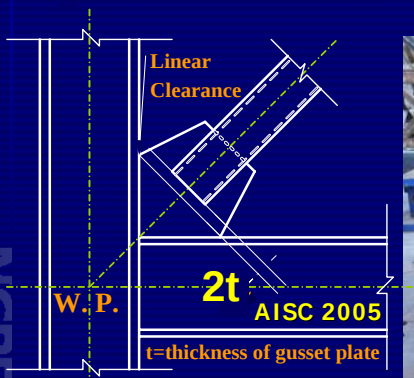


Test 3

Phase III (HSS-2t):

Braces: (A500)

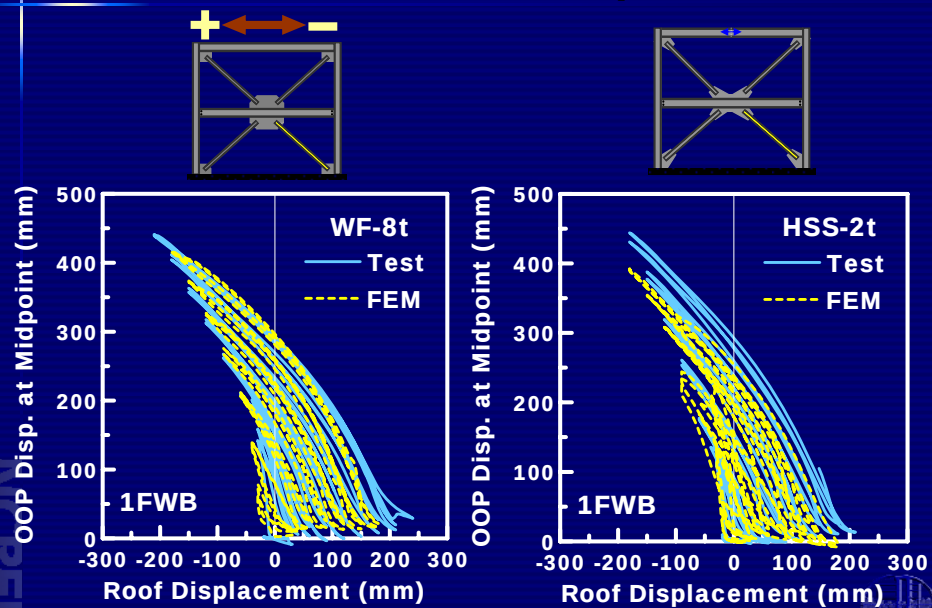
Gusset plate connections:



Braces and gusset plates buckled



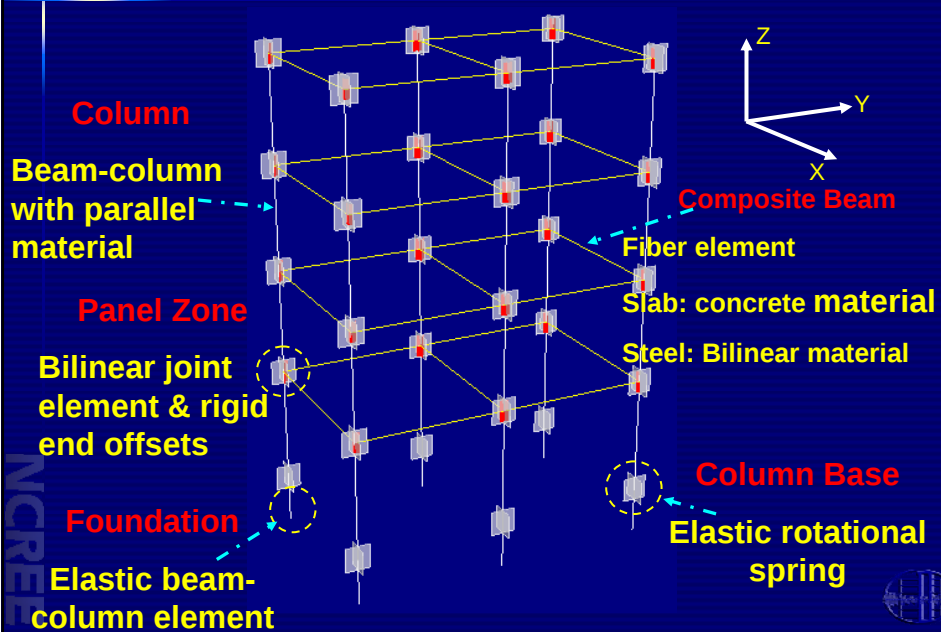
Test vs FE Brace OOP Displacements



NCREE's Participation in the 2007 E-Defense Blind Analysis for A Full-Scale 4-Story Steel Moment Resisting Frame



PISA3D Structural Model



Refined model: PZSlab

The diagram illustrates the refined model for PZSlab, showing a 3D view and a cross-section view.

3D View (Top):

- The structure is a rectangular slab with dimensions 5000 (width) and 6000 (depth).
- Supports are located at the corners, labeled C1 (bottom-left and top-right) and C2 (bottom-right and top-left).
- Stiffeners are labeled G1 (horizontal) and G11 (vertical).
- Points A and B are marked at the corners of the slab.
- Points 1, 2, and 3 are marked along the bottom edge.
- Coordinate axes X and Y are shown.

Cross-section View (Bottom):

- The cross-section shows the slab (gray) supported by a composite beam (yellow).
- The composite beam consists of a steel I-beam (yellow) and a concrete slab (gray).
- A red dot on the slab indicates the location of the mass.
- The reference axis of the composite beam is indicated by a dashed line.

Refined model: PZSlab

$$M_y = \frac{0.55 F_y t_j d_b d_c}{\left(1 - \frac{d_b}{H}\right)}$$

The diagram illustrates the PZSlab refined model under two loading conditions: positive bending moment (M_y^+) and negative bending moment (M_y^-). In the positive moment case, the slab is shown with a central column and a beam. The effective depth of the slab is labeled d_{eb} , and the effective depth of the beam is labeled d_c . The slab is shown with a yellow diagonal line indicating the location of the plastic hinge. In the negative moment case, the slab is shown with a central column and a beam. The effective depth of the slab is labeled d_b , and the effective depth of the beam is labeled d_c . The slab is shown with a yellow diagonal line indicating the location of the plastic hinge.

Number of Participants

- 47 teams from 7 countries in total
(7 teams participate in both 3D and 2D analysis.)

Table : Number of Participants

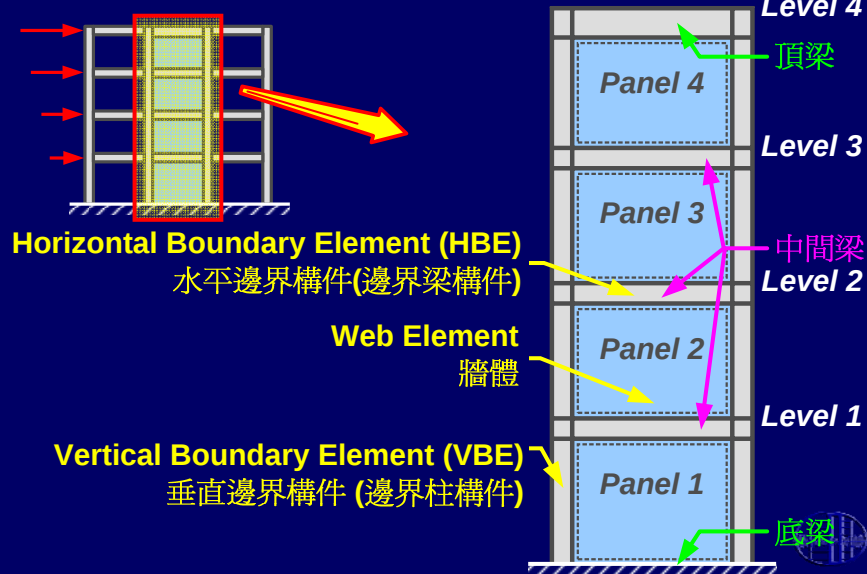
Country \ Category	3D-R	3D-P	2D-R	2D-P	TOTAL
Japan	6	5	4	2	17
U.S.	6	5	2	2	15
Taiwan	4	0	4	0	8
China	1	1	2	0	4
N.Z.	0	1	0	0	1
Italy	1	0	0	0	1
U.K.	0	0	0	1	1
TOTAL	18	12	12	5	47

3D : Three-dimensional Frame Analysis 2D : Plane Frame Analysis

R : Researcher P : Practicing Engineer

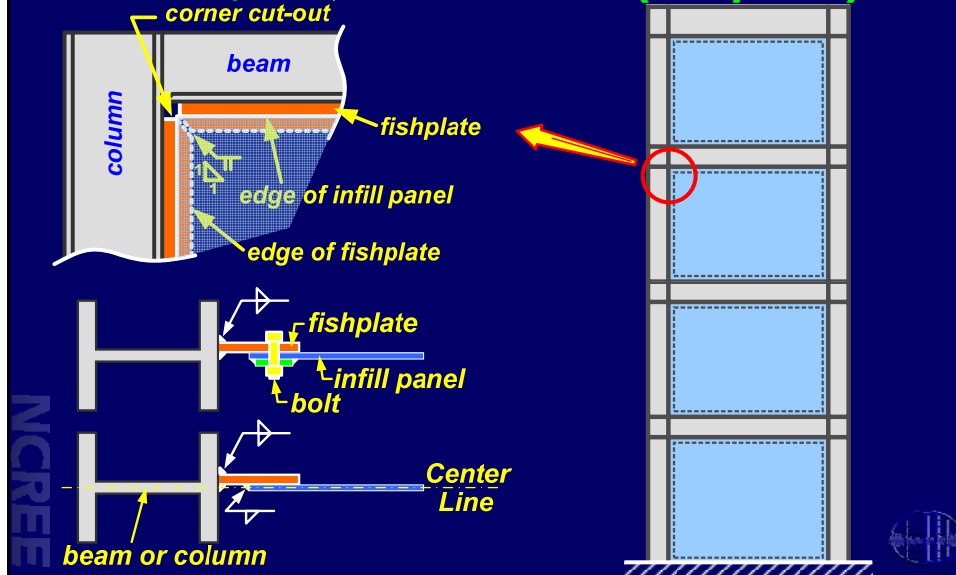
簡介

- 鋼板剪力牆 (steel plate shear wall, SPSW)



簡介

●鋼板與邊界構件透過魚尾板(fishplate)接合



國外應用案例

●U.S. Federal Courthouse, Seattle



國外應用案例

●Core wall of ING buildings, Canada



NCREE



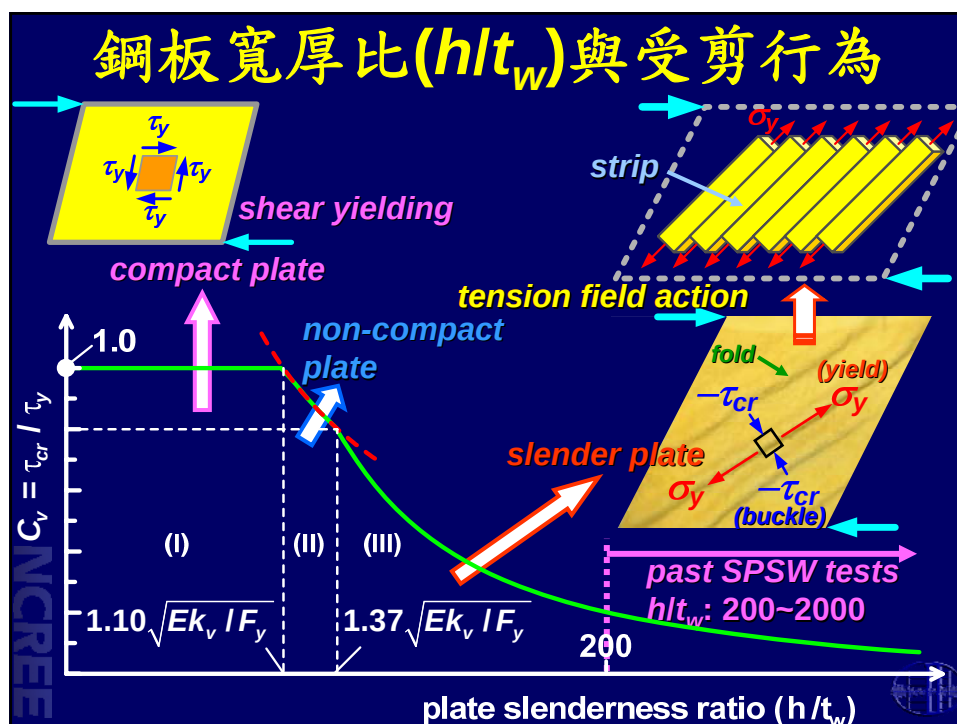
國外應用案例

●SPSW building in Mexico - elevator core



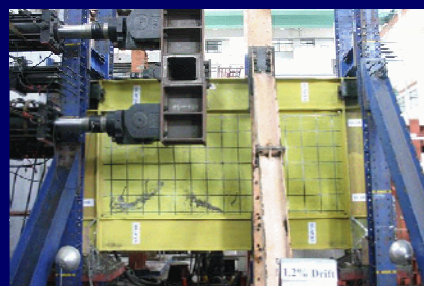
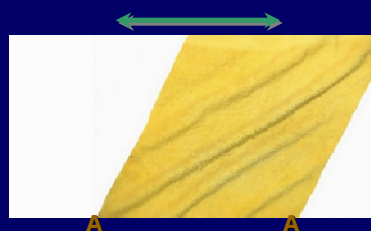
NCREE





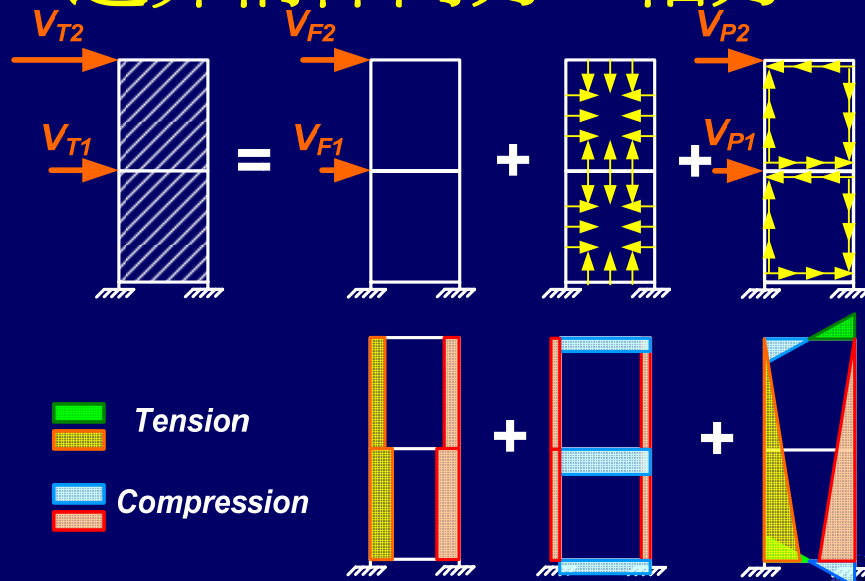
拉力場行為 Tension Field Action

- After buckling in shear, the thin plate behaves in a manner similar to a series of trusses. The tension force carried by tension-field action.
- By allowing the infill plate to develop diagonal tension field action, and then dissipate energy through the cyclic yielding of the infill in tension.

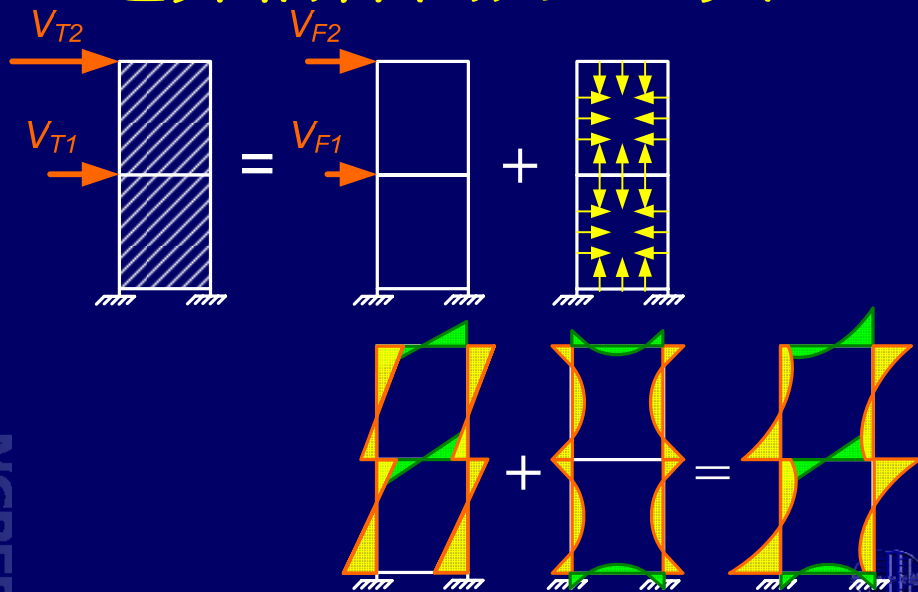


(蔡克銓與林盈成, 2004)

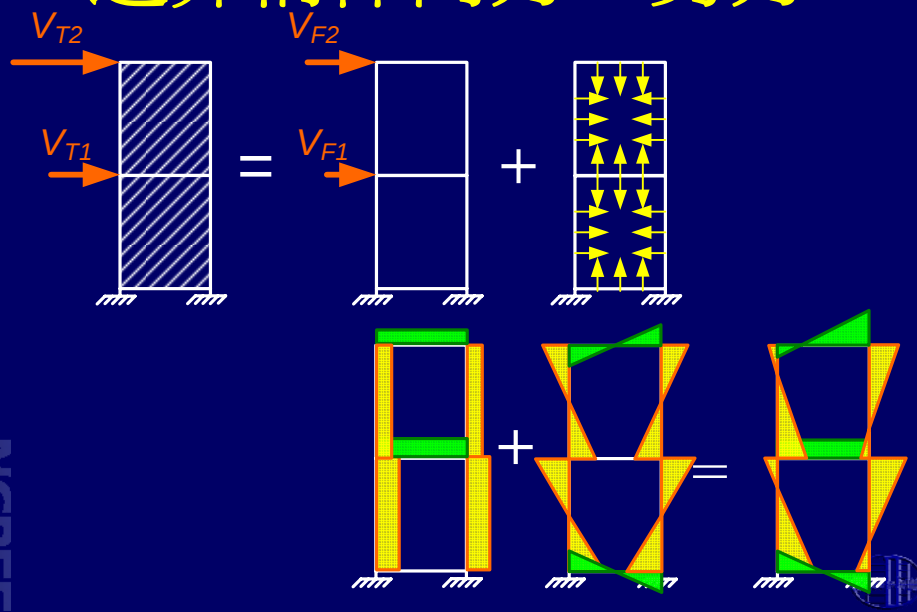
邊界構件內力－軸力



邊界構件內力－彎矩

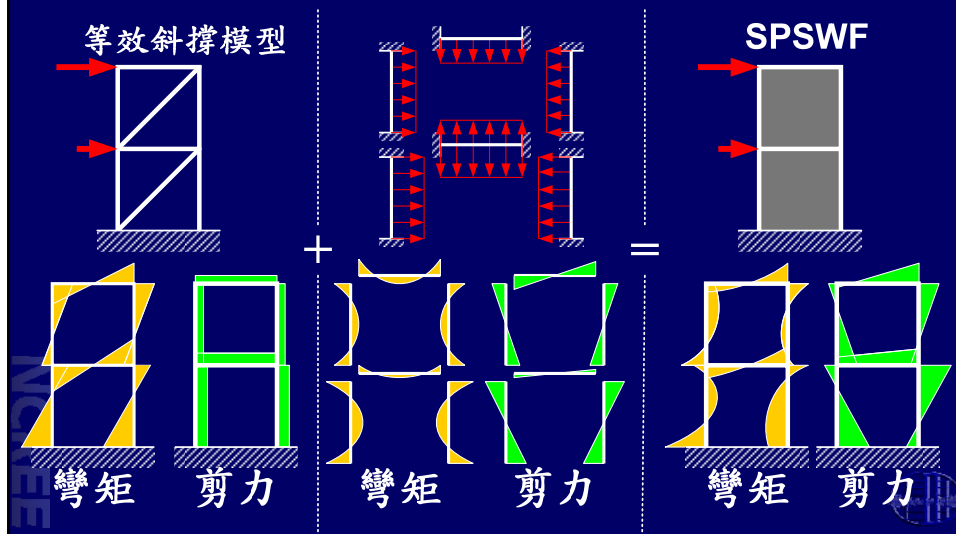


邊界構件內力－剪力



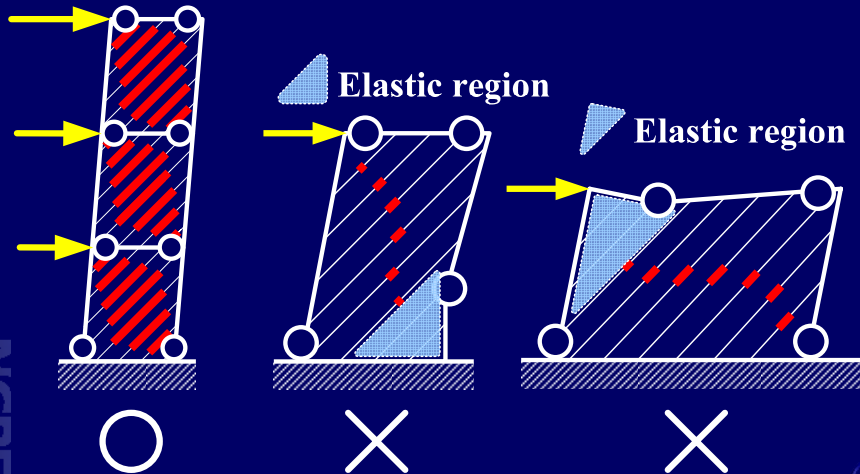
運用疊加法求算構件內力

●構架側位移行為 + 牆體拉力效應



容量設計目標

- 確保塑鉸發生在適當位置



側推反應

(1) 彈性階段
(State-E)

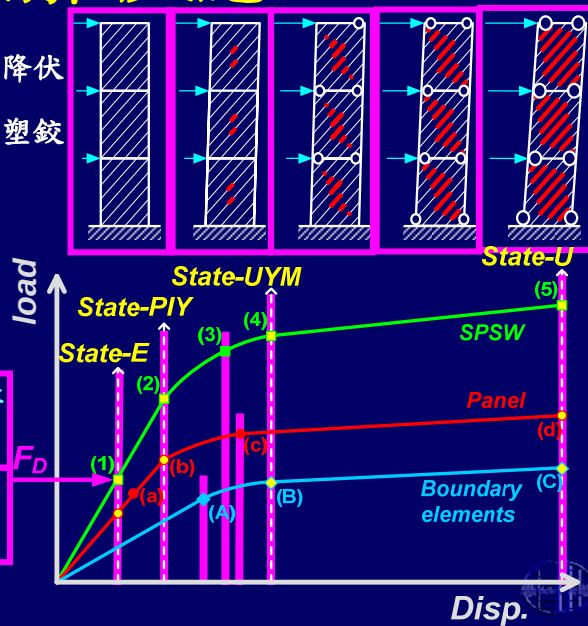
(2) 牆體初始降伏
0.3~0.5% Rad.

(3) 構架明顯降伏
0.5~0.75% Rad.

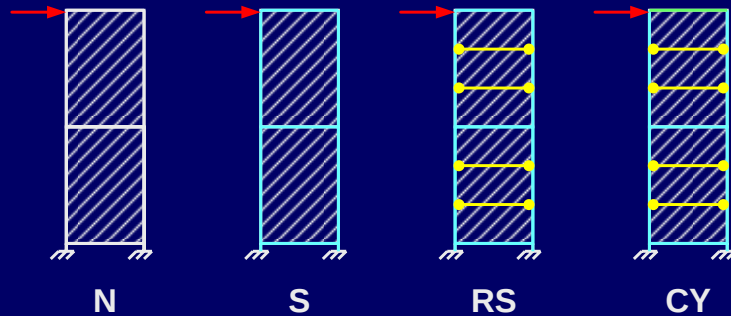
(4) 均勻降伏塑性機構
1% Rad.

(5) 極限狀態
2.5% Rads.

— 鋼板
— 拉力降伏
○ 撓曲塑鉸



Test Program



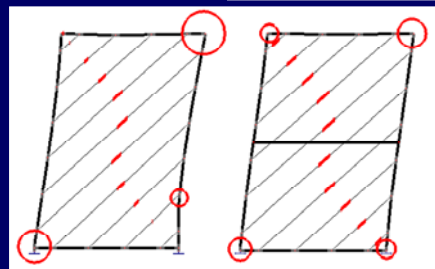
- Four 2-story SPSW specimens
- Story height is 3250 mm and the width is 2140 mm
- All panels are 2.6 mm thick LYS100 steel
- Key parameter is the boundary element size
- Cyclic loading tests up to 5% radians roof drift

2007 瘦長型鋼板剪力牆試驗

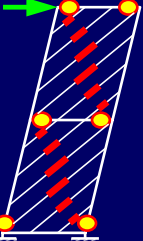
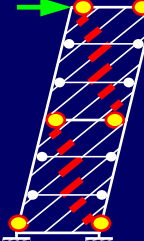
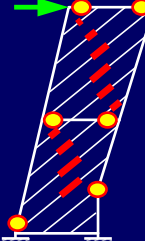
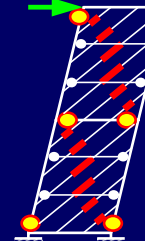


Objectives

- Column Capacity Design
- Narrow SPSW
- Restrained SPSW



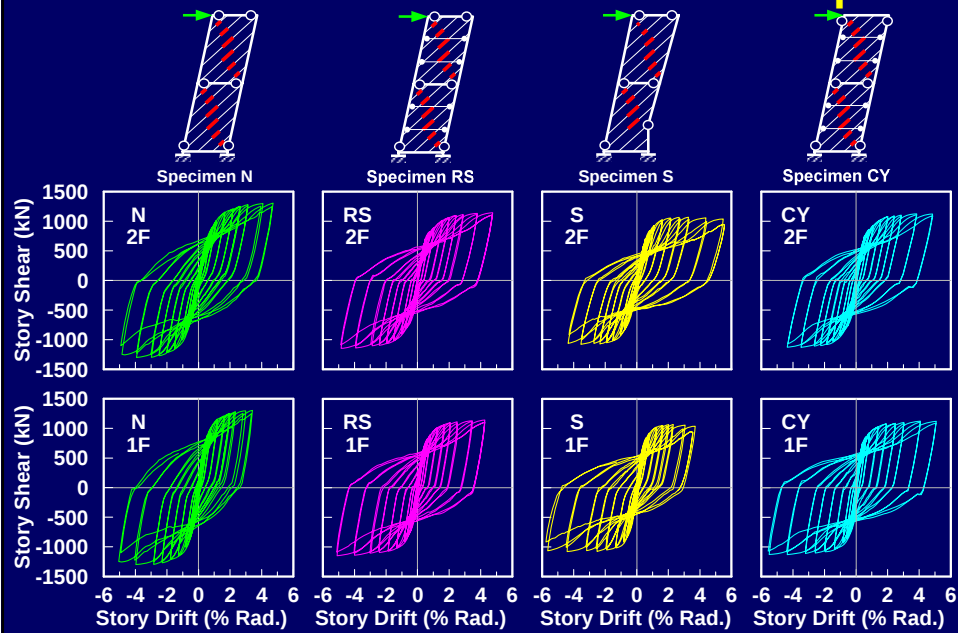
Specimen Design

			
			
Specimen N	Specimen RS	Specimen S	Specimen CY
Normal BE	R-SPSW with Small BE	Small BE	Column Yield at the column top
<i>"Comply with" the capacity design</i>	<i>"Comply with" the capacity design</i>	<i>"Do not comply with" the capacity design</i>	<i>"Comply with" the capacity design</i>

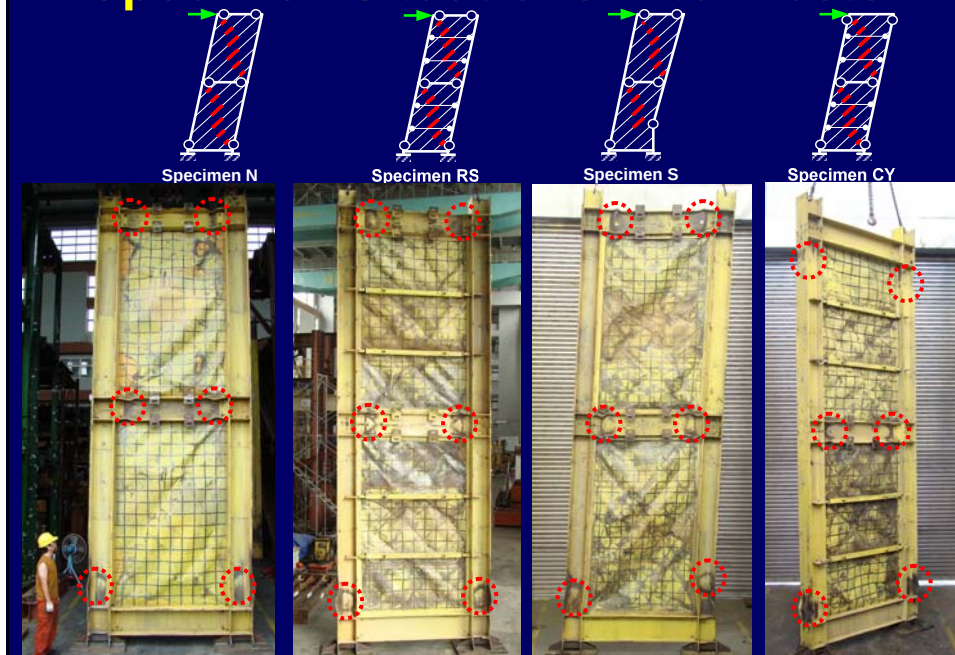
Restrainer Details



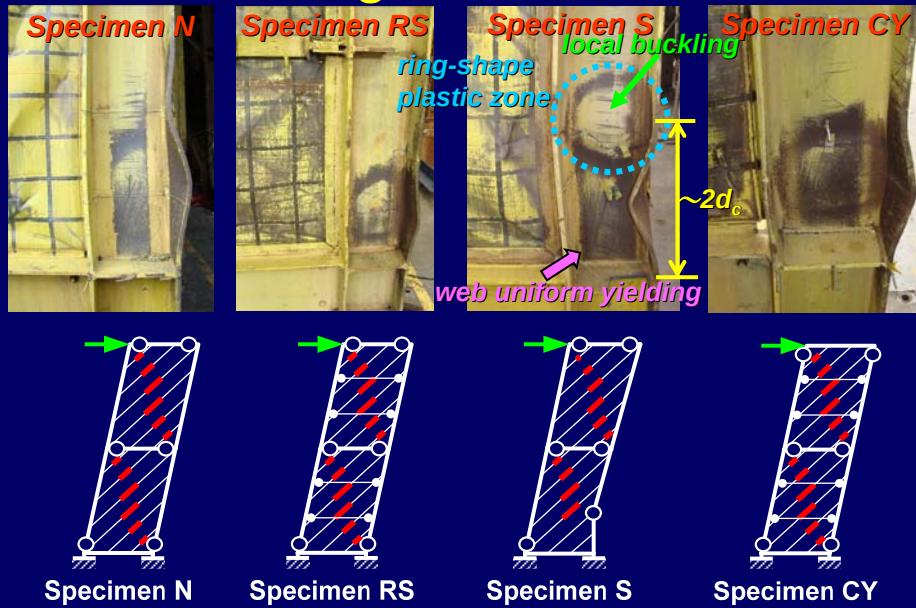
Force vs. Deformation Relationships



Specimen Situations After Tests



Plastic Hinges at the Columns



Thank you for your attentions !

國家地震工程研究中心

NCREE

特殊鋼骨結構物的製作及現場工程

-日本實例介紹-

新日鐵工程技術株式会社
白井貴志

新日鐵エンジニアリング株式会社

- 羽田空港国際線
旅客 Terminal Building
(東京)



- 沖縄県立博物館・美術館
(沖縄)



- 勝鬨GLASS CANOPY
(東京)



- NTT DOCOMO大阪 鉄塔
(大阪)



- 東京 Sky Tree
(東京)

ドコモ大阪第二ビル(仮称)新築工事 鉄塔工事

2005年度 日本鋼構造協会 業績賞 受賞

概要	
所在地	大阪市住之江区南港1-29-1
施工主	(株)エヌ・ティ・ティ・コム関西
設計	(株)エヌ・ティ・ティ ファシリティーズ/(ArupJapan)
施工	大林・竹中・鹿島・鴻池・共立・不動・新井 共同企業体
主要用途	通信鉄塔
建築面積	5,379.91m ² (下部建屋) 鉄塔最頂部高さ 198.455m
延べ床面積	60,993.42m ² (下部建屋)
鉄骨重量(当部施工)	1750ton (本体750ton 付帯200ton ケーブル250ton 鋳鋼550ton)

特徴

世界初の本格的ケーブル構造形式の鉄塔
鋼管柱(1700φ最大板厚80mm)と、各プラットホーム床の四隅にケーブルを配置した張弦構造の柱を8本のスチーケーブルにて支持した架構。N-PWS(7×499)ケーブル使用



全景



全景



マスト支承部取合(ステンレス硬化肉盛溶接)



施工中写真(仮支線にてリポート)

鋼結構與隔減震技術研討會

鋼結構設計之垂直載重系統與側向載重系統 之設計原理與實例

張敬禮 King-Le Chang

傑聯國際工程顧問

Jaelien International Engineering Consultants

2009.10.09

KLC

結構系統設計

- 建築物結構系統之規劃 (結構體的精神)
 - 側向系統與垂直載重系統的關係
 - 幾何的應用 (質心與剛心)
 - 能量消散機制
 - 設計規範的執行
 - 與建築設計的融合
- 側向系統設計
 - 集中及有效的側向系統
 - 合理及可驗證的消能機制
 - 明確的破壞程序
 - 耐震接頭設計
 - Diaphragm 的設計
- 垂直載重系統設計

KLC

Core Pacific City . Taipei, Taiwan (京華城 - 台灣, 台北)

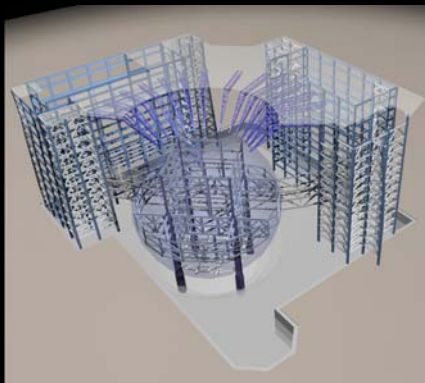


- 14 Storey above grade
- Retail and Entertainment
- 7 Storey below grade
- Retail, Parking and Services
- Total Building Area : 200,000m²
- Focal point - 11 stories sphere
- 60m diameter
- Support on only 4 columns



KLC

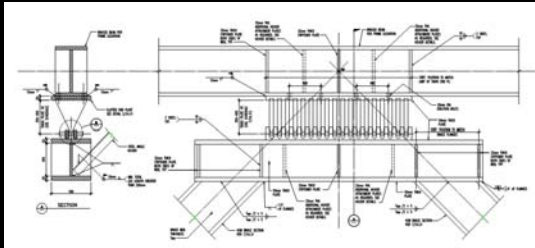
Structure System



- Dual Lateral System of Steel Special Moment Resisting Frame with TADAS Brace Frame
- Perimeter Moment
- TADAS Brace Frame Cores
- Sphere carried by 4 Mega Columns (2m x 2m)
- 2 storey truss between Level 5 and 7
- Level 2 to 4 hanging below
- Level 9 to 11 on column above
- Link elements between sphere and main building form Level 5 to 10
- Dual system with sphere Moment Frames

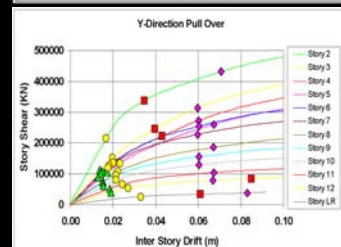
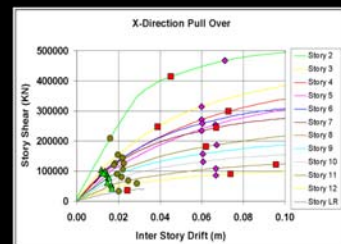
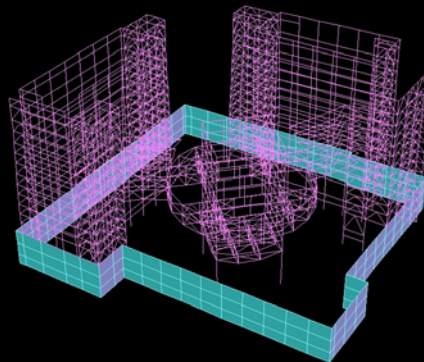
KLC

Triangular Added Damping and Stiffness (TADAS)



- The device consists of a series of triangular plate welded to a base plate under a beam above. The foot of each plate is detailed such that it can transmit only lateral forces to the brace below. No vertical load is carried in the device.
- This arrangement has proved to be very predictable in testing.
- Typical examples are capable of attaining rotations for in excess of those corresponding to code based seismic drift limits.

SAP2000 – 3D analysis model



▲ TADAS yield ● Beam first yield
◆ Drift Ratio Limit = 0.014 ■ Column first yield

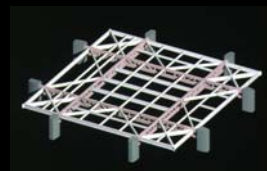
Static Pushover Analysis

CEC Headquarters Building. Taipei, Taiwan (大陸工程大樓 - 台灣，台北)

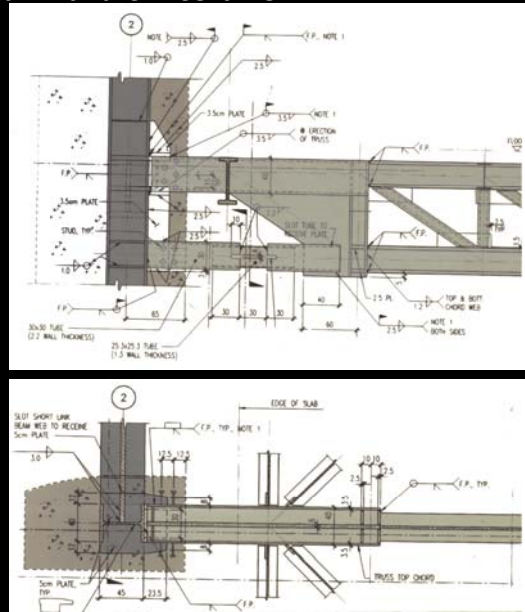


- 13 Storey above grade
- 4 Storey below grade
- Building Height : 60m
- Total Building Area : 17,572m²

Structure System



Floor Beam Failure Mechanic



KLC

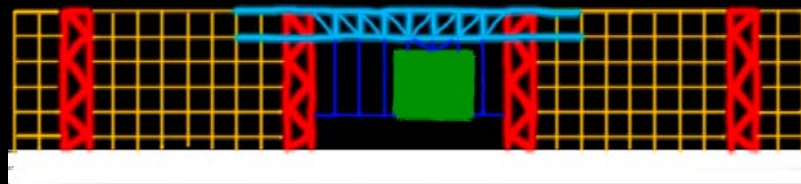
The New Taichung City Government Building. Taichung, Taiwan (台中市政府大樓 - 台灣，台中)



- 10 Storey above grade - Office
- 2 Storey below grade - Parking and Services
- The assembly hall "cube" and the grand opening at level 4 are the focal points of the building.
- The "cube" and the 67.2 meters opening are supported by story-height truss system spanning 67.2 meters between two brace core.
- Total Building Area : 120,000m²

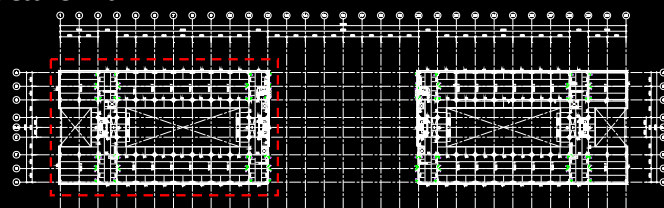
KLC

Design Concept

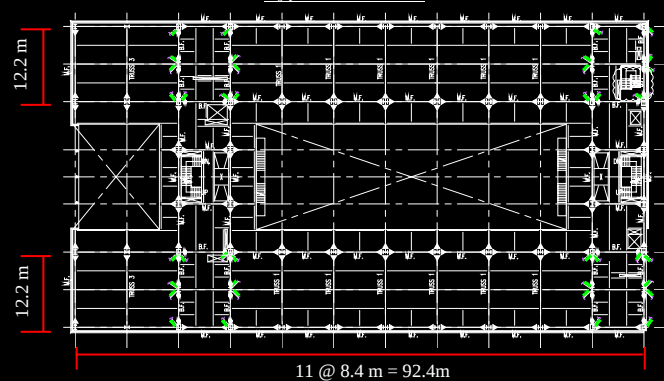


- 4 Brace Cores
- Two dimension roof truss system
- Dual Lateral System of Steel Special Moment Resisting Frame with Buckling Restrained Brace (BRB)

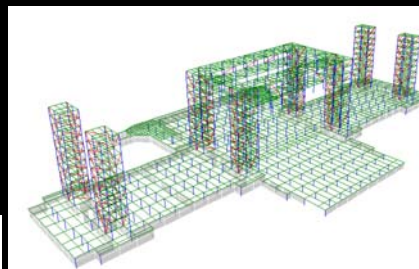
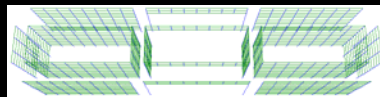
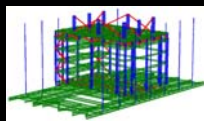
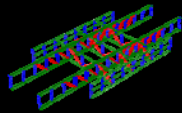
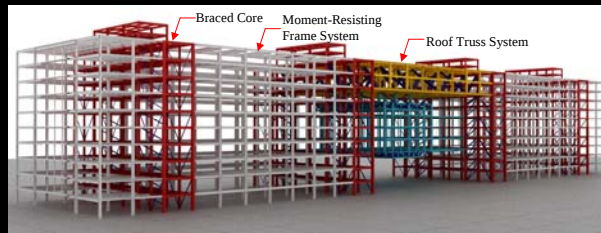
Structure Plan



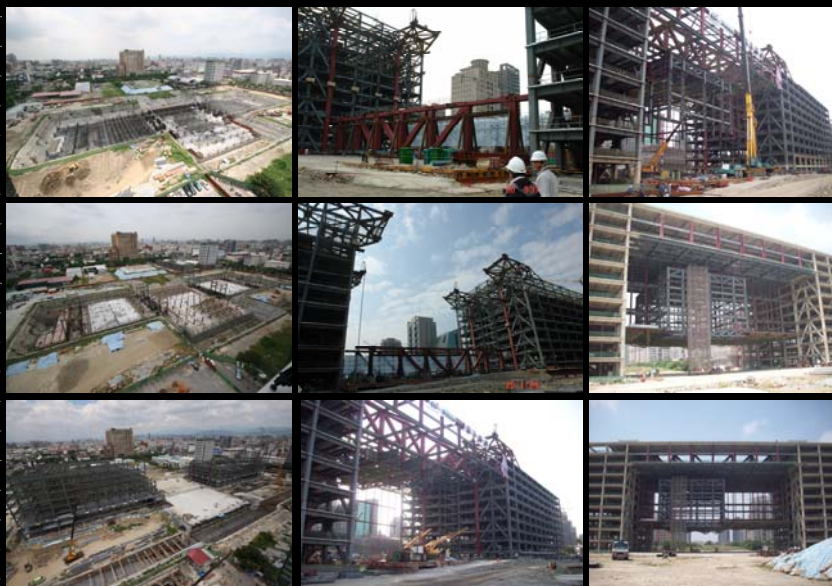
Typical Floor Plan



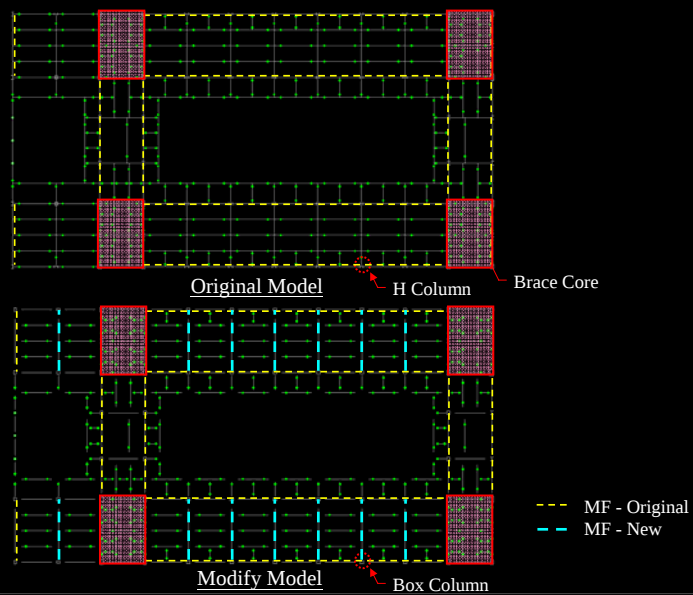
Structure System



Construction

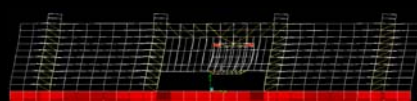


3D Analysis Model

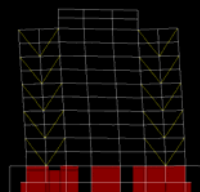


KLC

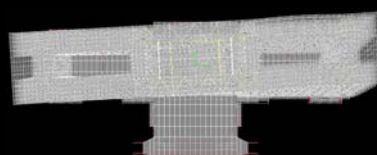
Eigen Value Analysis



Mode-1 (X) :
1.643 sec (Original)
1.600 sec (Modify)



Mode-2 (Y) :
1.301 sec (Original)
1.256 sec (Modify)



Mode-3 (Rz) :
1.296 sec (Original)
1.247 sec (Modify)

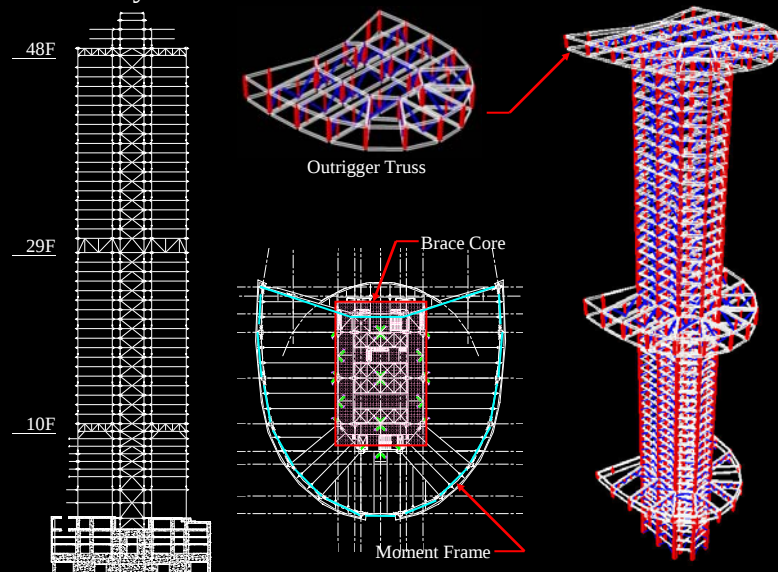
KLC

Far Eastern Ban Chiao Office Tower. Taipei, Taiwan (遠東板橋辦公大樓 - 台灣，板橋)

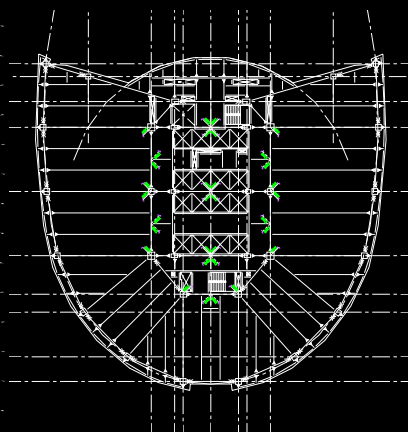


- 50 Storey above grade
- Retail, Parking and Office
- 4 Storey below grade
- Retail, Parking and Services
- Building Height : 202.6m
- Dual Lateral System of Steel Special Moment Resisting Frame with Buckling Restrained Brace (BRB)
- Total Building Area : 108,000m²

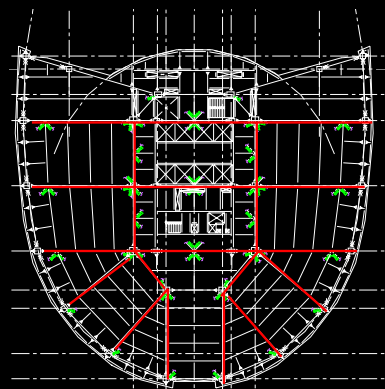
Structure System



Structure Plan

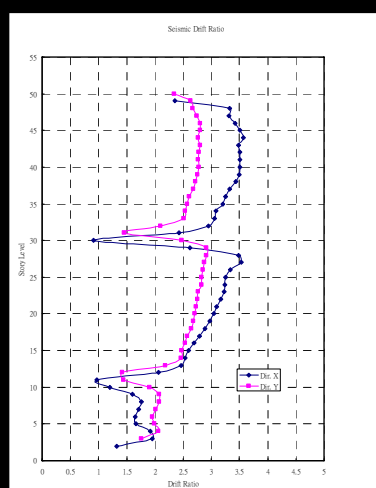


Typical Floor

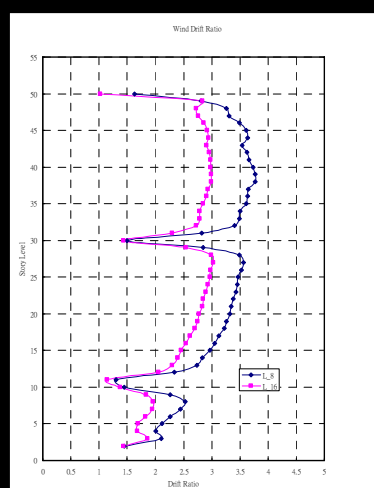


Outtrigger Floor

Structure Analysis

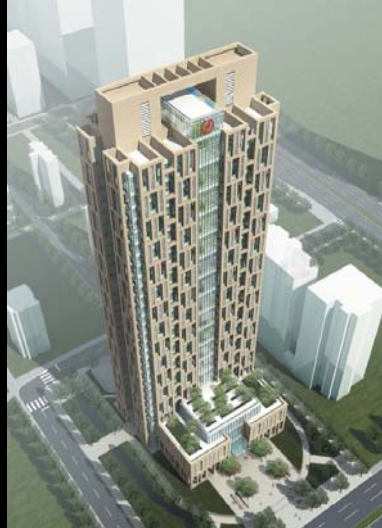


Drift Ratio under Seismic Load



Drift Ratio under Wind Load

Hua Nan Bank Headquarters Building. Taipei, Taiwan (華南銀行總行世貿大樓 - 台灣，台北)

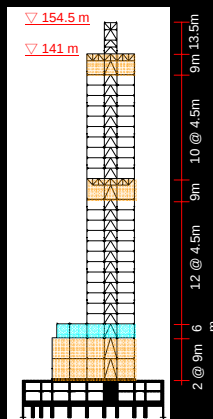


- 27 Storey above grade
- Banking and Office
- 2 Storey below grade
- Parking
- Building Height : 154.5m
- Dual Lateral System of Steel Special Moment Resisting Frame with Buckling Restrained Brace (BRB)
- Total Building Area : 52,000m²

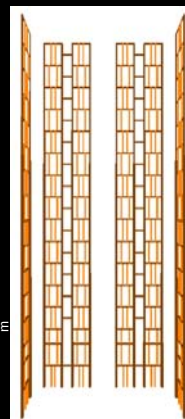
Design Concept



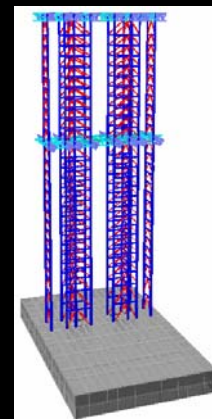
Architect Design Concept



Elevation

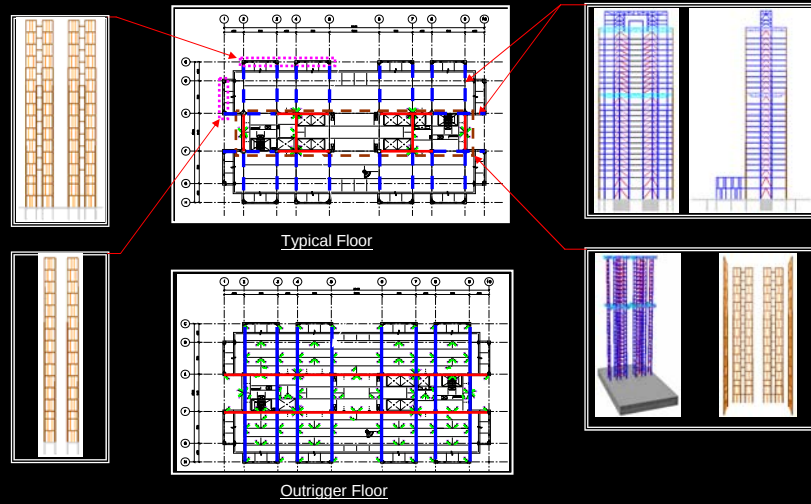


Outer Moment Frame

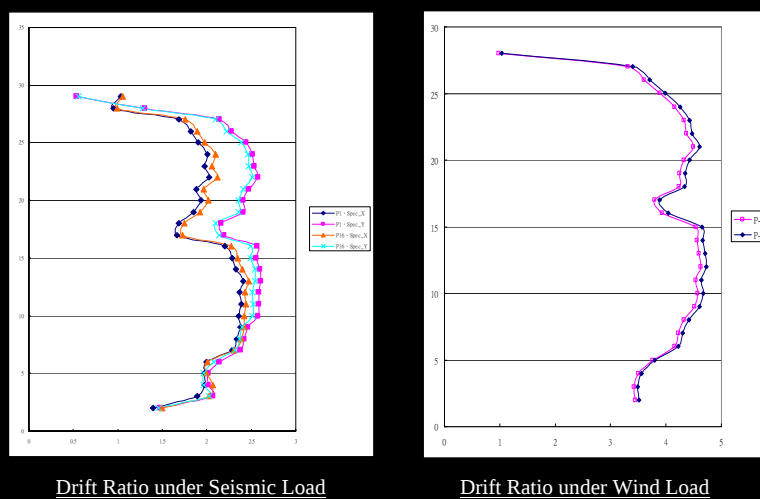


Brace Core & Outrigger Truss

Structure System



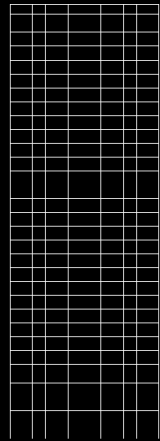
Structure Analysis



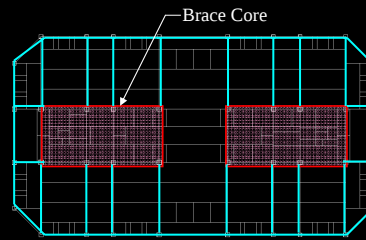
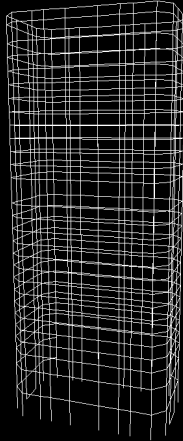
Drift Ratio under Seismic Load

Drift Ratio under Wind Load

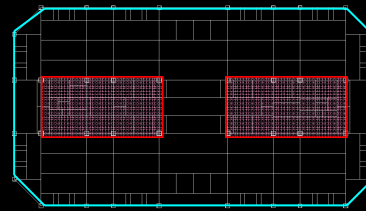
Study Analysis



Outer Moment Frame System

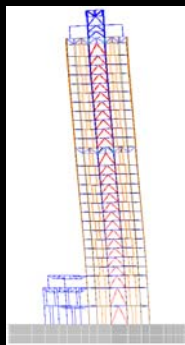


Study Model - 1

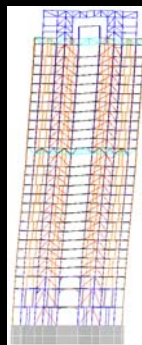


Study Model - 2

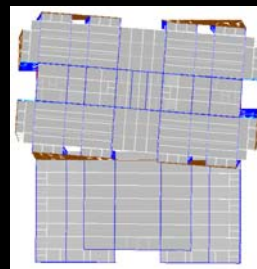
Eigen Value Analysis



Mode-1 (Y) :
3.7 sec (Model-1)
4.1 sec (Model-2)



Mode-2 (X) :
3.48 sec (Model-1)
3.58 sec (Model-2)



Mode-3 (Rz) :
3.18 sec (Model-1)
3.35 sec (Model-2)

Chinese Culture University, Taipei, Taiwan (中國文化大學體育館 - 台灣，台北)



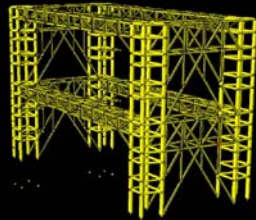
- 8 Storey above grade
- 4 Storey below grade
- Mix-use gymnasium with basketball arena, swimming pool and supporting facilities.
- Total Building Area : 55,000m²

Design Concept

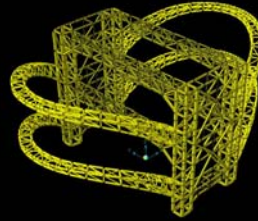


- The elliptical floor plate is 50 meters by 75 meters.
- 4 steel mega columns.
- Brace frames in X direction.
- Mega frames in Y direction.

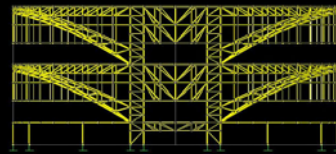
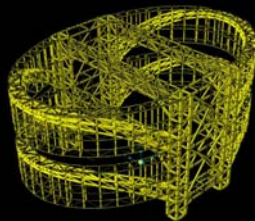
Structure System



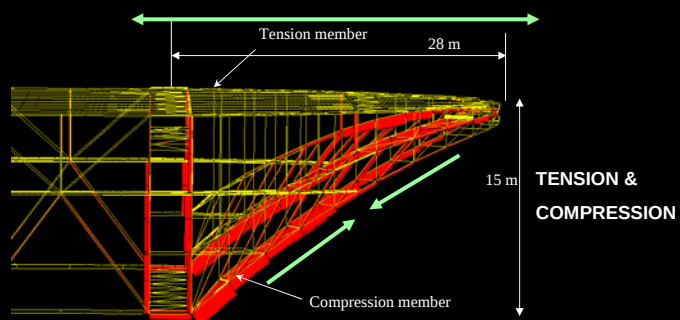
Mega Frame System



Mega Frame & Mega Brace System



Composite Action



TENSION : STEEL BEAMS & SLAB
COMPRESSION : ARCH MEMBERS

Design Criteria

Design Earthquake	Level 1 (Code Design)	Level 2 (Code Design)	Level 3 (Performance)
Qualitative Performance Level	No Damage	Repairable Damage	No Collapse
Gravity Load System including Main Arches and Columns	No Damage	No Damage	No Damage
BRB Devices Performance	Elastic	Ductility $u < 4$	Ductility $u < 8$
Allowable Story Drift Ratio	0.005	0.010	0.015
M.F. Beam Performance	Elastic	Rotation < 0.01 radians	Rotation < 0.15 radians

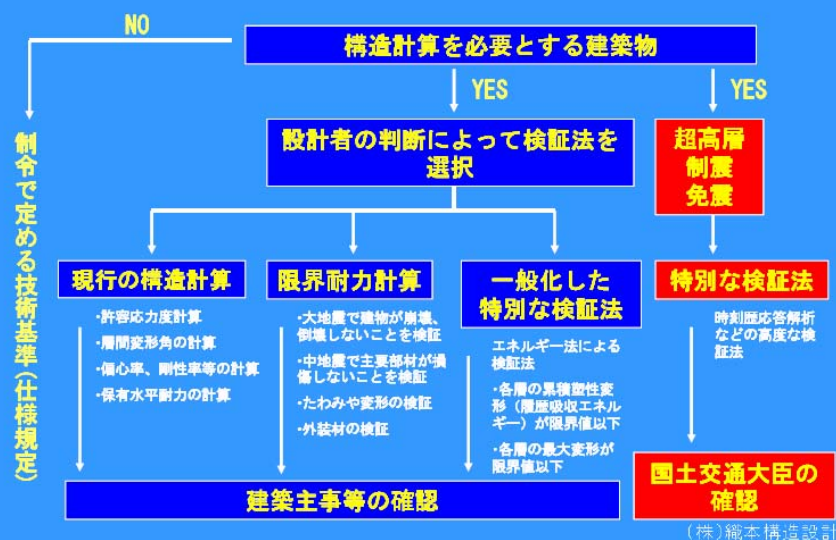
Thank you.

免震・制振技術

(株) 織本構造設計

(株) 織本構造設計

日本における構造設計システム



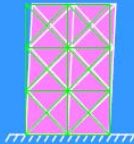
(株) 織本構造設計

大地震に対する耐震設計の考え方



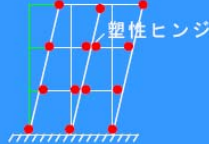
耐震構造

強度抵抗型 [弾性設計法]



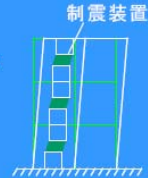
- ・部材断面大
- ・応答せん断力大

履歴減衰型 [塑性設計法]



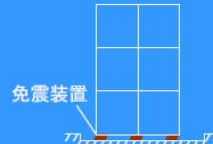
- ・塑性ヒンジによる地震エネルギー吸収
- ・損傷は許容するが人命保護を優先

制震構造



- ・制震装置による地震エネルギー吸収
- ・建築物の機能維持

免震構造



- ・免震装置による地震エネルギー吸収
- ・家具、什器の無損傷

地震に耐える建物
[耐震構造]

揺れを制御する建物
[制震構造]

揺れを吸収する建物
[免震構造]

(株)縮本構造設計

構造設計のための リスクマネジメント



(株)縮本構造設計

台湾を取り巻く主な自然災害



- 地震による災害
(結果として生ずる津波や火事による災害)
- 台風、竜巻による災害
- 大雨による土砂崩れ・洪水等の災害

(株)縮本構造設計

建物の設計に重要な事項



- リスクマネジメントシステムの構築
 - ① PML値(最大予想損害率)
 - ② LCC(ライフサイクルコスト)
 - ③ BCP(事業継続計画)

(株)縮本構造設計

● リスクマネジメント



リスクの想定

自然災害の想定

地震災害

台風災害

水害



耐震性能

台風性能

増水対策

立地条件

- ・地盤条件
- ・地形条件
- ・周辺条件

目標性能



PML値の設定
LCCを考慮
BCPの構築

(株)織本構造設計

○建物の証券化

○建物と保険との関わり方

○建物を一種の証券化としての資産の流動化

○建築主や保険会社の建物への耐震安全性に関する考え方が大きく変わる。

○設計者の耐震安全性に対する考え方も変わる。

○地震リスクを考える場合はその地域の地震ハザードを調査、分析し危険度の確率により、PML値を算出。

①新築の場合は、目標PML値を設定し、それを満足する値となるような設計方法を採用する。

②既存の場合は、目標PML値になるように改修補強する必要がある。



(株)織本構造設計

① P M L 値 (Probable Maximum Loss)

$$\blacksquare \text{ P M L } = \frac{\text{補修費}}{\text{再調達価値}} \times 100 (\%)$$

補 修 費 : 475年に一度の確率で起こりえる最大級地震が生じた建物を地震以前の状態に復旧するのにかかる必要な費用

再調達価値 : 全く同じ建物を建てるのに必要な費用
(新築工事費と同じ意味)

■補修費対策

構造躯体

仕上げ

設備機器

インフラ被害による機器停止による損失

(株)縮本構造設計

P M L (%)	危険度	予想される被害
0~5	極めて低い	軽微な構造体の被害
5~15	低い	局所的な構造体の被害
15~30	中位	中破の可能性が高い
30~60	高い	大破の可能性が高い
60~	非常に高い	倒壊の可能性が高い

(株)縮本構造設計

②LCC (Life Cycle Cost)

建物にかかる生涯コストのこと



建物の企画・設計に始まり、竣工・運用を経て、解体処分するまでを建物の生涯と定義して、その全期間に要する費用を意味する。

今日不動産を証券化する傾向にあるため、LCCの算定は、建物評価に欠かせないデューデリジェンス(建物評価)の判断基準ともなる指標として重要となりつつある。

また、LCCは長期的な費用を意味するため、地震リスクを維持コストに含める考え方になりつつある。

(株)織本構造設計

ライフサイクルコスト

維持コスト



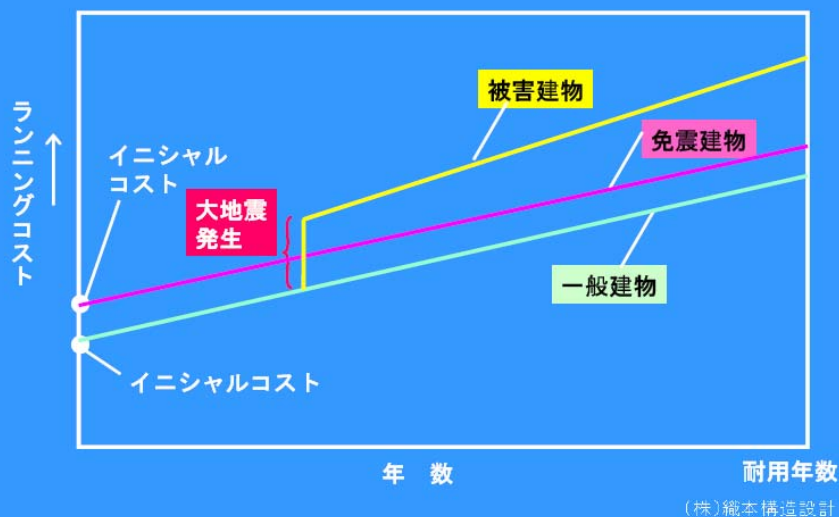
地震リスクの影響も考慮

- 物価変動によりランニングコストが変動する可能性がある。
- 地震リスクも視野に入れる。
- 維持コストは、LCCにおいて取得コストと同程度、又はそれ以上のコストがかかるといわれる。
- 10年、20年といった中期的な視点で、不確定要素も考慮しつつ、ランニングコストを検討しなければならない。

LCCを削減するためには、CM/PMによるVE、地震リスク分析によるPML値の算出と合わせた計画が必要である

(株)織本構造設計

大地震を対象としたLCC



③ BCP (Business Continuous Plan)



- 大きな災害（災難）が発生した場合でも企業としての事業が継続できるような対策
- BCPを対象とする緊急事態としての主な事項
 - (1) 地震
 - (2) 火災
 - (3) 地震以外の自然災害（台風、水害等）
 - (4) 情報システムの故障
 - (5) 顧客情報の漏洩
 - (6) 基幹系システムの利用不可
 - (7) 公共インフラの利用不可
 - (8) 法令違反
 - (9) テロの発生
 - (10) 社員の不祥事

自然災害に弱い免震層



敷地周辺の状況を調査する



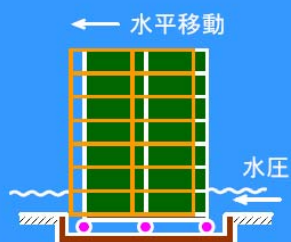
過去の自然災害の歴史を調べる



対処法を考える

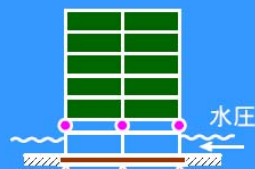
自然災害に対する対処法

津波又は洪水に対する対処法



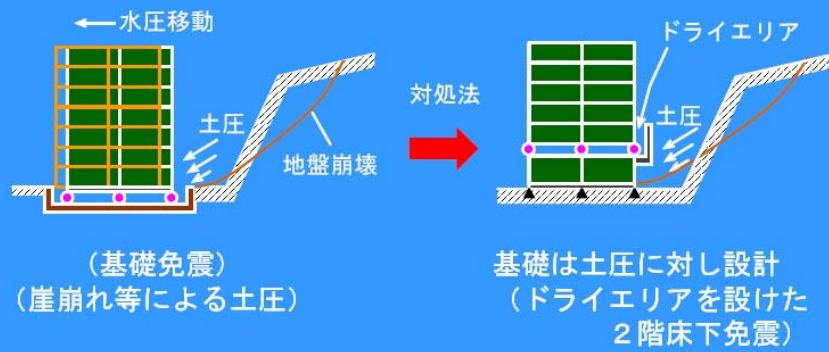
(基礎免震)

対処法



(2階床下中間免震層)

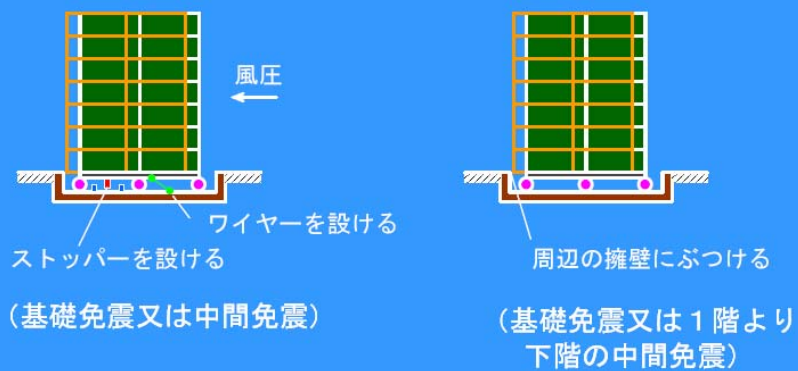
土砂崩れに対する対処法



(株)織本構造設計

台風に対する対処法

地震による応答水平変位 $\leq$ 風による応答水平変位



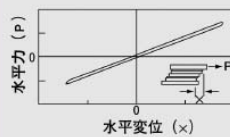
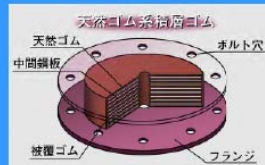
(株)織本構造設計

免震構造

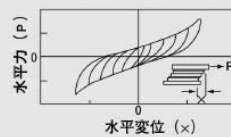
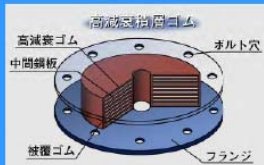
(株) 総本構造設計

免震材料の特性（支承系－１）

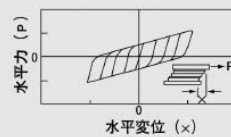
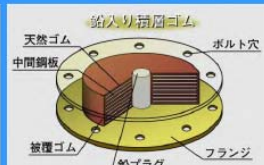
天然ゴム系積層ゴム



高減衰積層ゴム



鉛入り積層ゴム

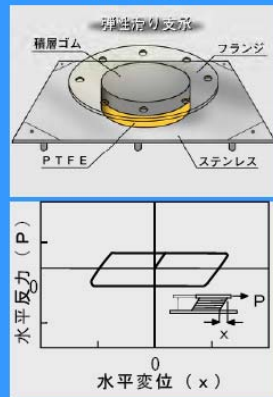


(株) 総本構造設計

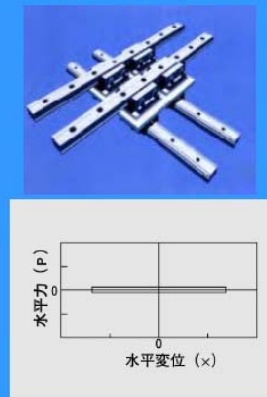
免震材料の特性（支承系－２）



弾性滑り支承



転がりローラー支承



(株)織本構造設計

免震材料の特性（減衰系－１）



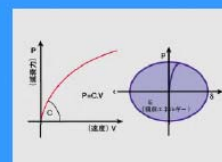
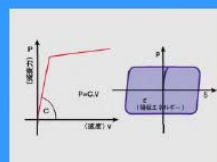
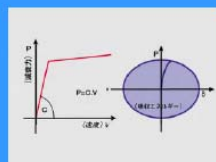
オイルダンパー



軟塑性体ダンパー



減衰こま



(株)織本構造設計

免震材料の特性（減衰系－２）



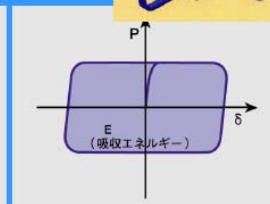
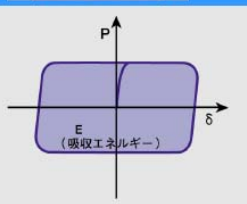
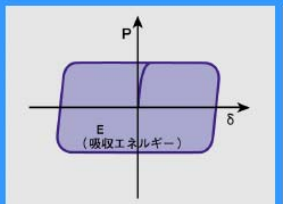
鋼棒ダンパー



鉛ダンパー



U型ダンパー



(株) 縮本構造設計

地盤周期と免震建物の固有周期に対する応答値



同じ免震建物でも地盤周期により応答値が異なる



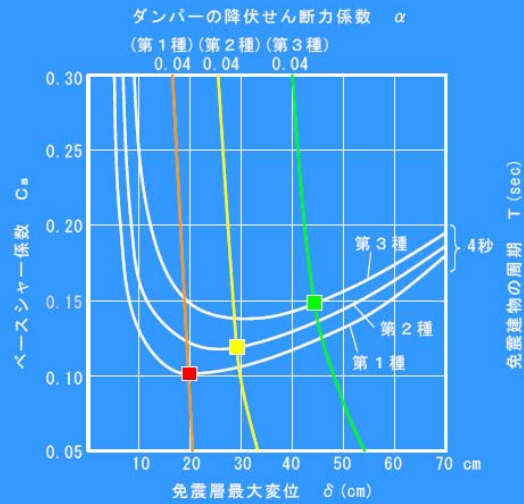
地盤種別に適した免震システムを考える



最適応答値となる免震システム

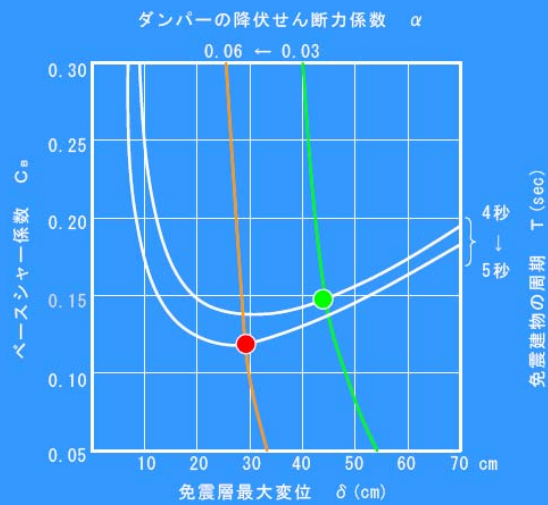
(株) 縮本構造設計

各地盤種別における建物周期と免震装置の降伏せん断力係数と各応答値との関係



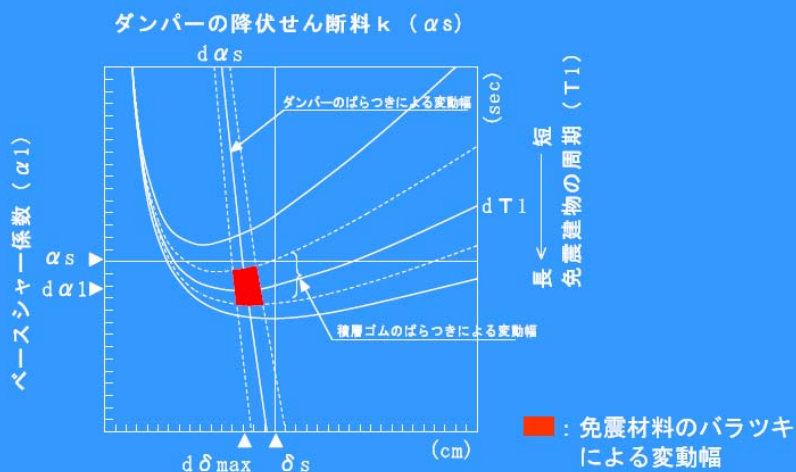
(株)縮本構造設計

第3種地盤における建物周期と免震装置の降伏せん断力係数と各応答値との関係



(株)縮本構造設計

免震材料のバラツキによる変動値



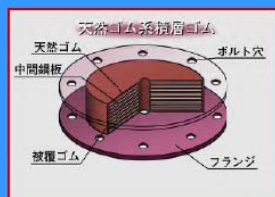
(株) 綿本構造設計

免震材料を組み合わせた免震システム



① 天然ゴム＋鉛ダンパー＋鋼棒ダンパー (U型ダンパー)

天然ゴム系積層ゴム



鉛ダンパー



鋼棒ダンパー



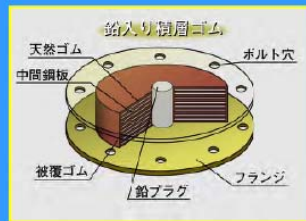
U型ダンパー



(株) 綿本構造設計

② 鉛入り積層ゴム+弾性すべり+粘性ダンパー

鉛入り積層ゴム



弾性滑り支承



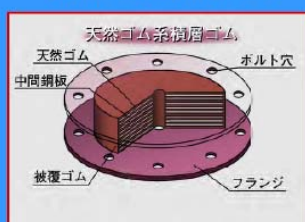
粘性ダンパー



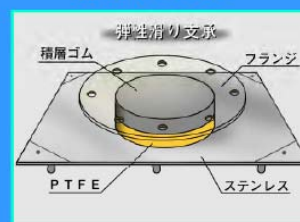
(株) 綱本構造設計

③ 天然ゴム+弾性すべり+粘性ダンパー

天然ゴム系積層ゴム



弾性滑り支承



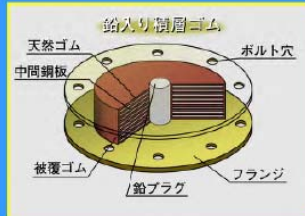
粘性ダンパー



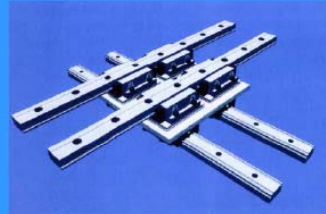
(株) 綱本構造設計

④ 鉛入り積層ゴム＋転がり＋粘性ダンパー

鉛入り積層ゴム



転がりローラー支承

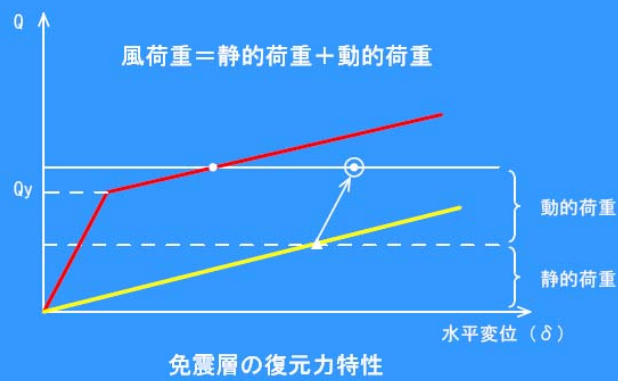


粘性ダンパー



(株)総本構造設計

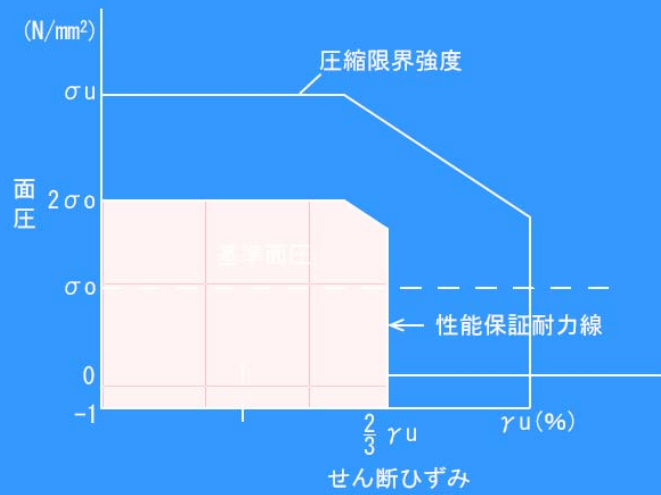
強風に対する免震層の残留変形



注) 鉛ダンパーや粘性ダンパーは風の動的荷重には寄与するが、静的荷重に寄与しない。

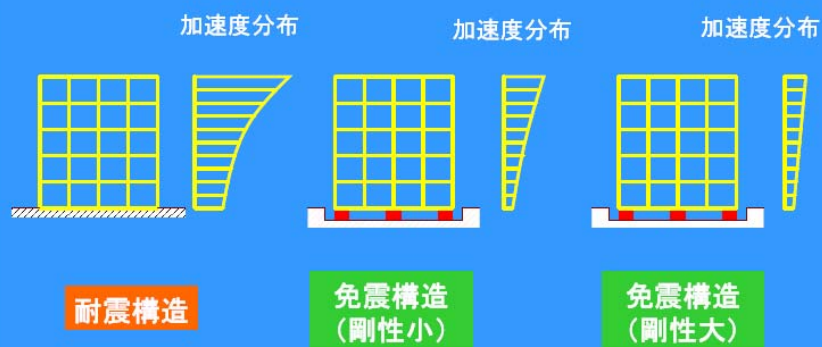
(株)総本構造設計

積層ゴム支承の面圧－せん断ひずみ



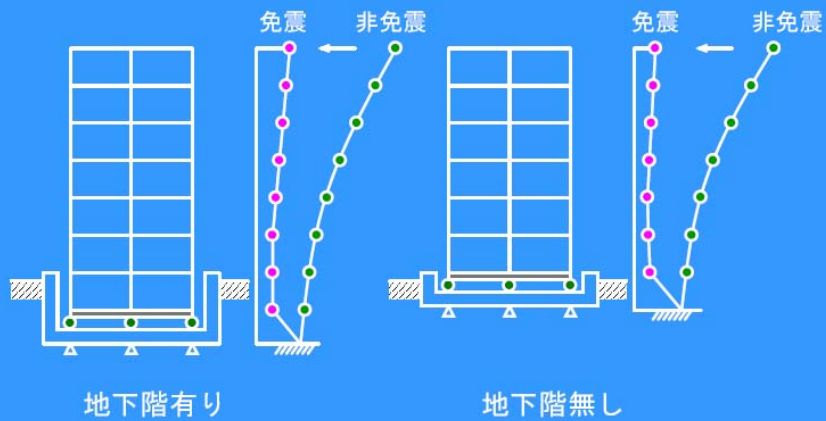
(株) 総本構造設計

従来の耐震構造と 免震構造の地震力の比較



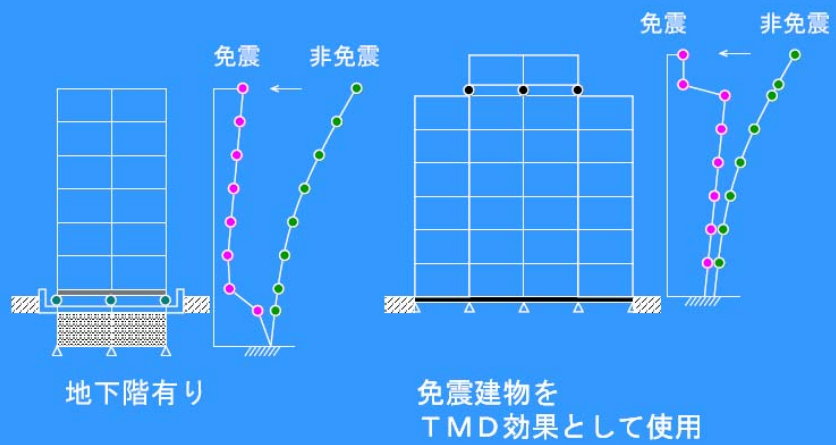
(株) 総本構造設計

免震層の位置と応答加速度（基礎免震）



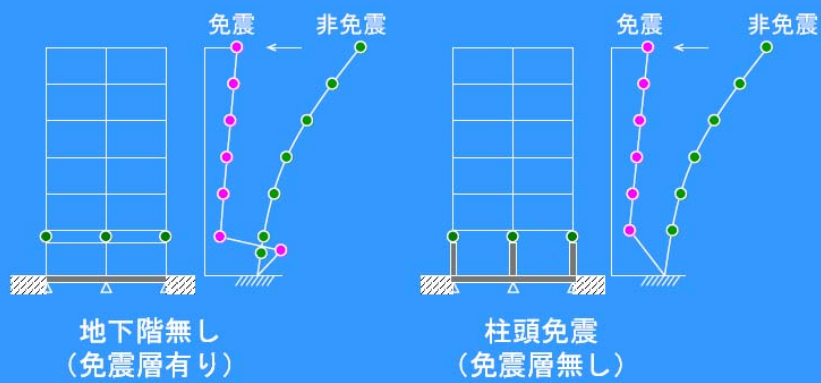
(株) 縮本構造設計

中間免震（１）



(株) 縮本構造設計

中間免震（２）



(株)綿本構造設計

人工地盤と免震層



(株)綿本構造設計

ファーストプレイス横浜



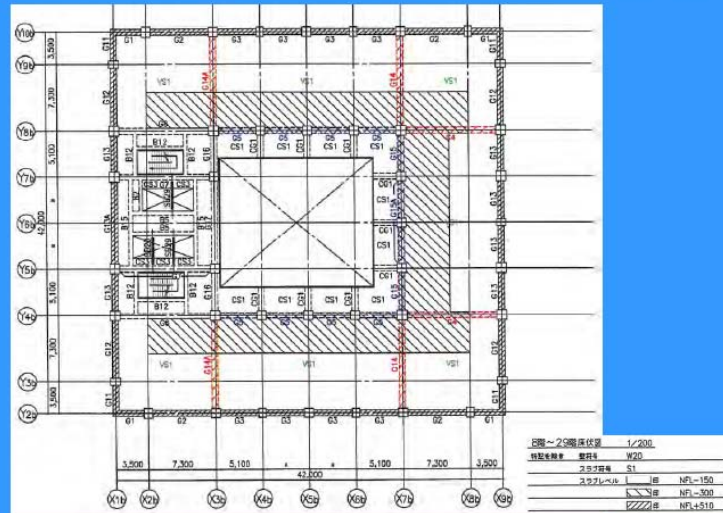
(株)綿本構造設計



制振構造

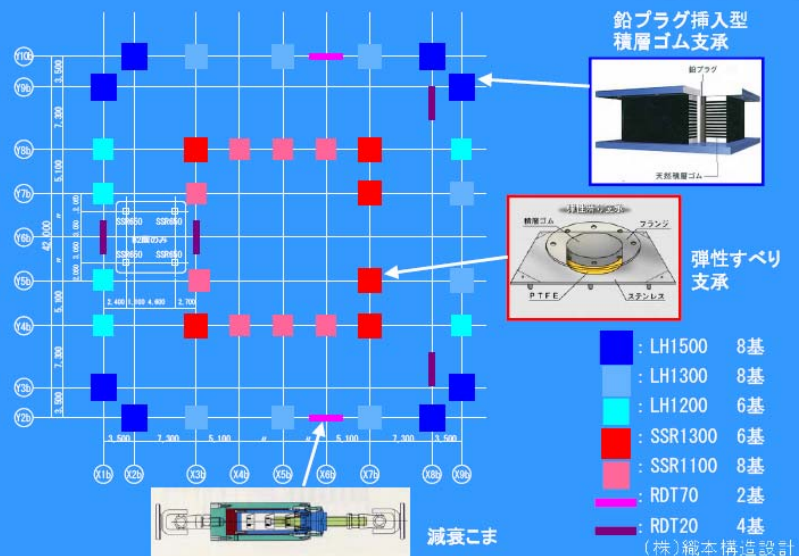
(株)綿本構造設計

住宅階基準図



(株) 総本構造設計

免震装置配置図



3～6階制振壁配置図

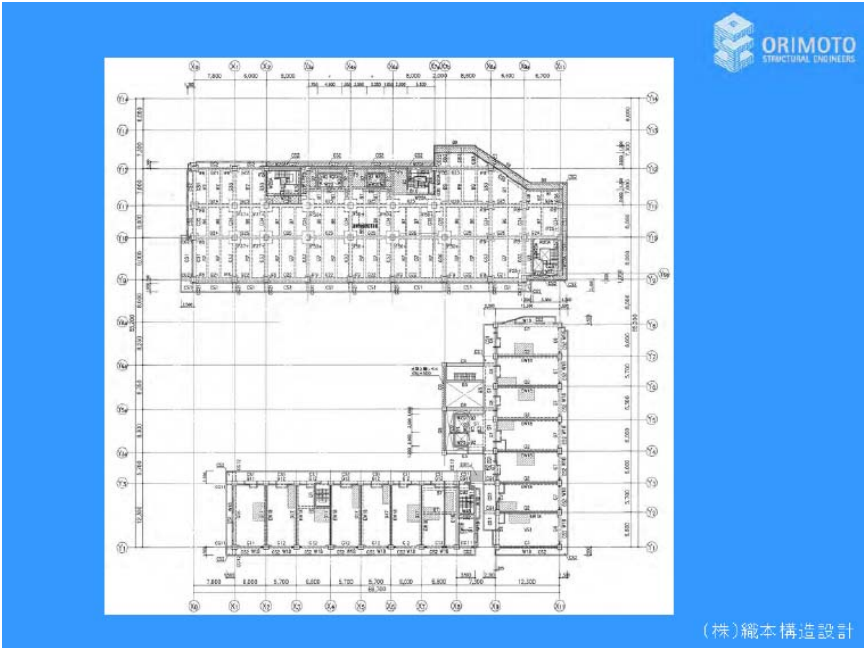


(株)織本構造設計

ルリエ川崎

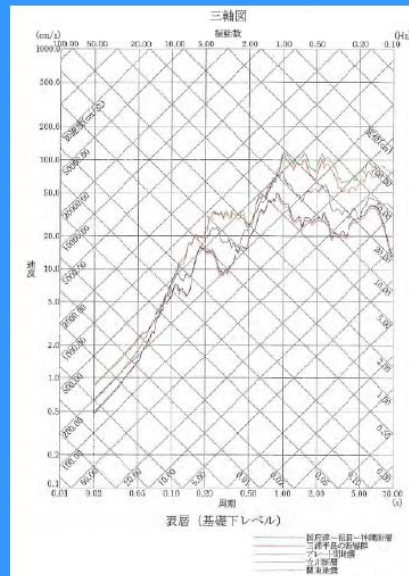


(株)織本構造設計

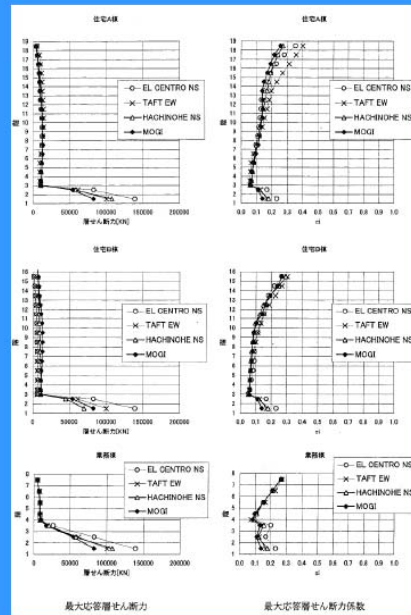




(株) 織本構造設計

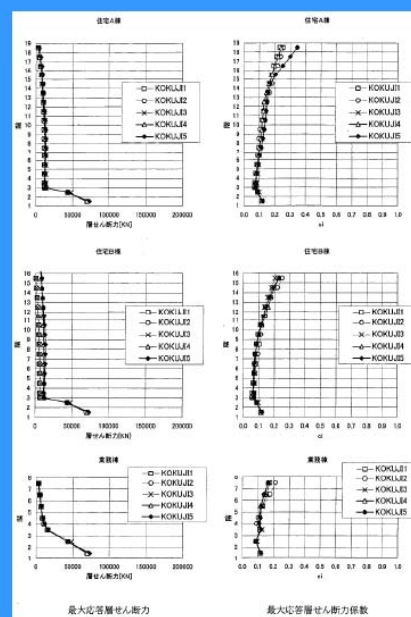


(株) 織本構造設計



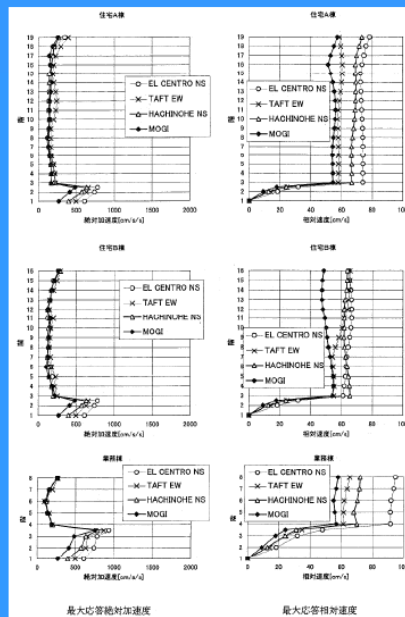
X方向
最大応答層せん断力
最大応答層せん断力係数

(株) 総本構造設計



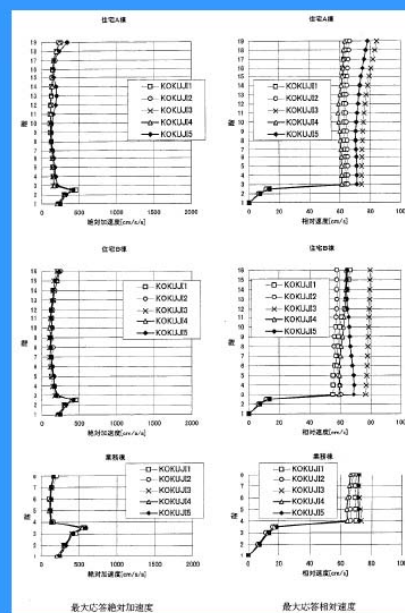
X方向
最大応答層せん断力
最大応答層せん断力係数
(告示5波)

(株) 総本構造設計



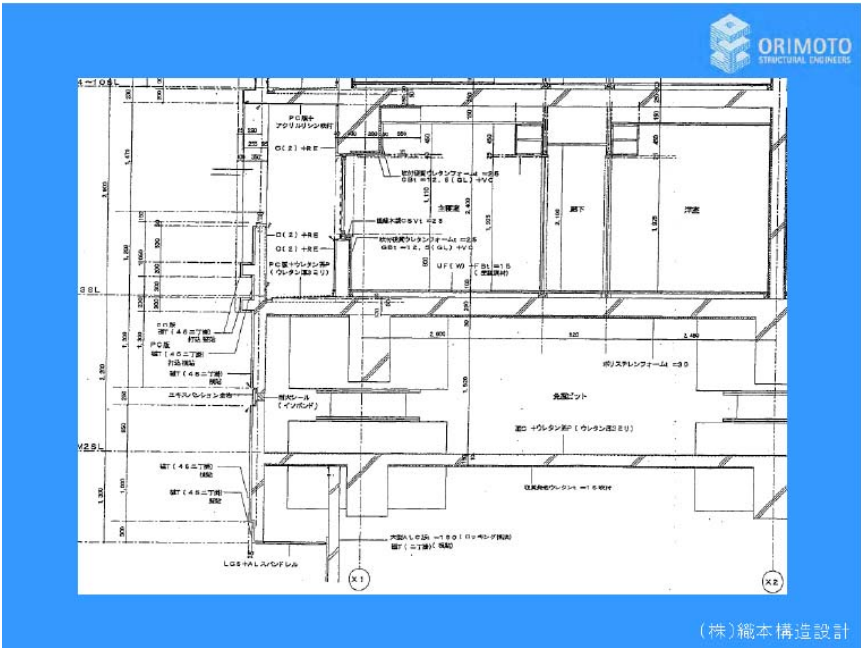
X方向
最大応答絶対加速度
最大応答相対速度

(株)綿本構造設計



X方向
最大応答絶対加速度
最大応答相対速度
(告示5波)

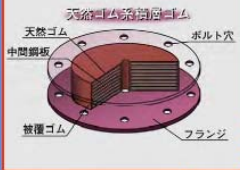
(株)綿本構造設計





ORIMOTO
STRUCTURAL ENGINEERS

天然ゴム系
積層ゴム

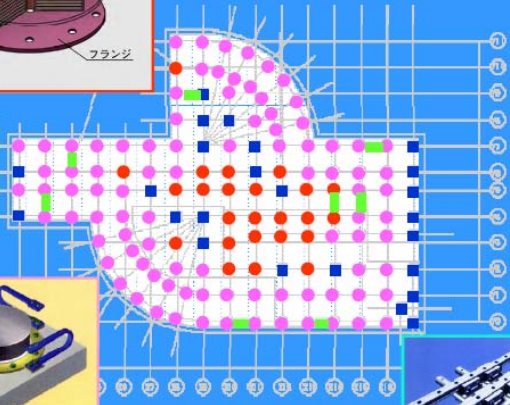




オイルダンパー



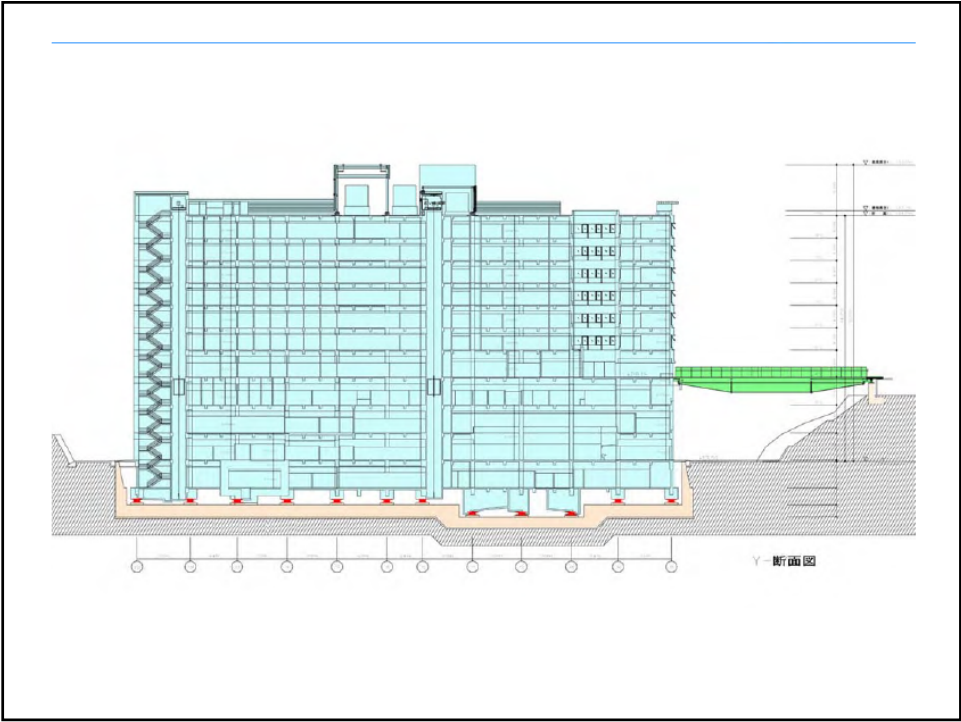
U型ダンパー
一体積層ゴム



CLB



(株) 縄本構造設計



制振構造

(株) 総本構造設計

制振ダンパーの分類

制
震
ダンパー

履歴減衰付加型

- 鋼材(弾塑性)ダンパー
- 摩擦ダンパー
- 鉛ダンパー

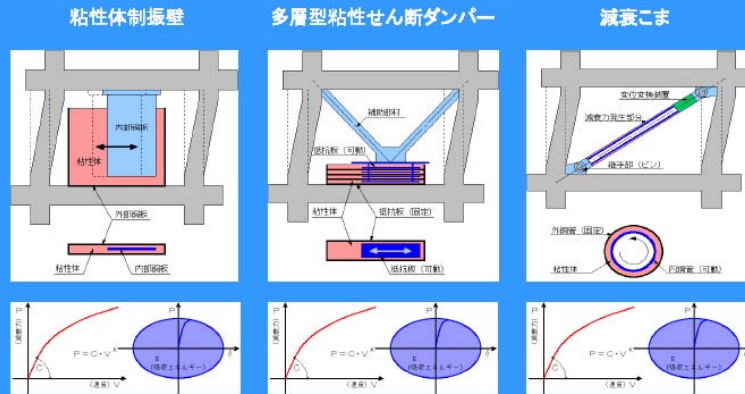
粘性減衰付加型

- オイルダンパー
- 粘性体ダンパー
- 粘弾性体ダンパー

建物の構造種別，形状などに応じて，
最適なダンパー選定と配置を行う

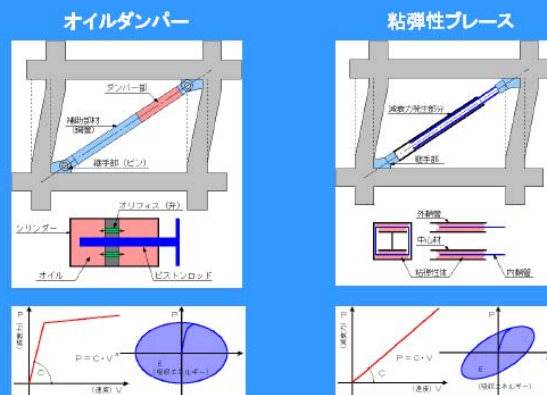
(株) 総本構造設計

制振装置いろいろ（速度依存系 1）



(株) 織本構造設計

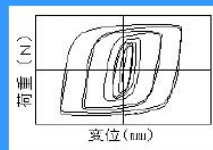
制振装置いろいろ（速度依存系 2）



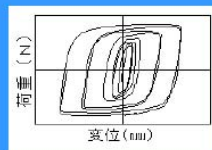
(株) 織本構造設計

制振装置いろいろ（歪み依存系 1）

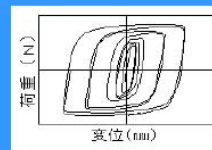
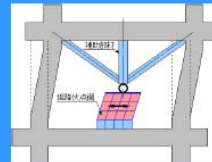
低降伏点鋼間柱(1)



低降伏点鋼間柱(2)



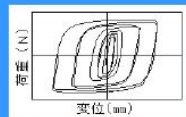
低降伏点鋼間柱(3)



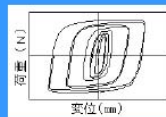
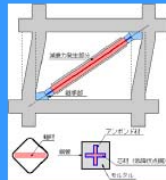
(株)縮本構造設計

制振装置いろいろ（歪み依存系 2）

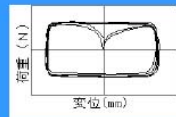
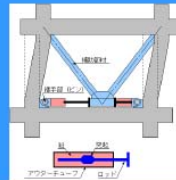
低降伏点鋼パネル



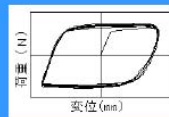
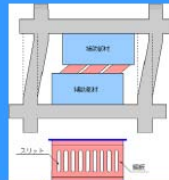
アンボンドブレース



鉛押し出し型ダンパー



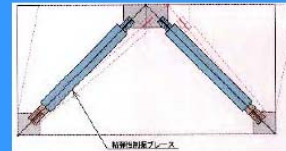
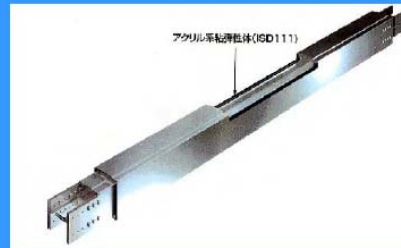
鋼製スリットダンパー



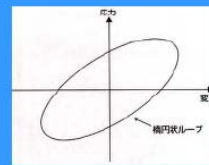
(株)縮本構造設計

粘弾性ダンパー

<粘弾性制振ブレース>



柱梁構面内ブレース

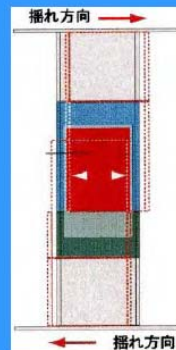


履歴特性: 楕円ループ

(株) 織本構造設計

粘弾性ダンパー

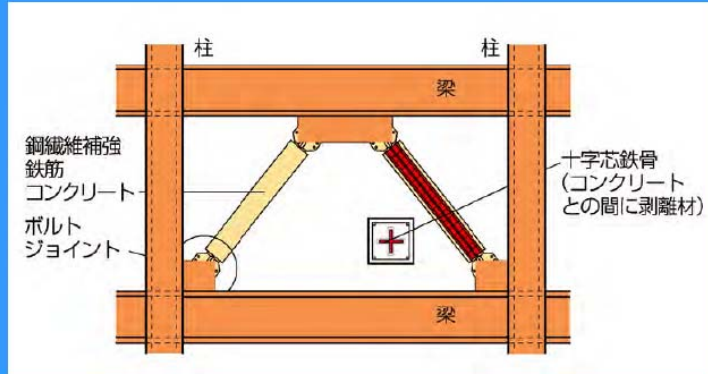
<パネル型粘弾性体ダンパー>



VEMダンパー納まり

(株) 織本構造設計

アンボンドブレースダンパー



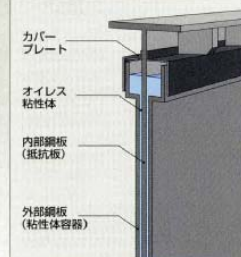
(株) 綱本構造設計

粘性ダンパー（粘性壁）

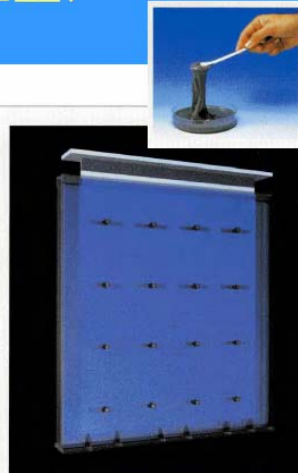
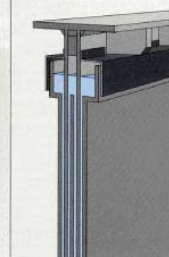
■制震壁の構造と働き

高粘度の粘性体の粘性せん断抵抗力を利用した制振装置で、粘性体を充填した外部鋼板（粘性体容器）の間に、内部鋼板（抵抗板）を挿入したものです。

シングルタイプ



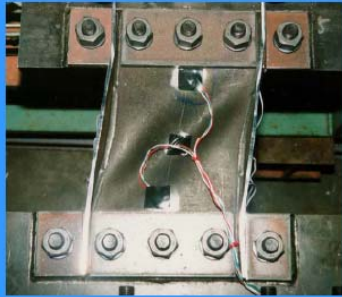
ダブルタイプ



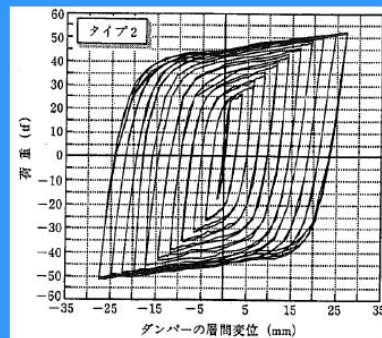
(株) 綱本構造設計

履歴ダンパー

(せん断降伏型)



鋼材の変形による
エネルギー吸収を利用



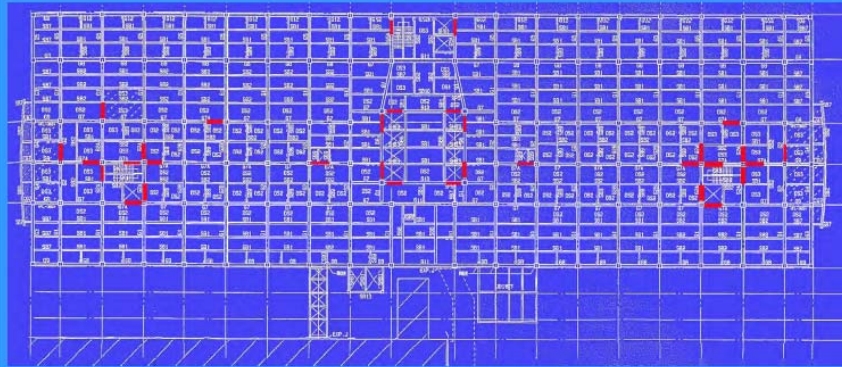
荷重－変位曲線

(株)綿本構造設計

東北大学医学部付属病院病棟

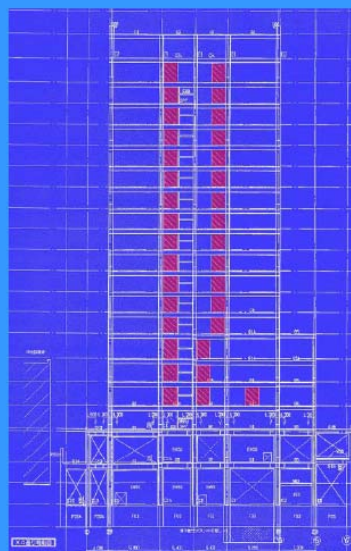


(株)綿本構造設計



基準階伏図

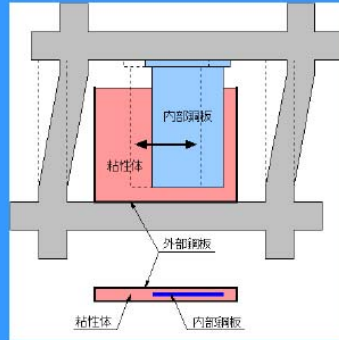
(株)縮本構造設計



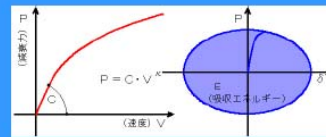
軸組図

(株)縮本構造設計

採用した制震装置

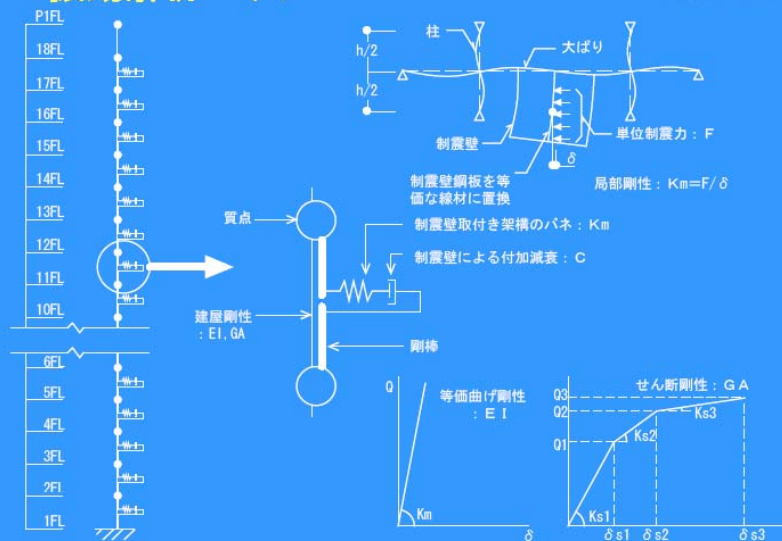


粘性体制振壁



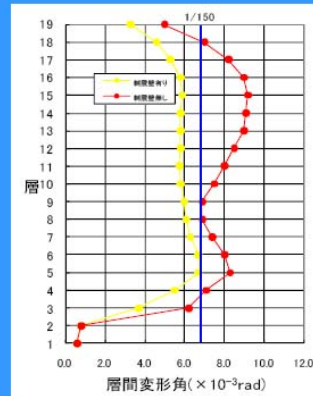
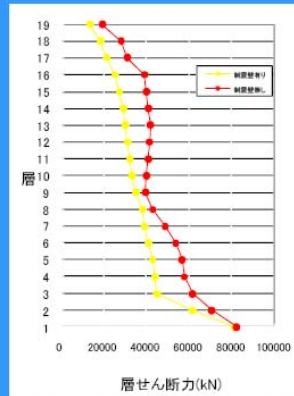
(株) 総本構造設計

振動解析モデル



(株) 総本構造設計

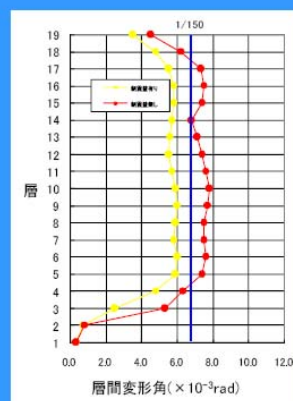
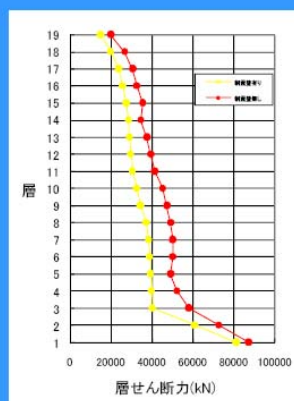
応答比較



EL CENTRO NS
レベル2最大応答図

(株)綿本構造設計

応答比較



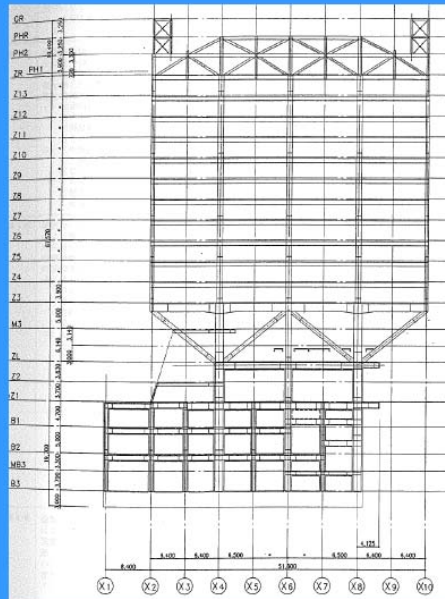
TAFT EW
レベル2最大応答図

(株)綿本構造設計

J R東海ビル（東京）



(株)縮本構造設計



Y 4 通軸組図

(株)縮本構造設計

中間層隔震結構之研究與應用

張國鎮教授

2009.10.09

鋼結構與隔減震技術研討會



National Taiwan University

National Center for Research on Earthquake Engineering

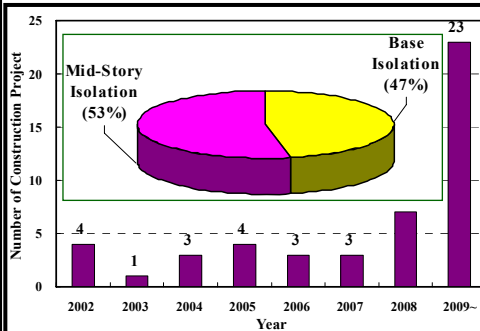


Outline

- Introduction
- Motivation
- Analytical Study
- Experimental Study
- Following Experimental Schemes
- Conclusions

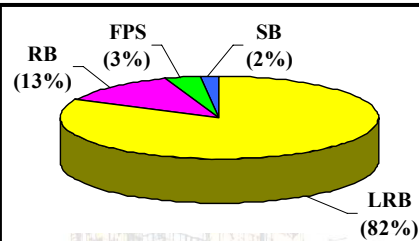
Statistics of Seismically Isolated Buildings in Taiwan (I)

- Applications of seismic isolation design in Taiwan have been extensive after 1999 Chi-Chi earthquake
- The latest statistics indicates that about 50 construction projects adopting seismic isolation systems in Taiwan



Number of Construction Project

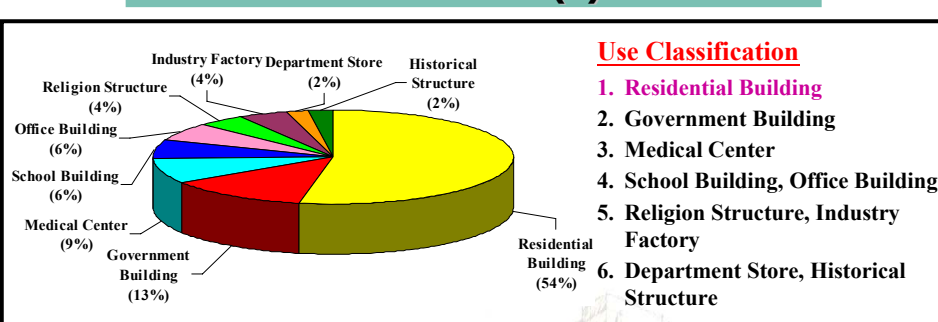
- 48 construction projects



Commonly Used Seismic Isolator

- Lead Rubber Bearing (LRB)
- Friction Pendulum System (FPS)
- Rubber Bearing (RB)
- Sliding Bearing (SB)

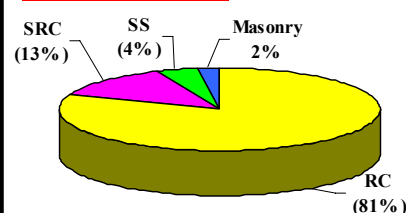
Statistics of Seismically Isolated Buildings in Taiwan (II)



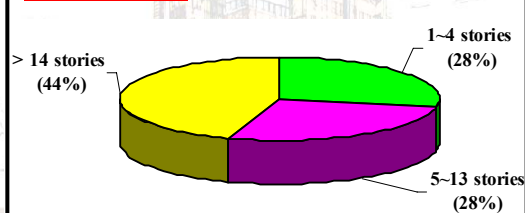
Use Classification

- Residential Building
- Government Building
- Medical Center
- School Building, Office Building
- Religion Structure, Industry Factory
- Department Store, Historical Structure

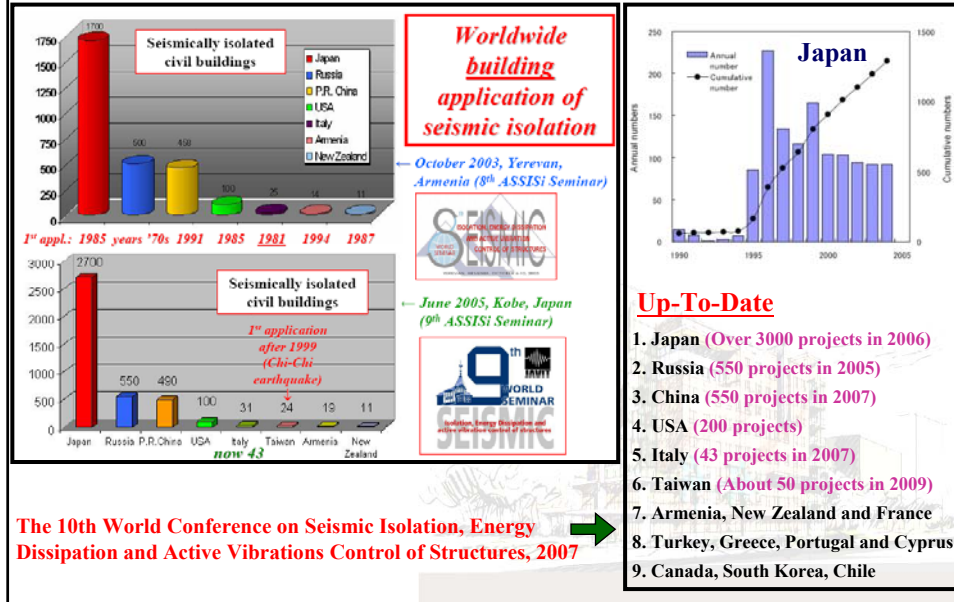
Structural System



Story Number



Statistics of Seismically Isolated Buildings Worldwide



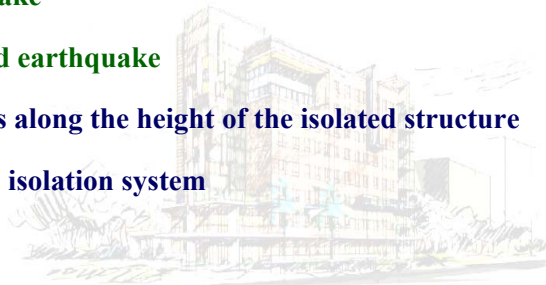
Domestic Building Seismic Design Code

- Design guidelines for seismically isolated buildings (1997)
- First official seismic isolation design code for buildings (2002)
- Current seismic design code for buildings (2005)
 - Ch9 — Structures with seismic isolation systems
 - Ch10 — Structures with passive energy dissipation devices
 - Drawn similar to FEMA 273 and FEMA 356
 - Analysis procedures and test requirements
- Under revision and update of the current seismic design code

Further Refinement of Design Code (I)

Structures with Seismic Isolation Systems

- Different design requirements - Different structural performances
 - Structural elements other than seismic isolation devices remain elastic
 - Design basis earthquake
 - Maximum considered earthquake
- Distribution of seismic forces along the height of the isolated structure
- Fire resistance rating for the isolation system

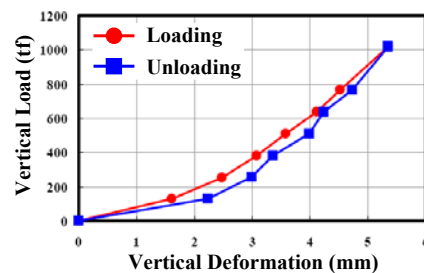


Further Refinement of Design Code (II)

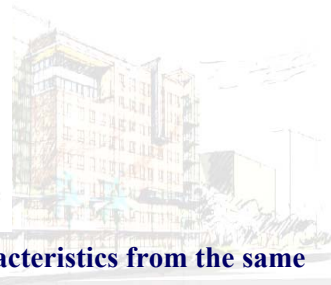
Structures with Seismic Isolation Systems

- Test protocols of prototype test
 - Vertical Load - Lateral Displacement
 - Vertical Stress - Lateral Strain
- Vertical loading test requirements of prototype test

“免震部材標準品リスト”
The Japan Society of Seismic
Isolation (JSSI), 2005



- Bearings with similar dimensions and characteristics from the same manufacturers have been tested in past five years



Applications of Mid-Story Isolation Design in Taiwan

- The isolation system is designated at the top of the **first story** of the building



Design Practices for Mid-Story Isolated Buildings

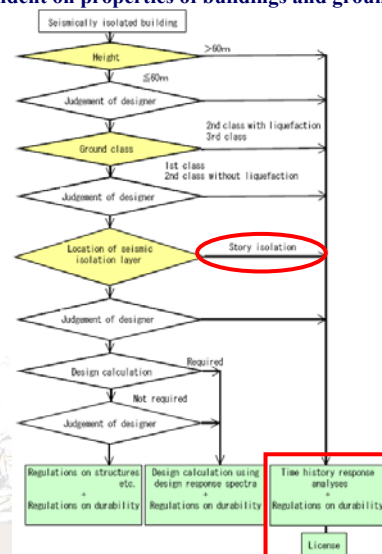
Taiwan

- Comply with the current design guidelines for superstructure
- Substructure generates very little interference
- Nonlinear dynamic response history analyses
- Design review by independent committees

Japan

- Nonlinear dynamic response history analyses
- Approved license

Classification of design methods in Japan (dependent on properties of buildings and grounds)



Motivation of This Study

■ Mid-Story Isolated Building

- The isolation system is incorporated into the mid-story rather than the base of the building, often at the top of the first two stories in Taiwan

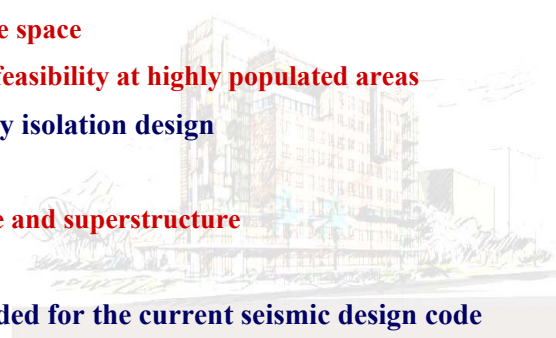
■ As an alternative to base isolation design, mid-story isolation design can

- Satisfy architectural concerns of aesthetics and functionality
- Facilitate the construction in site
- Utilize the limited available space
- Enhance the construction feasibility at highly populated areas

■ Major concerns for mid-story isolation design

- Flexibility of substructure
- Interaction of substructure and superstructure
- Higher mode participation

■ Further refinements are needed for the current seismic design code



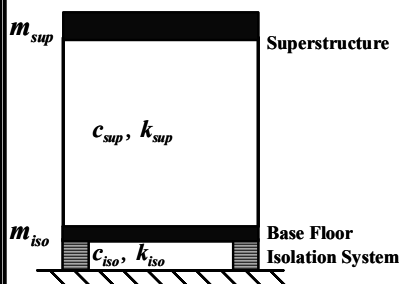
Simplified Structural Model (I)

■ Equivalent Linear Analysis

- All structural elements except isolation system remain elastic
- Isolation System → Equivalent Linear Model

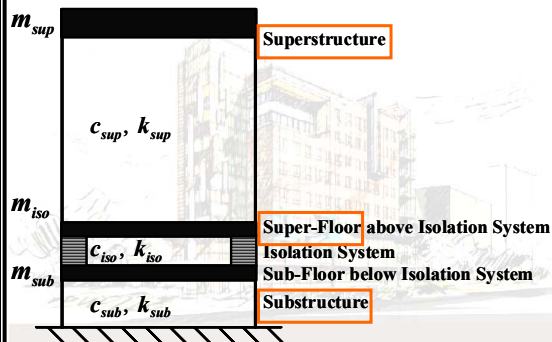
■ Base Isolation

Simplified 2-Lumped-Mass Structural Model (Kelly)



■ Mid-Story Isolation

Simplified 3-Lumped-Mass Structural Model (Proposed)



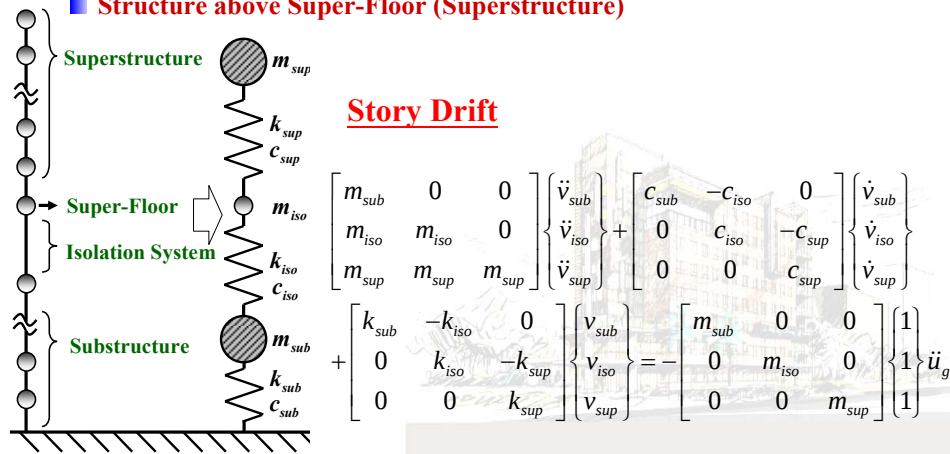
Simplified Structural Model (II)

■ Simplified 3-Lumped-Mass Structural Model

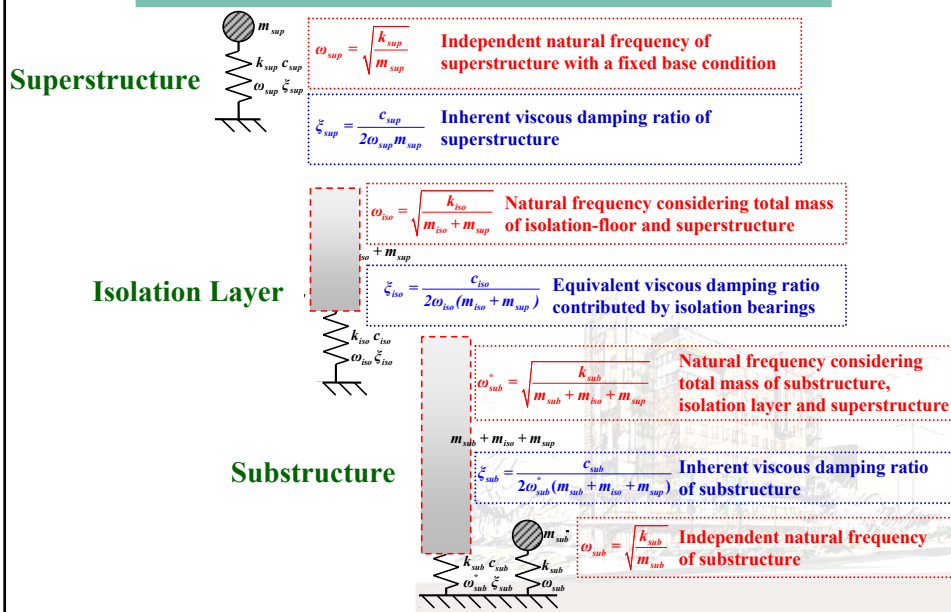
■ Structure below isolation system (Substructure)

■ Floor above isolation system (Isolation-Floor)

■ Structure above Super-Floor (Superstructure)



Nominal Frequencies & Component Damping Ratios



Modal Analysis

- Definitions of mass ratios: $r_{sub} = \frac{m_{sub}}{m_{iso}}$ and $r_{sup} = \frac{m_{sup}}{m_{iso}}$
- Eigenvalue problem → Characteristic equation: $\lambda^3 + a\lambda^2 + b\lambda + c = 0$
- Assuming $\omega_1 \approx \omega_{iso}$
- Neglecting terms $\omega_{iso}/\omega_{sub}$ and $\omega_{iso}/\omega_{sup}$ with power of higher than 2

First Modal Composite Damping Ratio

$$\frac{\xi_{iso}}{\left(1 + \frac{2(1+r_{sup})}{r_{sub}} \left(\frac{\omega_{iso}}{\omega_{sub}}\right)^2 + \frac{2r_{sup}}{1+r_{sup}} \left(\frac{\omega_{iso}}{\omega_{sup}}\right)^2\right)}$$

First Modal Participation Mass Ratio

$$\frac{r_{sub} + 2(r_{sub} + r_{sup} + 1) \left(\frac{\omega_{iso}}{\omega_{sub}}\right)^2 + \frac{2r_{sub}r_{sup}}{1+r_{sup}} \left(\frac{\omega_{iso}}{\omega_{sup}}\right)^2}{(1+r_{sub}+r_{sup}) \left(\frac{r_{sub}}{1+r_{sup}} + 2 \left(\frac{\omega_{iso}}{\omega_{sub}}\right)^2 + \frac{2r_{sub}r_{sup}}{(1+r_{sup})^2} \left(\frac{\omega_{iso}}{\omega_{sup}}\right)^2\right)}$$

Parametric Study

■ $r_{sub} = \frac{m_{sub}}{m_{iso}} = 1, r_{sup} = \frac{m_{sup}}{m_{iso}} = 5$



To discuss effect of r_{sub}

$r_{sub} = \frac{m_{sub}}{m_{iso}} = 2, r_{sup} = \frac{m_{sup}}{m_{iso}} = 5$

$r_{sub} > r_{sup}$



Mass absorber or building mass damper (excluded)

■ $\omega_{sub}/\omega_{iso} = 3 \sim 40$ and $\omega_{sup}/\omega_{iso} = 3 \sim 40$

■ $\xi_{sub} = \xi_{sup} = 0.05$ and $\xi_{iso} = 0.2$

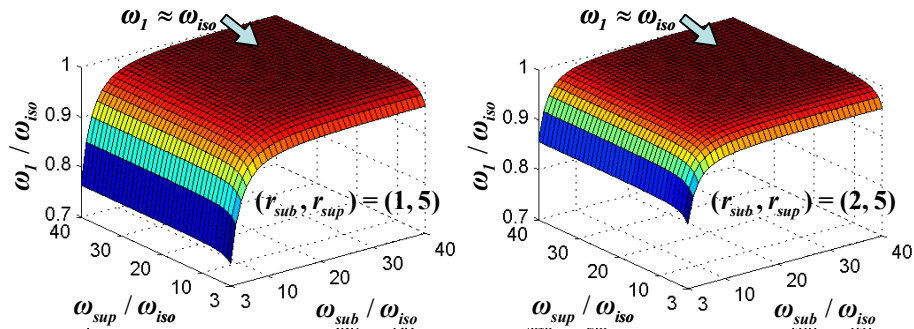
■ Effective period of isolation system is 2sec ($\omega_{iso} = \pi$)

■ $S_{DS} = 0.8$ and $S_{DI} = 0.4 \rightarrow$ Response Spectrum Analysis

■ Complete Quadratic Combination (CQC) method

First Modal Natural Frequency

- Comparison between fundamental modal natural frequency and nominal frequency of isolation system

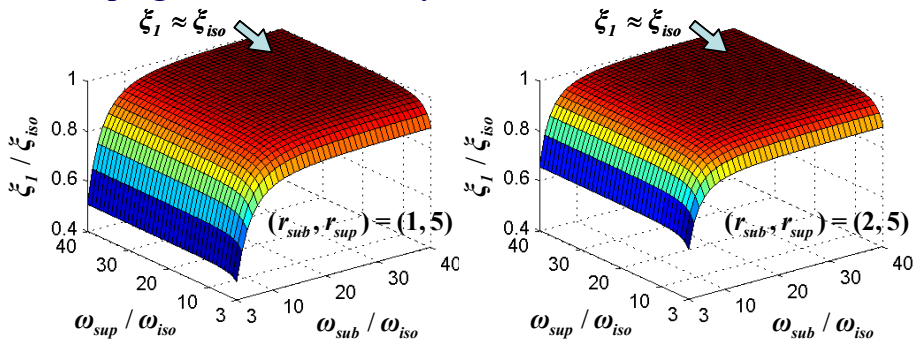


Both nominal frequencies of substructure and superstructure are much higher than nominal frequency of isolation system

→ $\omega_1 \approx \omega_{iso}$

First Modal Composite Damping Ratio

- Comparison between first modal damping ratio and equivalent damping ratio of isolation system



$\omega_{sub}/\omega_{iso} \uparrow$

$\omega_{sup}/\omega_{iso} \uparrow$

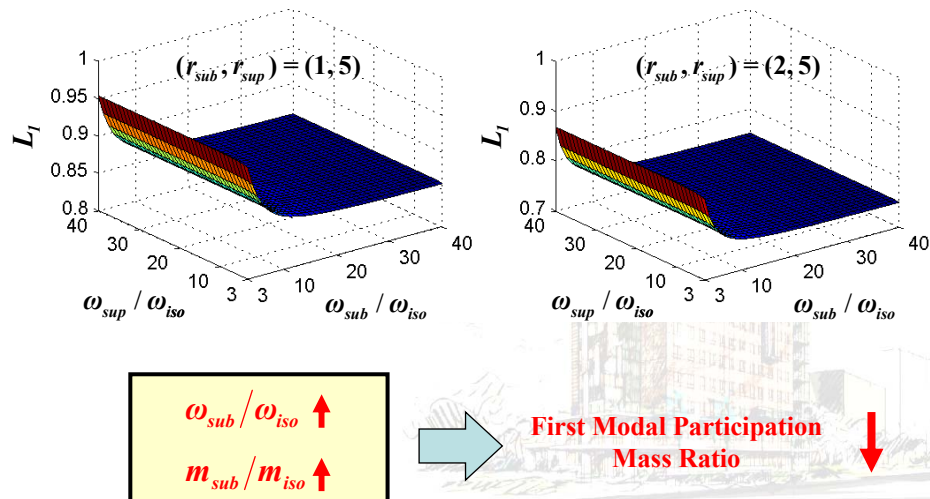
$m_{sub}/m_{iso} \uparrow$

First Modal
Damping Ratio

→ $\xi_1 \approx \xi_{iso}$

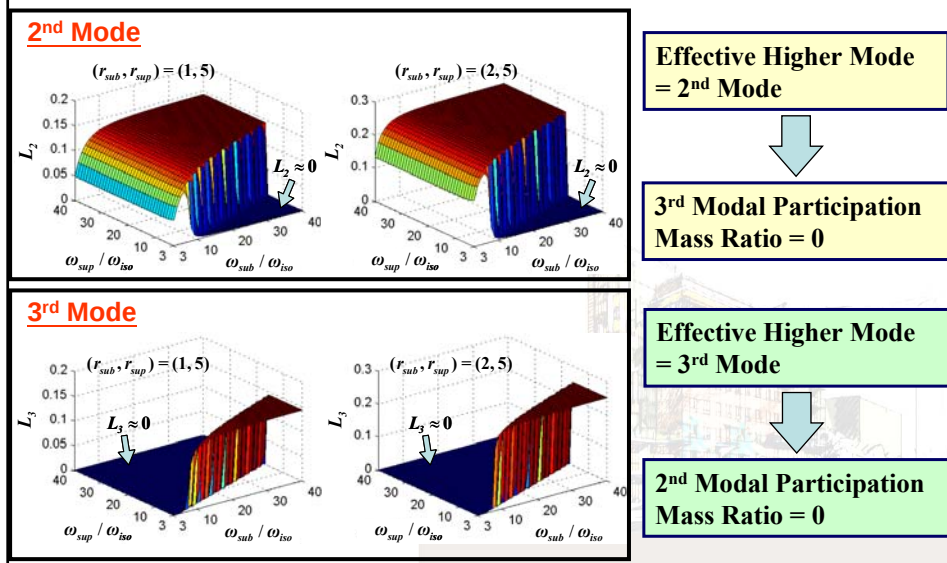
First Modal Participation Mass Ratio

Variations of fundamental modal participation mass ratio



Higher Modal Participation Mass Ratios

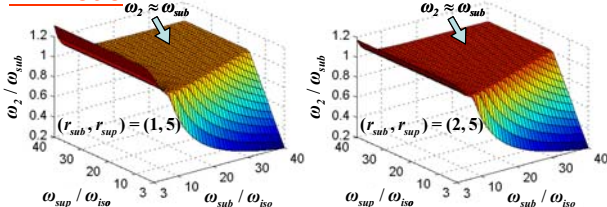
Variations of higher modal participation mass ratios



Higher Modal Natural Frequencies

■ Comparison between higher modal natural frequencies and nominal frequency of substructure

2nd Mode

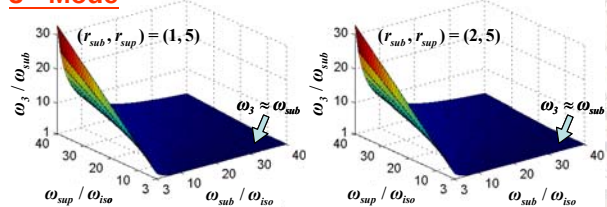


Effective Higher Mode
= 2nd Mode



2nd Modal Natural
Frequency = ω_{sub}

3rd Mode



Effective Higher Mode
= 3rd Mode

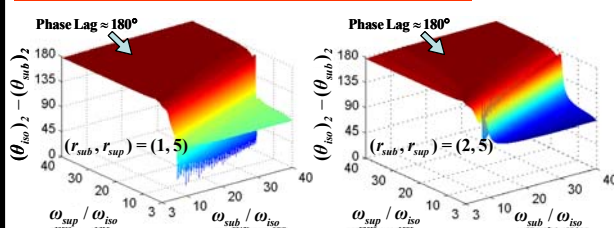


3rd Modal Natural
Frequency = ω_{sub}

Phase Lags of Higher modes (I)

■ Phase lag in 2nd mode shape

Super-Floor relative to Substructure

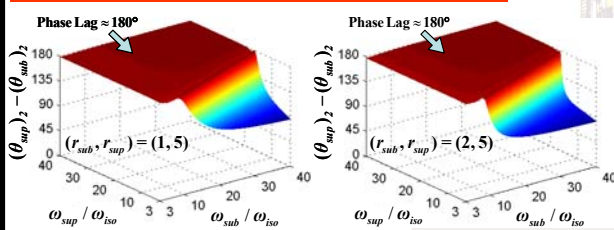


Effective Higher Mode
= 2nd Mode



Phase Lag = 180 degrees
(Super-Floor—Substructure)
(Superstructure—Substructure)

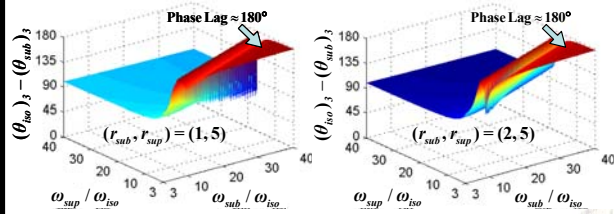
Superstructure relative to Substructure



Phase Lags of Higher modes (II)

■ Phase lag in 3rd mode shape

Super-Floor relative to Substructure

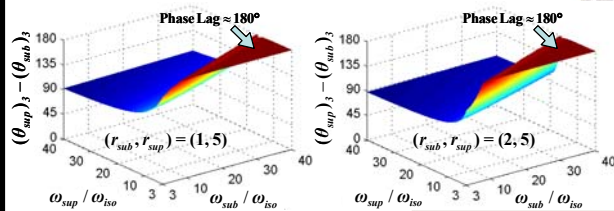


Effective Higher Mode
= 3rd Mode

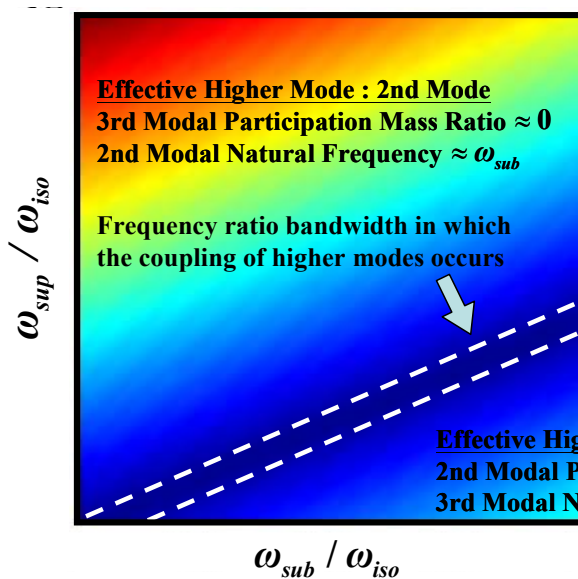


Phase Lag = 180 degrees
(Super-Floor—Substructure)
(Superstructure—Substructure)

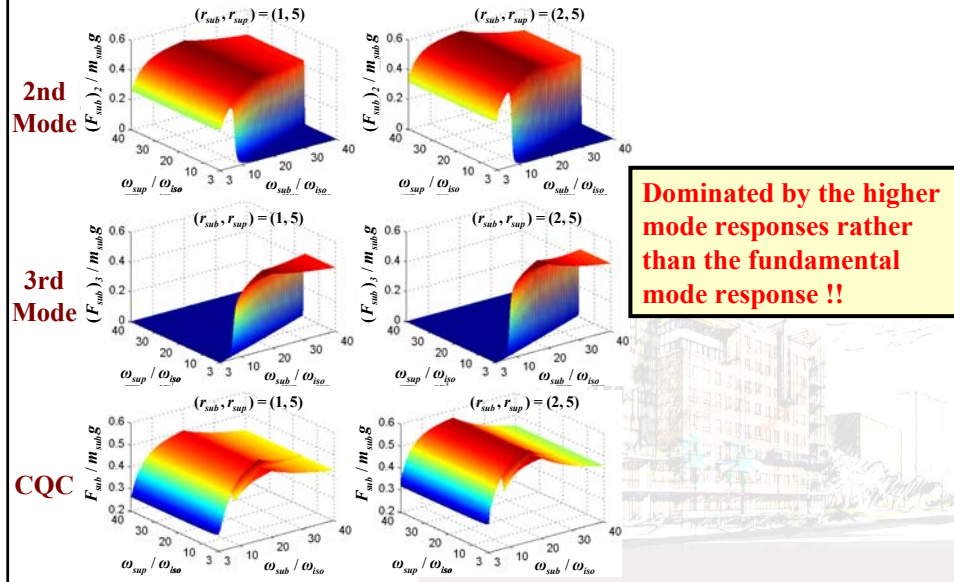
Superstructure relative to Substructure



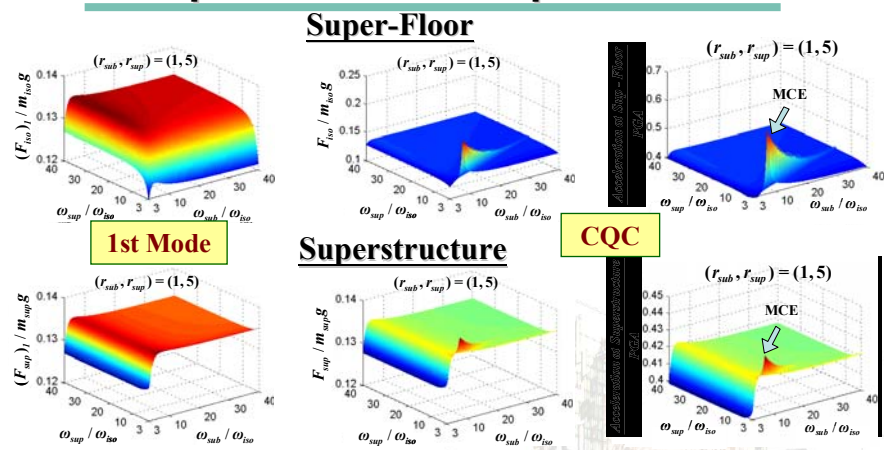
Summarized Dynamic Characteristics of Higher Modes



Inertia Force Response at Substructure

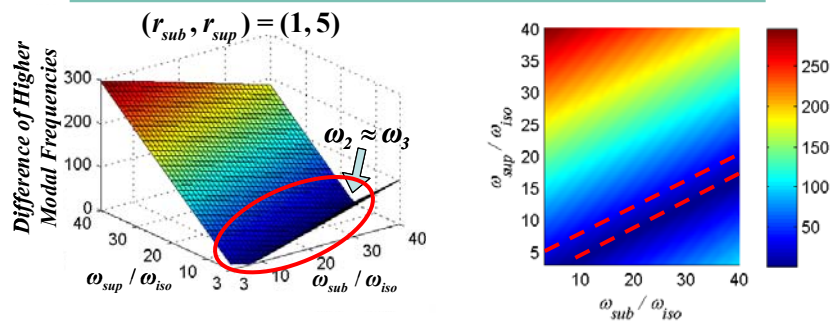


Inertia Force (Acceleration) Responses at Super-Floor and Superstructure



- Dominated by the fundamental mode unless the coupling of higher modes occurs !!
- Significant response amplifications within the frequency ratio bandwidth due to modal coupling effect (MCE)!!

Coupling of Higher Modes (MCE)



■ Modal natural frequencies of higher modes are almost identical ($\omega_2 \approx \omega_3$)

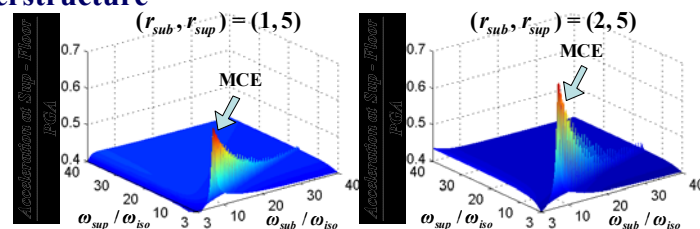
■ Within the bandwidth (MCE)

■ Modal participation mass ratios of higher modes have rapid and unstable variations

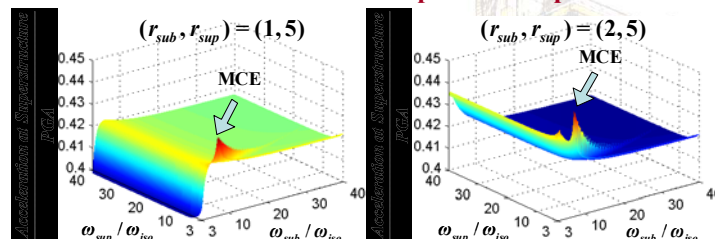
■ Acceleration (inertia force) responses at super-floor and superstructure are enlarged greatly

Adverse Effect Arising from Coupling of Higher Modes

■ Significantly enlarged acceleration responses at super-floor and superstructure



Ratios of maximum acceleration responses at super-floor to PGA



Ratios of maximum acceleration responses at superstructure to PGA

Coupling of Higher Modes due to Improper Design (I)

- ω_{iso}^2 is one root of the characteristic equation and $\omega_{iso}^2 / \omega_{sub}^2 \ll 1$
- The other two roots of the characteristic equation are identical

Standard forms of hyperbolas

$$\left(\frac{\omega_{sub}}{\omega_{iso}}\right)^2 = (1 + r_{sup}) \left(\frac{\omega_{sup}}{\omega_{iso}}\right)^2 + \frac{r_{sub} r_{sup} - r_{sub} - r_{sup} - 1}{r_{sub}}$$

$$\left(\frac{\omega_{sub}}{\omega_{iso}}\right)^2 = (1 + r_{sup}) \left(\frac{\omega_{sup}}{\omega_{iso}}\right)^2 + \frac{(r_{sub} - 1)(1 + r_{sup})}{r_{sub}}$$

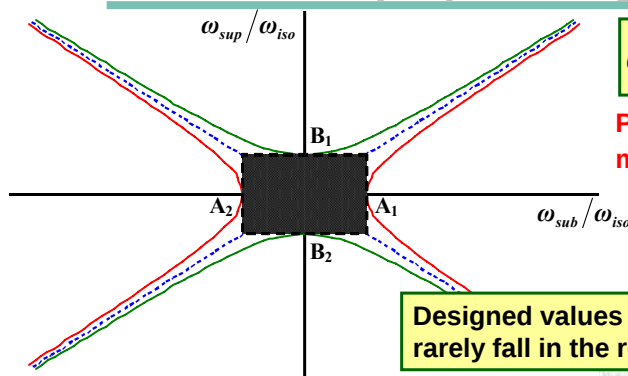
Asymptote in the first quarter

$$\omega_{sub} = \omega_{sup} \sqrt{1 + r_{sup}}$$

If the equation is satisfied

Coupling of higher modes occurs !!

Coupling of Higher Modes due to Improper Design (II)



$$\omega_{sub} = \omega_{sup} \sqrt{1 + r_{sup}}$$

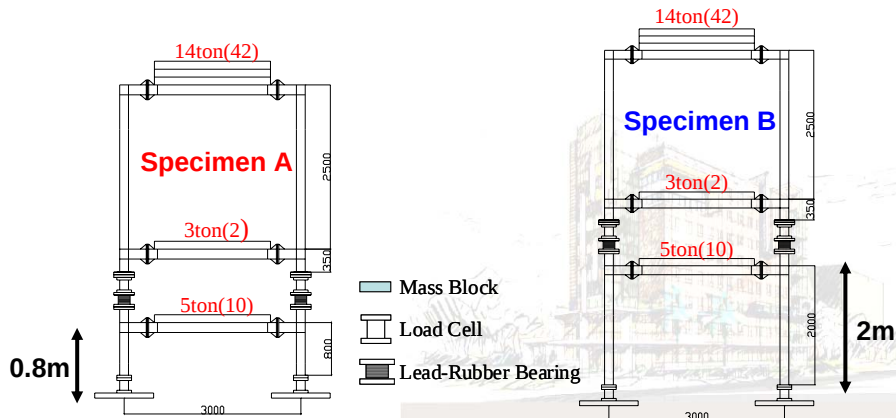
Provide more useful and meaningful information!!

Designed values of $\omega_{sub} / \omega_{iso}$ and $\omega_{sup} / \omega_{iso}$ rarely fall in the rectangular area

- Capable of accurately predicting the condition that MCE occurs
- Linear function
- Independent of ω_{iso} → Condition that MCE occurs is independent of
 - Various types of isolation bearings
 - Diverse mechanical properties of isolation bearings
 - Different deformation extent of isolation bearings (linear or nonlinear)

Test Structures (I)

- Shaking table test, 2 test structures, 1/4 scaled down models
- 3-story steel frame - Substructure, Super-Floor and Superstructure
- Mid-story isolation system composed of 4 lead-rubber bearings
- Plane dimension: 3m (in the direction of shaking) × 2m

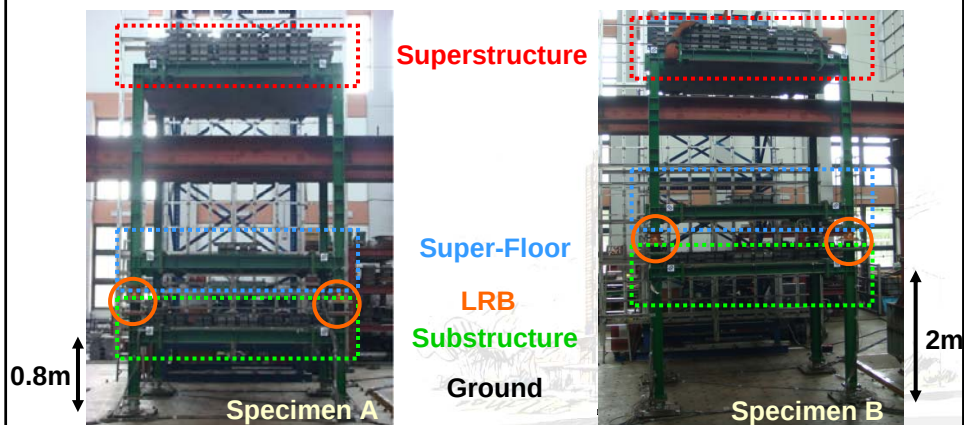


Test Structures (II)

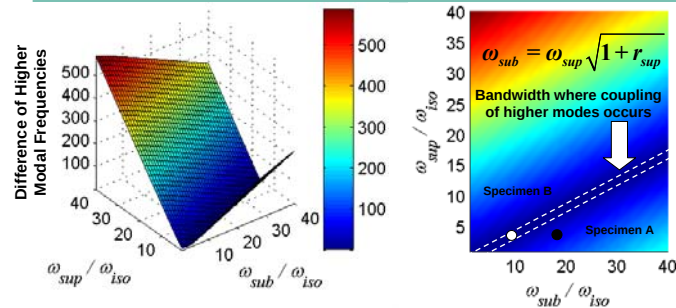
Effective isolated period at design displacement of 55mm (T_{eD}) = 1sec

Inherent damping ratios for substructure and superstructure = 5%

Equivalent damping ratio at design displacement of 55mm (ξ_{eD}) = 12%



Design Parameters of Test Structures



	m_{sub} / m_{iso}	m_{sup} / m_{iso}	$\omega_{sub} / \omega_{iso}$	$\omega_{sup} / \omega_{iso}$
Specimen A	1.7	4.7	17.7	3.3
Specimen B	1.7	4.7	7.8	3.3

A typical mid-story isolated building
w/o "Modal Coupling Effect"

$$\approx 3.3 \sqrt{(1 + 4.7)}$$

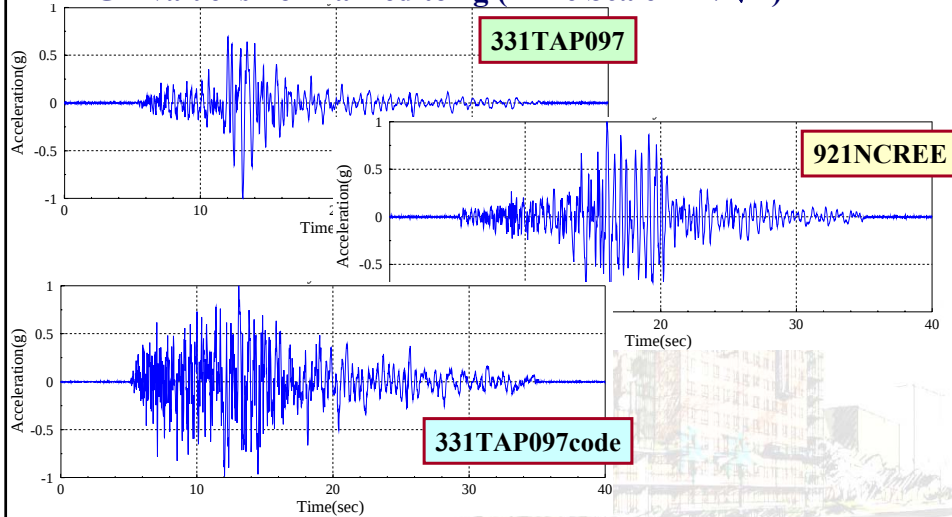
Improper design with
"Modal Coupling Effect"

Earthquake Test Program

Test Name	Input Excitation	Earthquake Component	Time Scale	Shaking Direction	Test PGA Value (g)
331TAP097	TAP097 Station 2002 Hualien Earthquake Real Earthquake	EW	$1/\sqrt{4}$	Uniaxial X	0.08 0.16 0.24 0.32
921NCREE	TAPBAF Station 1999 Chi-Chi Earthquake Real Earthquake	EW	$1/\sqrt{4}$	Uniaxial X	0.08 0.16 0.24 0.32
331TAP097code	TAP097 Station 2002 Hualien Earthquake Artificial Earthquake	EW	$1/\sqrt{4}$	Uniaxial X	0.08 0.16 0.24 0.32

Ground Acceleration Histories

■ PGA value is normalized to 1g (Time Scale = $1/\sqrt{4}$)

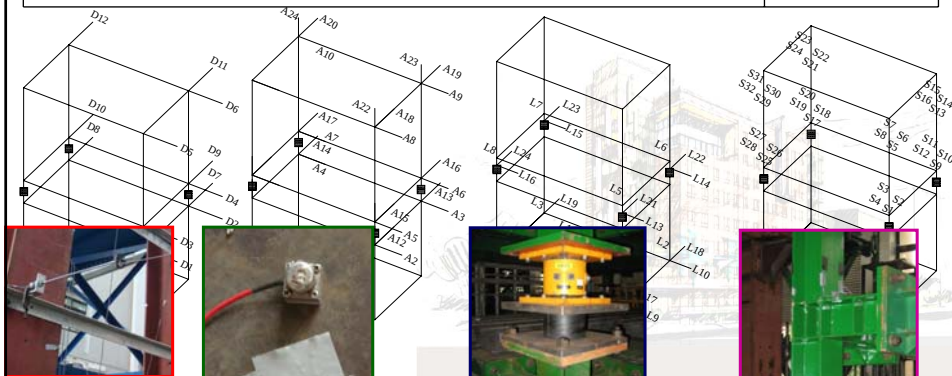


1999 Chi-Chi Earthquake: Intra-plate earthquake inside Taiwan Island

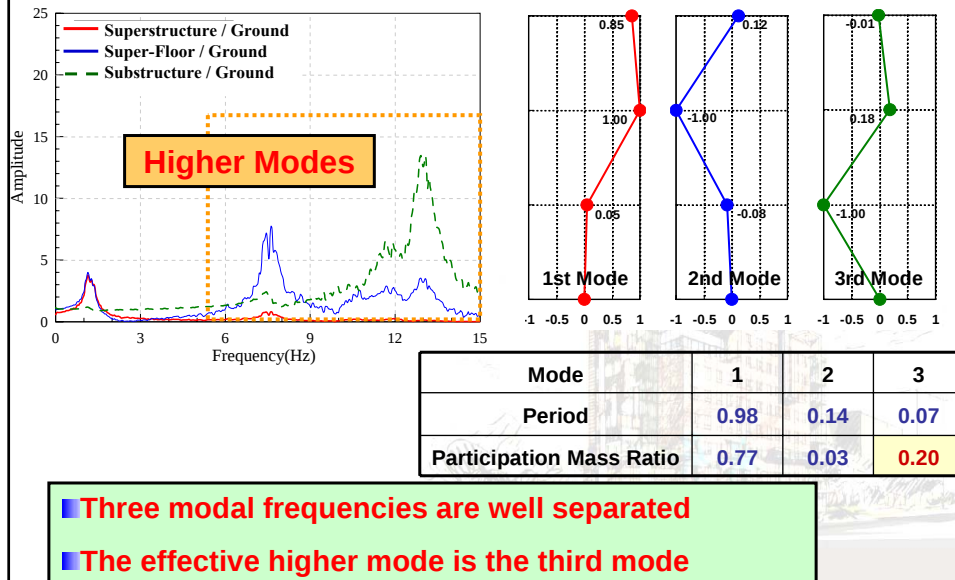
2002 Hualien Earthquake: Inter-plate earthquake around Taiwan Island

Measurement Instrumentations

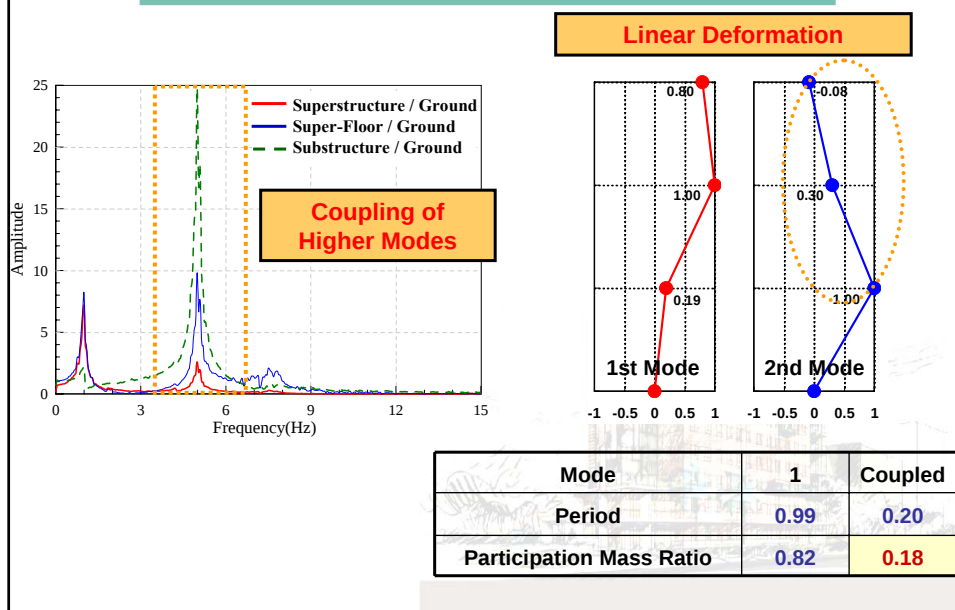
No.	Transducer	Measurement	ID	Channel Number
1	Diaplacement Transducer (Temposonics II)	Relative Displacement	D	12
2	Accelerometer	Absolute Acceleration	A	24
3	Load Cell	Axial Load and Shear Force	L	24
4	Strain Gauge	Axial Strain	S	32
Total Number				92



System Identification – Specimen A

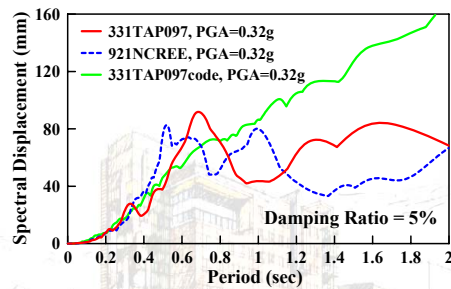
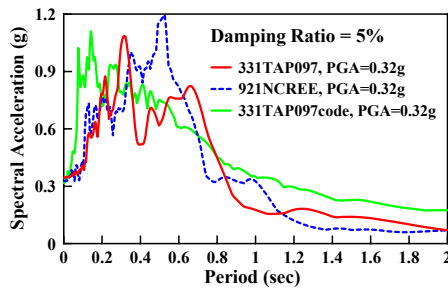


System Identification – Specimen B



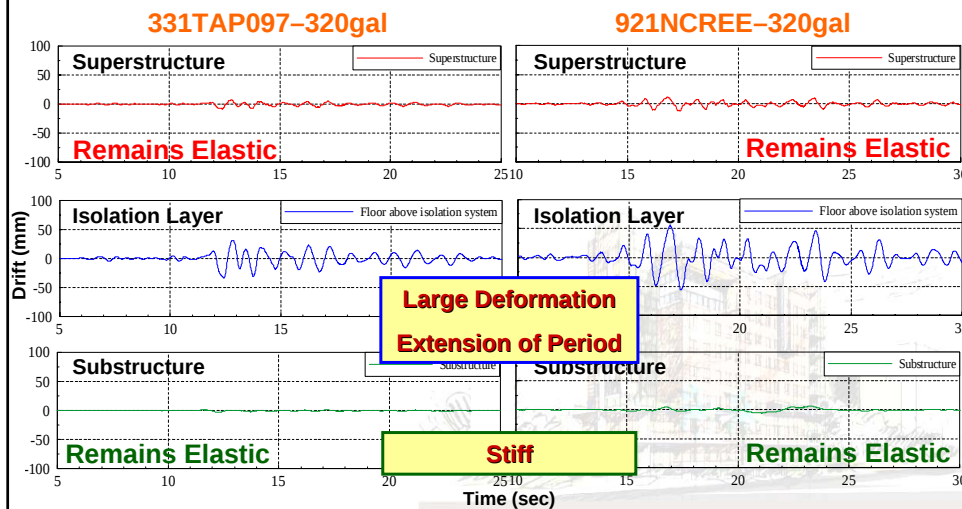
Response Spectra of Input Ground Motions

■ PGA value is normalized to 0.32g (Time Scale = $1/\sqrt{4}$)



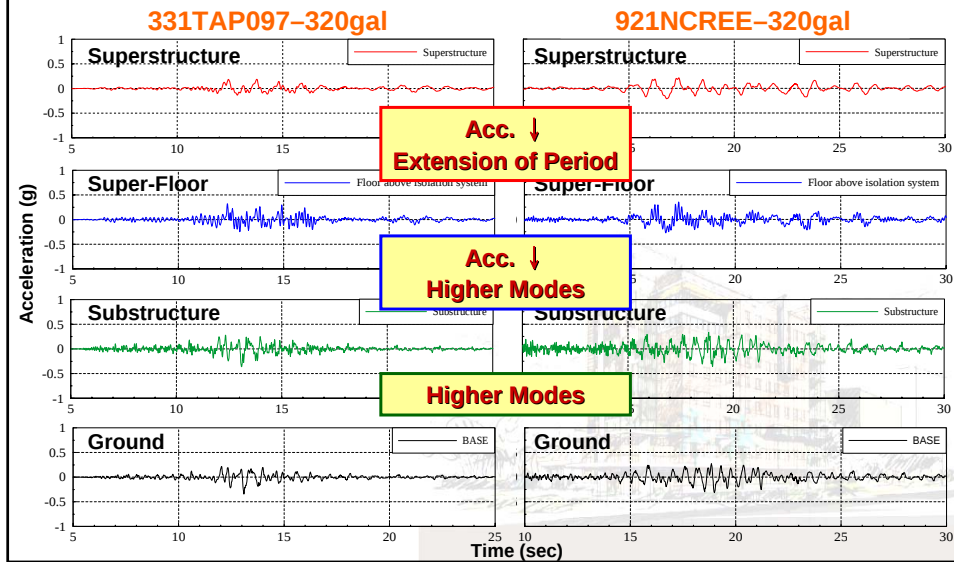
Response Time Histories – Specimen A

■ Drift Responses



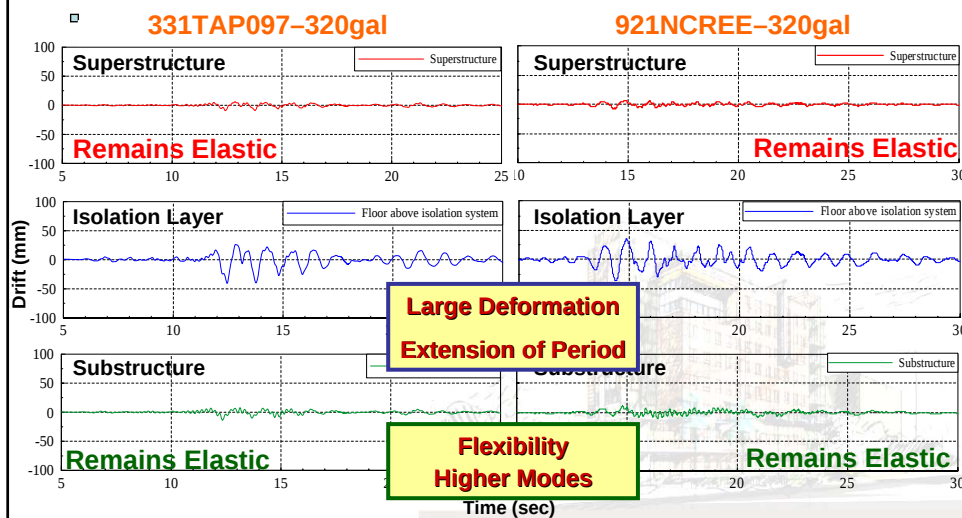
Response Time Histories – Specimen A

Acceleration Responses



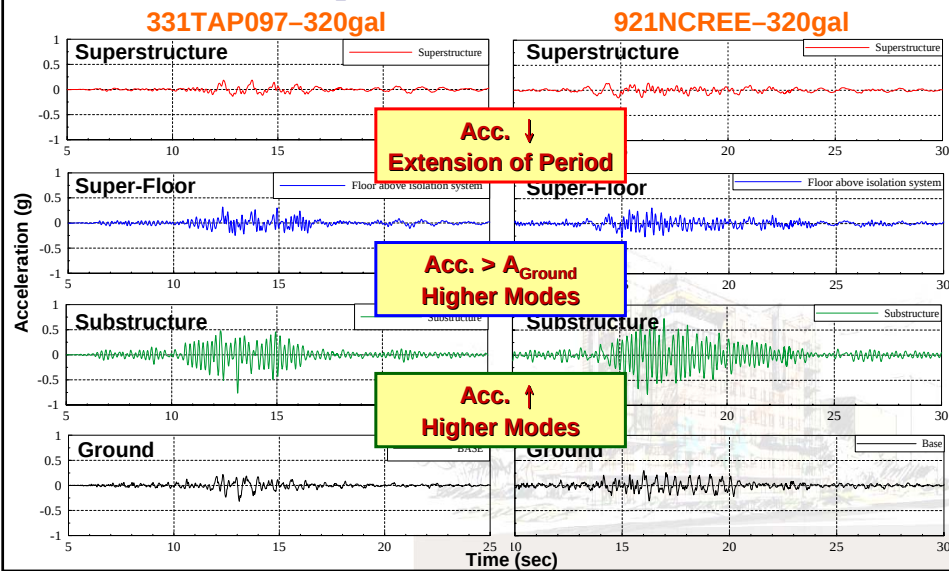
Response Time Histories – Specimen B

Drift Responses



Response Time Histories – Specimen B

Acceleration Responses



Video of Shaking Table Test - Specimen A

331TAP097 – 320gal



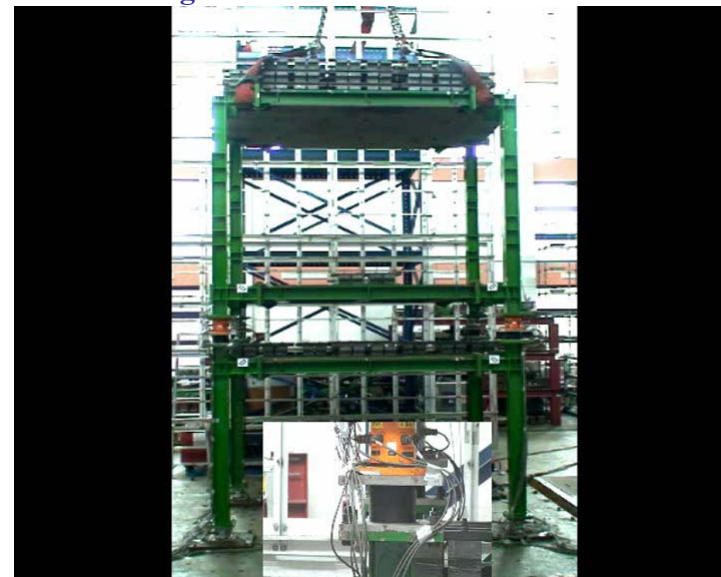
Video of Shaking Table Test - Specimen A

■ 921NCREE – 320gal



Video of Shaking Table Test - Specimen B

■ 331TAP097 – 320gal



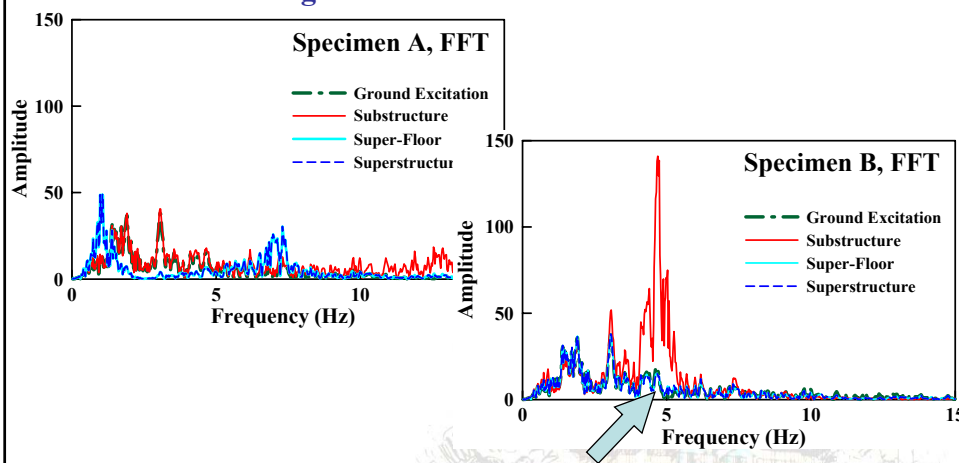
Video of Shaking Table Test - Specimen B

■ 921NCREE – 320gal



Frequency Responses

■ 331TAP097 – 320gal



Significant frequency response at the coupled frequency (about 5Hz)

Comparisons of Test Results (I)

Acceleration Responses

Specimen A - w/o Modal Coupling Effect

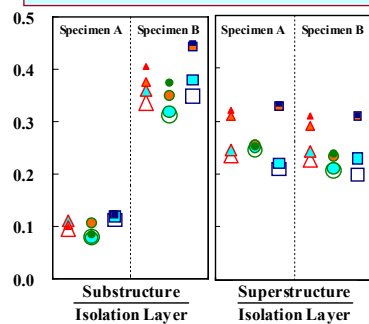
Peak Acceleration Ratio	Substructure PGA	Super-Floor PGA	Superstructure PGA
331TAP097-0.32g	1.05	0.62	0.41
921NCREE-0.32g	1.34	0.85	0.65
331TAP097code-0.32g	2.41	1.28	0.72

Specimen B - with Modal Coupling Effect

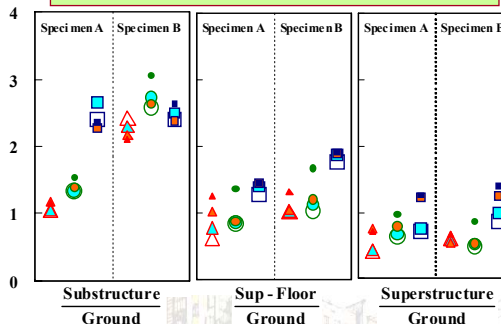
Peak Acceleration Ratio	Substructure PGA	Super-Floor PGA	Superstructure PGA
331TAP097	2.43	1.02	0.60
921NCREE	2.58	1.03	0.49
331TAP097code-0.32g	2.41	1.77	0.86

Comparisons of Test Results (II)

Peak Drift Response Ratio



Peak Acceleration Response Ratio



331TAP097: ▲ PGA=0.08g ▲ PGA=0.16g ▲ PGA=0.24g ▲ PGA=0.32g
 921NCREE: ● PGA=0.08g ● PGA=0.16g ● PGA=0.24g ● PGA=0.32g
 331TAP097code: ■ PGA=0.08g ■ PGA=0.16g ■ PGA=0.24g ■ PGA=0.32g

- Coupling of higher modes leads to higher acceleration response at super-floor
- Condition in which coupling of higher modes occurs is independent of intensity of earthquake excitation
- Seismic responses at superstructure are almost not affected by flexibility of substructure and coupling of higher modes

Following Experimental Scheme (I)

■ Field Test Program at New Civil Engineering Building of NTU

■ Ambient Vibration Test

■ Forced Vibration Test (Shaker)



Following Experimental Scheme (II)

■ MDOF models for superstructure and substructure

■ Test Specimens

■ Structural Model

- Base Isolation
- Mid-Story Isolation

■ Bearing Type

- LRB
- HDRB
- RTB

■ Supplemental Damping

- w/o & with Viscous Damper (VD)

■ Shaking Direction

- Uniaxial Test
- Biaxial Test



Test Structures (I)

Effective isolated period at design displacement of 55mm (T_{eD}) = **1sec**

Inherent damping ratios for substructure and superstructure = **5%**

Equivalent damping ratio at design displacement of 55mm (ξ_{eD}) = **20%**



Base Isolation



One-Story Substructure

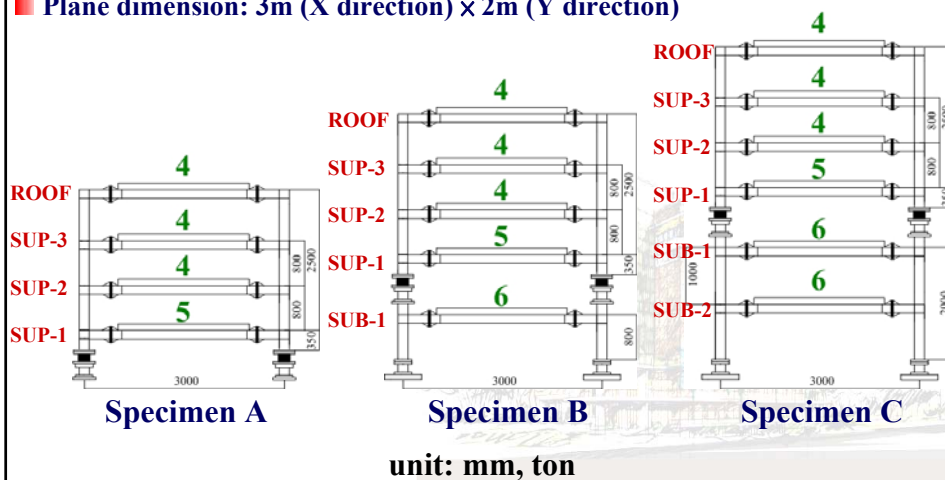


Two-Story Substructure

Mid-Story Isolation

Test Structures (II)

- Shaking table test, 3 test structures, 1/4 scaled down models
- Mid-story isolation system composed of 4 high damping rubber bearings
- Plane dimension: 3m (X direction) × 2m (Y direction)



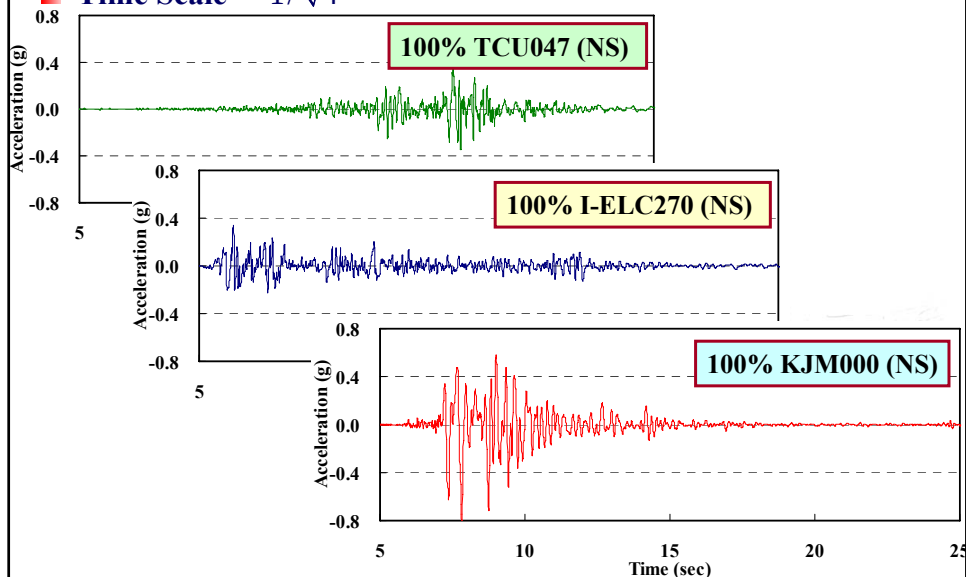
Earthquake Test Program

■ Time Scale = $1/\sqrt{4}$

Input Excitation	Test PGA Percentage	Shaking Direction		Earthquake Component	Test PGA Value (g)
Chi-Chi/ TCU047 Chi-Chi, Taiwan 1999/09/21	300%	Uniaxial	X	NS	1.19
	200%	Biaxial	X	NS	0.79
			Y	EW	0.6
El Centro/ I-ELC270 Imperial Valley, U.S. 1940/05/19	200%	Uniaxial	X	NS	0.70
	150%	Biaxial	X	NS	0.52
			Y	EW	0.32
KJMA/ KJM000 Kobe, Japan 1995/01/16	100%	Uniaxial	X	NS	0.83
	100%	Biaxial	X	NS	0.83
			Y	EW	0.63

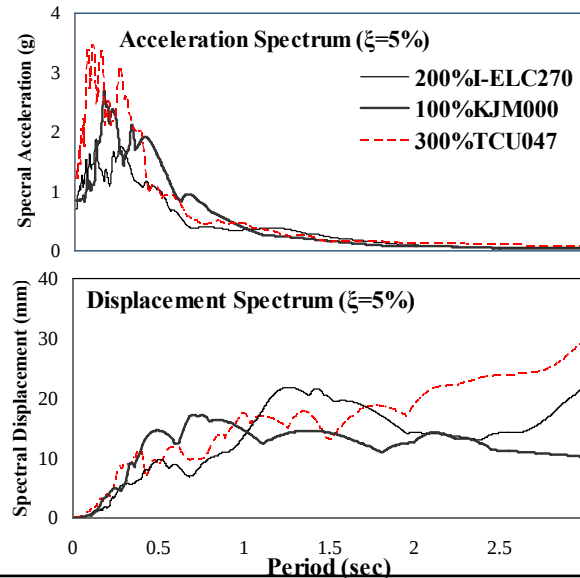
Ground Acceleration Histories

■ Time Scale = $1/\sqrt{4}$



Response Spectra of Input Ground Motions

■ Time Scale = $1/\sqrt{4}$



System Identification

■ 300%TCU047 Uniaxial Test

Specimen		A	B	C
Modal Natural Period (sec)	1 st mode	0.71	0.82	0.90
	2 nd mode	0.10	0.11	0.22
Agreeable with effective period of isolation system corresponding to experimental maximum deformation				
Modal Damping Ratio (%)	1 st mode	17.90	16.70	13.70
	2 nd mode	2.50	2.20	8.10
	4 th mode	no	no	3.40
Modal Participation Mass Ratio	1 st mode	99.88	77.16	67.63
	2 nd mode	0.01	2.41	14.96
	4 th mode	no	no	8.52

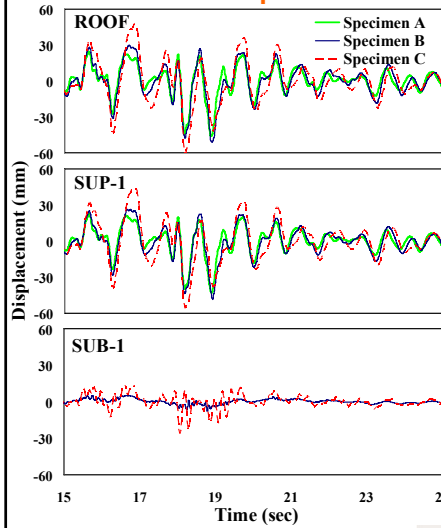
Less fundamental modal damping ratio in mid-story isolation design

More significant participation of higher modes in mid-story isolation design

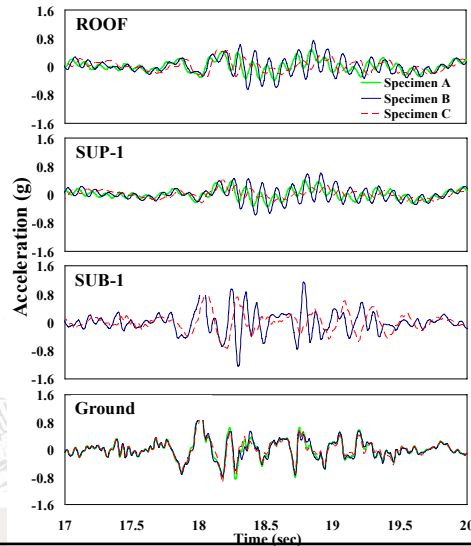
Response Time Histories (I)

300%TCU047 Uniaxial Test

Relative Displacement



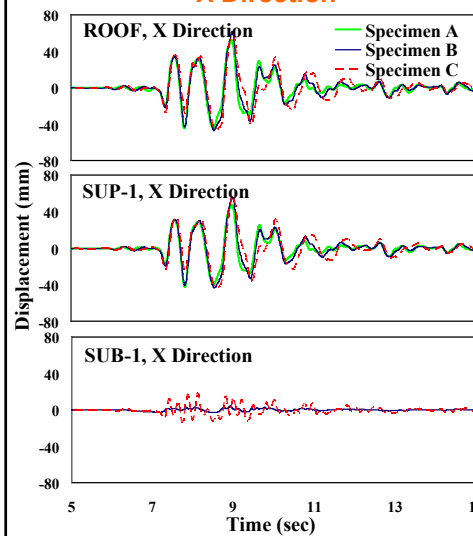
Absolute Acceleration



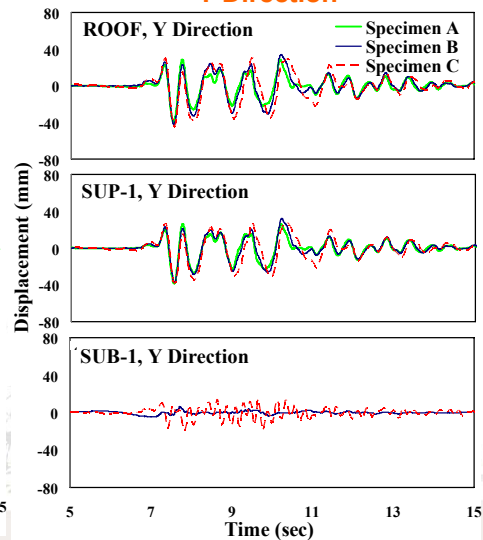
Response Time Histories (II)

100%KJM000 Biaxial Test, Relative Displacement Responses

X Direction

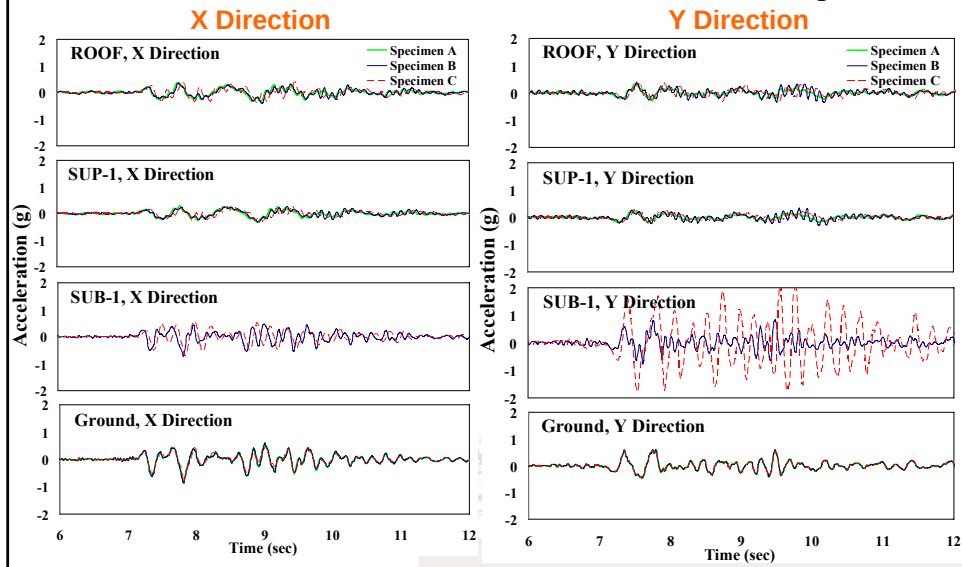


Y Direction



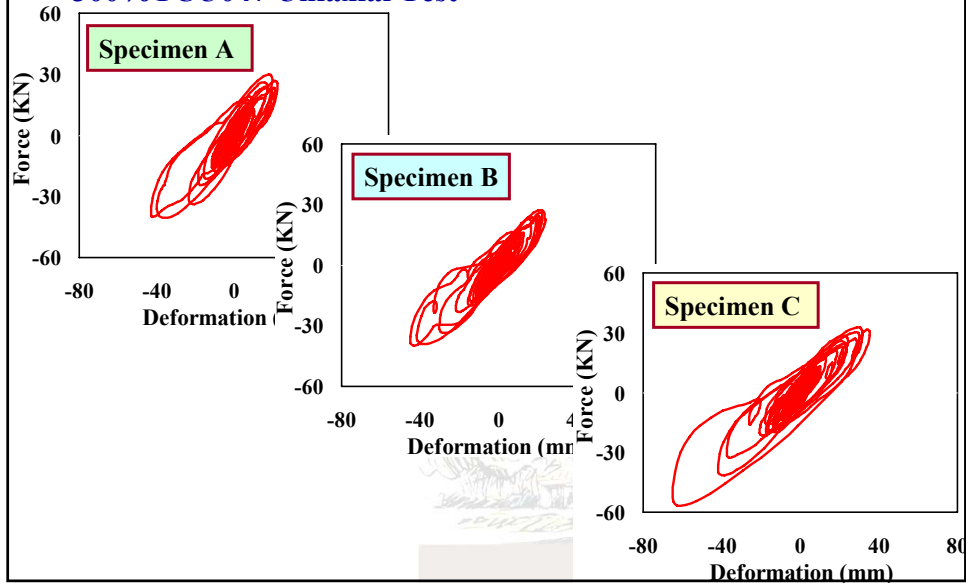
Response Time Histories (III)

100%KJM000 Biaxial Test, Absolute Acceleration Responses



Hysteresis Loop of HDRB

300%TCU047 Uniaxial Test



Comparisons of Test Results

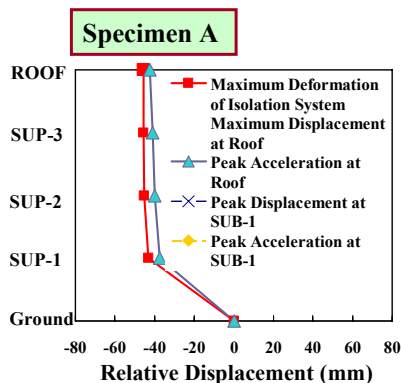
Input Excitation		300% TCU047			200% I-ELC270			100% KJM000		
Specimen		A	B	C	A	B	C	A	B	C
Maximum Deformation of HDRB (mm)		43.0	45.1	65.1	33.7	38.5	53.8	48.3	54.5	58.0
Ratio of Peak Acceleration Response to PGA	ROOF	0.39	0.59	0.42	0.53	0.74	0.61	0.5	0.51	0.65
	SUP3	0.28	0.32	0.38	0.47	0.60	0.54	0.44	0.48	0.51
	SUP2	0.26	0.34	0.38	0.41	0.52	0.54	0.39	0.37	0.39
	SUP1	0.33	0.48	0.37	0.47	0.66	0.51	0.40	0.39	0.48
	SUB1	no	0.97	0.82	no	0.94	0.97	no	0.76	0.95
	SUB2	no	no	1.00	no	no	1.27	no	no	1.13
Peak Shear Force across Isolation System (kN)		46.8	45.3	68.6	44.8	44.2	55.2	52.8	52.8	58.6
Peak Base Shear (kN)		46.8	81.9	87.1	44.8	53.4	76.2	52.8	69.1	84.4

- Fundamental modal damping ratio may be significantly affected by substructure
- Higher mode participation

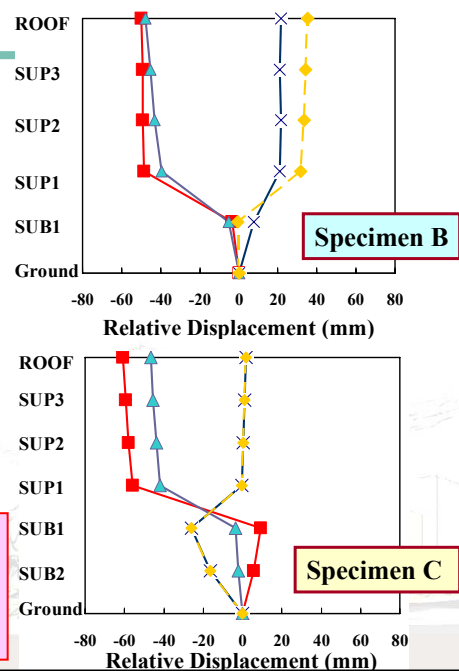
Maximum deformation of isolation system is increased when isolation system is installed at higher story

Displacement Response Vertical Distribution

300%TCU047 Uniaxial Test



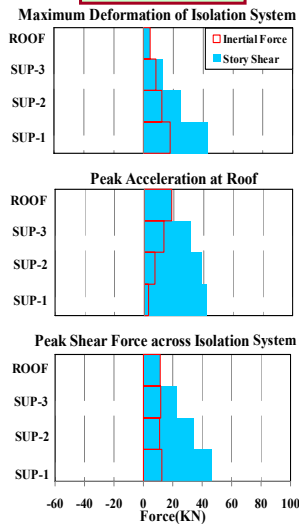
Drift responses at superstructure are really limited (i.e. behaves as a rigid body) regardless of base isolation or mid-story isolation design



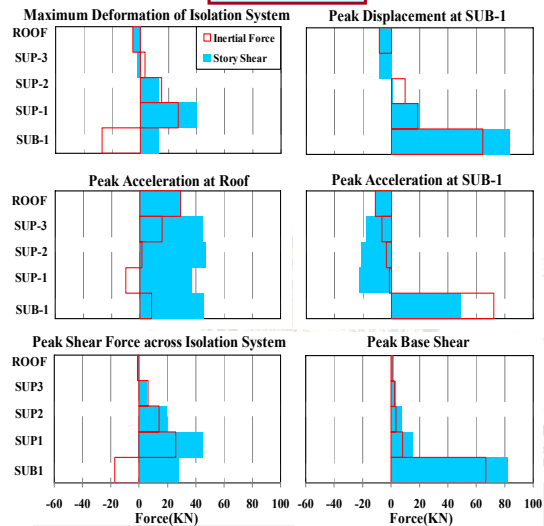
Force Response Vertical Distribution (I)

300%TCU047 Uniaxial Test

Specimen A



Specimen B

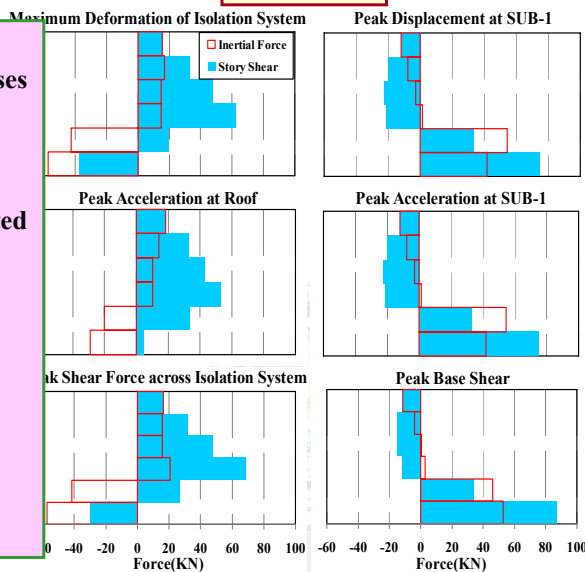


Force Response Vertical Distribution (II)

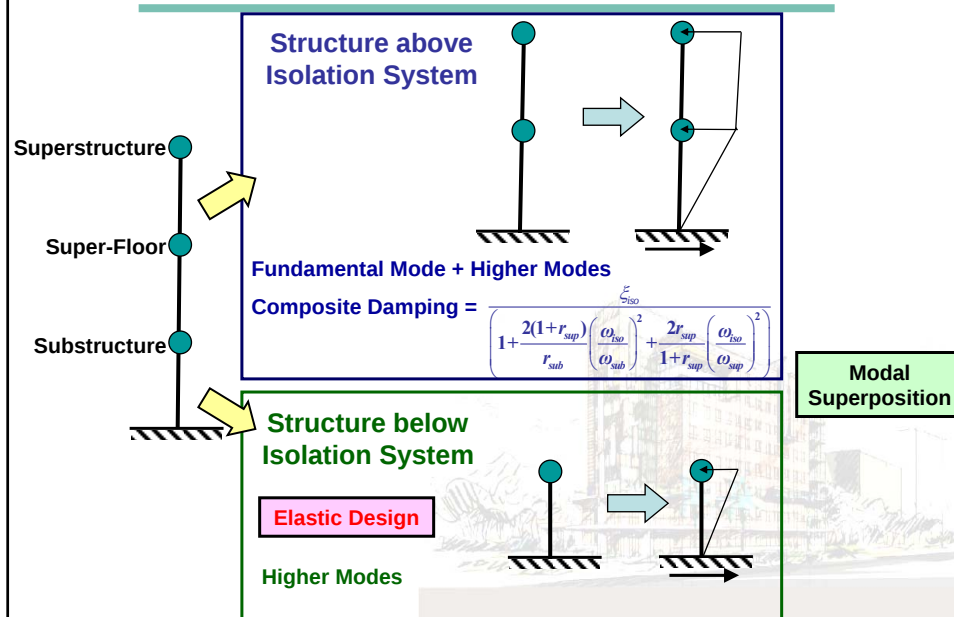
300%TCU047 Uniaxial Test

Specimen C

- Phase lag of 180 degrees between inertia force responses at substructure and superstructure
- Inertia force responses at superstructure are very limited when peak seismic responses occur at substructure
- Force responses at superstructure are mainly attributed to fundamental mode response
- Contribution of higher mode responses at substructure is significant

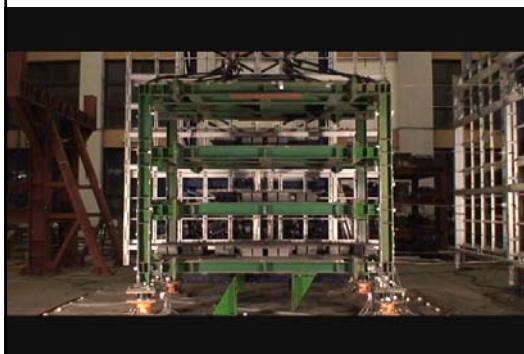


Equivalent Lateral Force Design Philosophy



Video of Shaking Table Test - Specimen A

■ KJM000 Uniaxial Test



Video of Shaking Table Test - Specimen B

■ TCU047 Uniaxial Test



Video of Shaking Table Test - Specimen C

■ KJM000 Uniaxial Test



Conclusions

- Mid-story isolated buildings should not be designed by simply considering superstructure without any influence attributed to substructure
- **First modal composite damping ratio** is derived based on a simplified three-lumped-mass structural model
- Fundamental and higher mode responses should be considered in **equivalent lateral force design procedure**
- Experimental and analytical results indicate that **coupling of higher modes** results in undesired seismic responses
- Access to guarantee mid-story isolation design against **coupling of higher modes** is deduced
- Existence of substructure leads to **decrease of fundamental modal damping ratio and participation mass ratio together with increase of deformation of isolation system**
- Mid-story isolation design is **effective in reducing seismic demand** if substructure and superstructure are designed appropriately

~THE END~
THANK YOU FOR YOUR ATTENTION

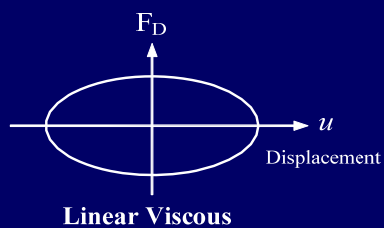


Distribution Methods for Damping Coefficients of Viscous Dampers to Buildings

Jenn-Shin Hwang

Professor and Division Head
National Taiwan University of Science and Technology
National Center for Research on Earthquake Engineering

Force-Displacement Relationship



- ◆ No restoring force (effective stiffness)
- ◆ does not change the frequency of structure
- ◆ simple design method
- ◆ 90 degree out of phase between force and displacement

$$F_D = C |\dot{u}|^\alpha \operatorname{sgn}(\dot{u})$$

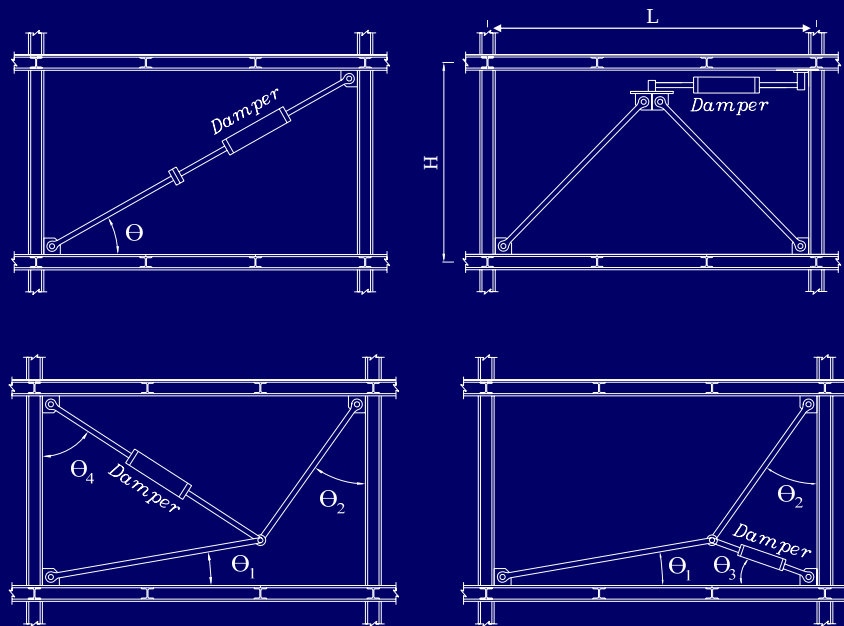
Practical Applications to Bridges



Headquarter of Buddhist Association



Installation Configuration of Viscous Dampers to Buildings



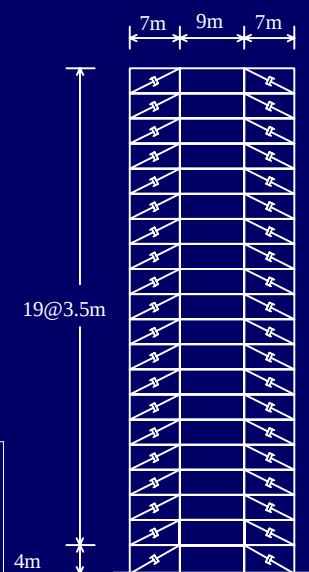
Equivalent viscous damping ratio of buildings

--- linear viscous dampers

$$\xi_d = \frac{W_D}{4\pi W_s} \quad \xi_{eff} = \xi_o + \frac{\sum_j W_j}{4\pi W_s}$$

$$\xi_{eff} = \xi_o + \xi_d = \xi_o + \frac{T \sum_j C_j f_j^2 \phi_{rj}^2}{4\pi \sum_i m_i \phi_i^2}$$

- ◆ Infinite number of selections for C_j
- ◆ not necessary to locate dampers at every story



Frequently Asked Questions by Engineers:

Where to locate dampers?

How many dampers are Needed?

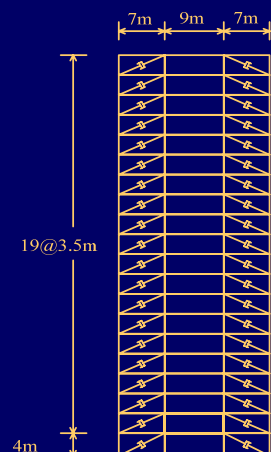
How much damping coefficient is needed for each damper?

Uniform Distribution for Damping Coefficients — Method 1

$$\xi_{eff} = \xi_0 + \xi_d = \xi_0 + \frac{T \sum_j C_j f_j^2 \phi_{rj}^2}{4\pi \sum_i m_i \phi_i^2}$$

$$C_j = \frac{\xi_d 4\pi \sum_{i=1}^{Roof} m_i \phi_i^2}{T \sum_{j=1}^{Roof} f_j^2 \phi_{rj}^2}$$

- same damping coefficient
- dampers in every story
- simplest and currently often used
- easy construction



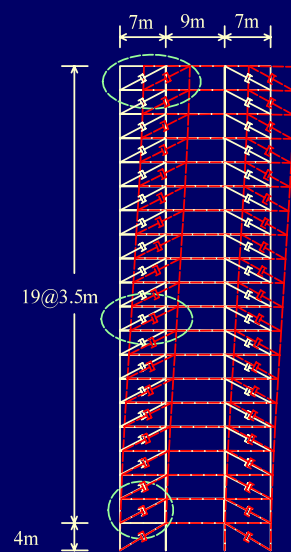
Simplified Sequential Search Algorithm (SSSA)
– Garcia and Soong (2001)

Simplified Effective Damping Ratio: Inverse triangular
model shape

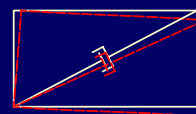
Optimal Index : relative velocity

Computational efforts

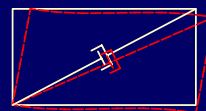
Modal Deformation



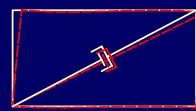
(a) Mode Shape



(b.1) 20F



(b.2) 10F



(b.3) 2F

(b) Relative Modal Displacement

Distribution Proportional to Story Shear – Method 2

$$\text{Story shear} = V_j \propto \omega^2 \sum_j^{\text{Roof}} m_j \phi_j$$

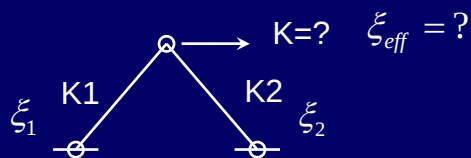
$$\text{Defining } S_j = \sum_j^{\text{Roof}} m_j \phi_j$$

$$C_j = \frac{S_j}{\sum_i S_i} \sum_i C_i$$

$$C_j = \frac{4\pi \xi_d S_j \sum_i m_i \phi_i^2}{T \sum_i S_i (f_i \phi_{ri})^2}$$

Old Concept, New Application

- It is well known to combine stiffness of elements connected in parallel or in series, and to formulate a stiffness matrix for a structural system



- How to combine the damping ratios of various elements with different element damping ratios?

Idea:

In 1975 Prof. Raggett of USC proposed an idea for damping combination of elements:

The contribution of the element damping ratio to the system damping ratio is equal to the element damping ratio weighted by the element strain energy divided by the total system strain energy

$$\xi_e = \sum_i \xi_i \frac{U_i}{U_t}$$

Concept Application:

Johnson and Kienholz (AIAA journal 1982)

- ♦ Realized Raggett's concept by proposing modal strain energy method
- ♦ Composite damping ratio in Taiwan Seismic Design Code of Buildings ξ_c

$$(\xi_e)_j = \frac{\sum_i (\phi_j)_i^T K_i (\phi_j)_i \xi_i}{\phi_j^T K_s \phi_j}$$

$$\xi_e = \frac{\sum K_{eff,i} D_{d,i}^2 \left[\xi_{eq,i} + \xi_{p,i} \frac{K_{eff,i}}{K_{p,i}} + \xi_{T,i} \frac{K_{eff,i}}{K_{T,i}} + \xi_{R,i} \frac{K_{eff,i} H^2}{K_{R,i}} \right]}{\sum K_{eff,i} D_{d,i}^2 \left[1 + \frac{K_{eff,i}}{K_{p,i}} + \frac{K_{eff,i}}{K_{T,i}} + \frac{K_{eff,i} H^2}{K_{R,i}} \right]} \quad \text{Isolated bridges}$$

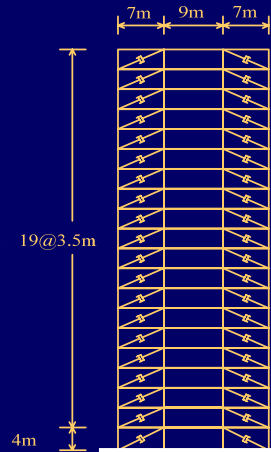
Distribution Proportional to Story Strain Energy – Method 3

$$\text{Story shear} = V_j \propto \omega^2 \sum_j^{\text{Roof}} m_j \phi_j$$

$$\text{Defining } S_j = \sum_j^{\text{Roof}} m_j \phi_j$$

$$\text{Story strain energy} \propto \phi_{rj} S_j = \phi_{rj} \sum_j^{\text{Roof}} m_j \phi_j$$

$$C_j = \frac{4\pi\xi_d \phi_{rj} S_j \sum_i^{\text{Roof}} m_i \phi_i^2}{T \sum_i^{\text{Roof}} \phi_{ri} S_i (f_i \phi_{ri})^2}$$



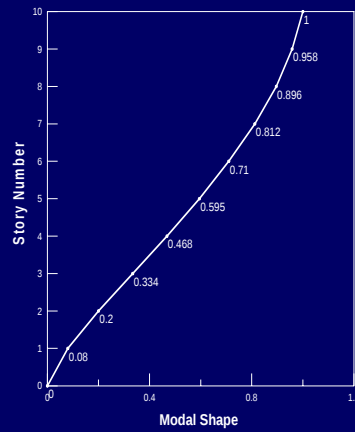
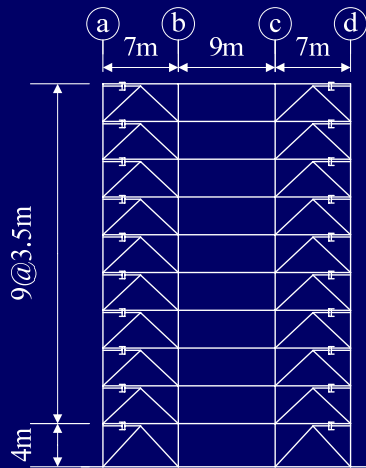
Extended Strain Energy Distribution Method– Method 4

- Further question: “is it necessary to put dampers to every story of the building structure?”
- A possible answer: the damping coefficient may be distributed to those stories (efficient stories) with their strain energy equal to or larger than the average of the total story shear strain energy

$$C_j = \frac{4\pi\xi_d \phi_{rj} S_j \left(\sum_i m_i \phi_i^2 \right)}{T \sum_{i=1}^k \phi_{ri} S_i (f_i \phi_{ri})^2}$$

$k < N$

A ten-story vertically regular building

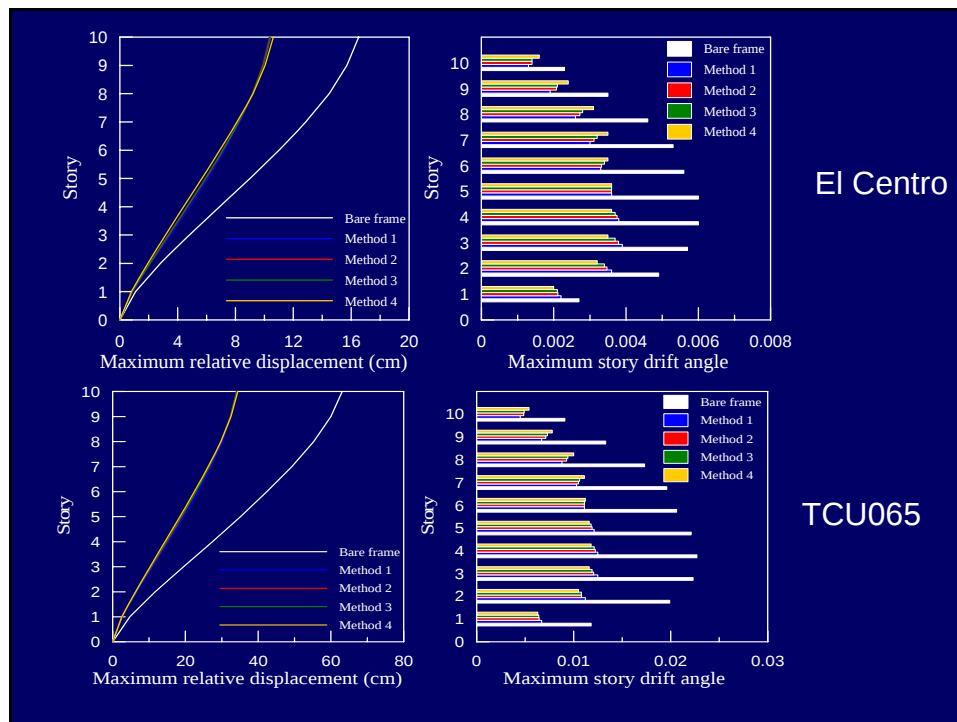


Target damping ratio = 15%
with 10% from viscous dampers

$T_1 = 1.09$ sec

Damping Coefficient

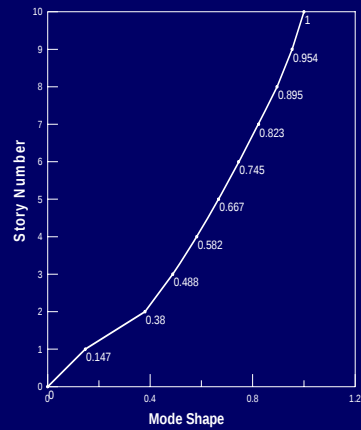
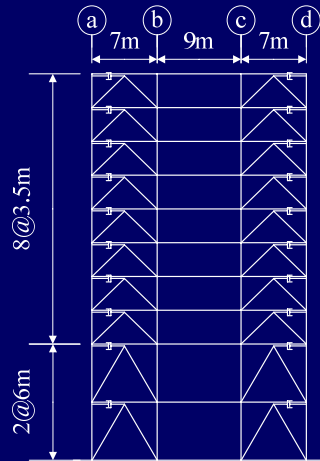
Story	Distribution method 1 (kN-sec/m)	Distribution method 2 (kN-sec/m)	Distribution method 3 (kN-sec/m)	Distribution method 4 (kN-sec/m)
10	5995	1211	458	no damper
9	5995	2370	1340	no damper
8	5995	3449	2606	no damper
7	5995	4421	3924	no damper
6	5995	5267	5115	5724
5	5995	6086	6523	7300
4	5995	6731	7578	8481
3	5995	7222	8142	9112
2	5995	7518	7621	8529
1	5995	7638	5261	5888
Total	59950	51913	48568	45034



Maximum damper force – TCU065

Story	Distribution method 1 (kN)	Distribution method 2 (kN)	Distribution method 3 (kN)	Distribution method 4 (kN)
10	203	50	20	no damper
9	332	147	86	no damper
8	453	272	209	no damper
7	539	403	357	no damper
6	606	536	517	566
5	706	714	756	830
4	787	868	963	1065
3	843	987	1098	1220
2	808	982	998	1112
1	595	726	526	589
Total damper force	5872	5685	5530	5382
Total damper number	20	20	20	12

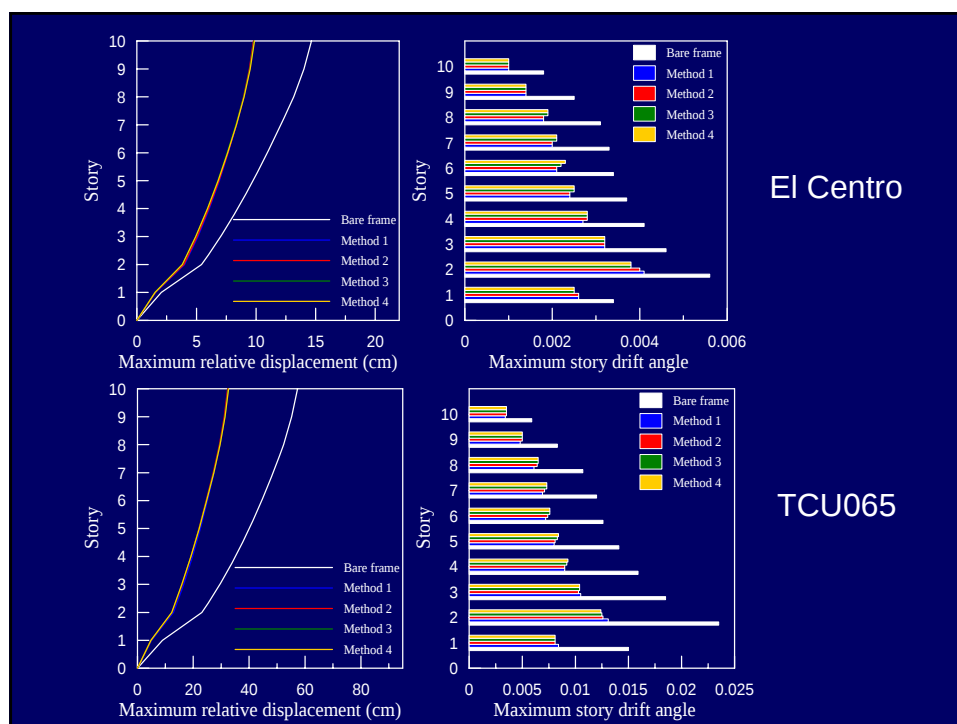
A ten-story vertically irregular structure (with soft stories)



$T=1.14$ sec

Damping Coefficient

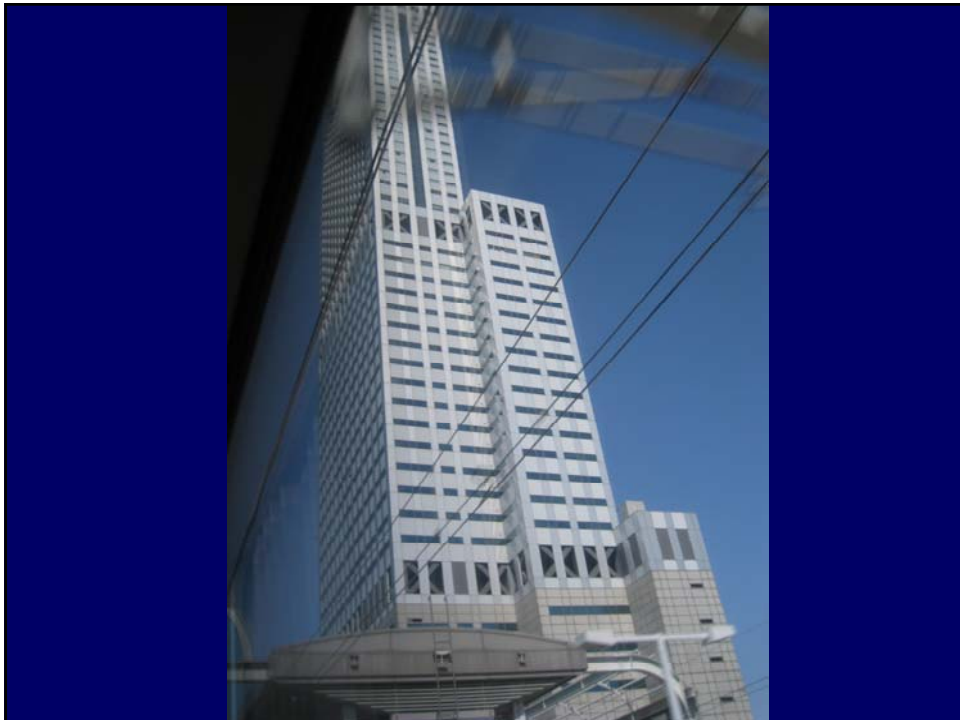
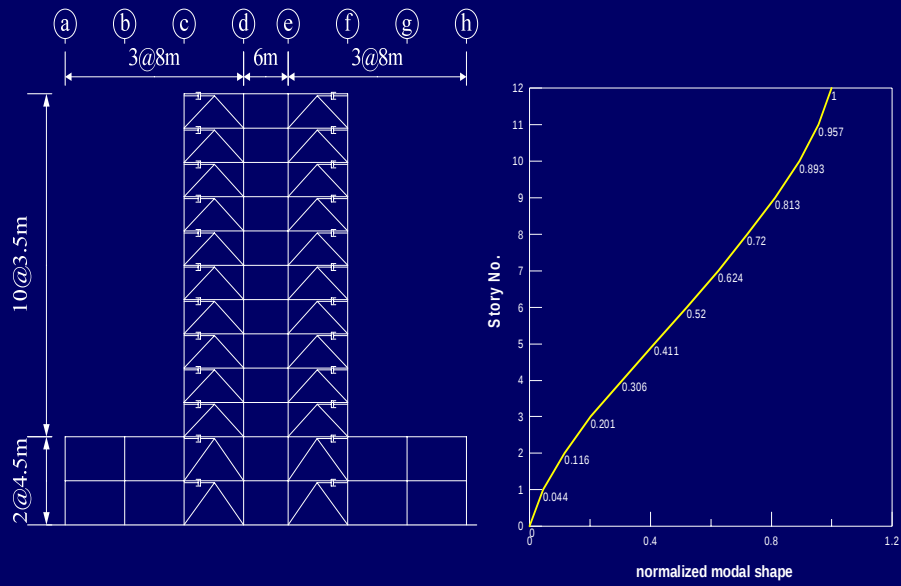
Story	Distribution method 1 (kN-sec/m)	Distribution method 2 (kN-sec/m)	Distribution method 3 (kN-sec/m)	Distribution method 4 (kN-sec/m)
10	5525	857	163	no damper
9	5525	1684	451	no damper
8	5525	2468	854	no damper
7	5525	3197	1244	no damper
6	5525	3864	1588	no damper
5	5525	4558	2101	no damper
4	5525	5168	2697	2833
3	5525	5716	3510	3688
2	5525	6145	8213	8628
1	5525	6311	5371	5643
Total	55250	39968	26192	20792



Maximum damper force -- TCU065

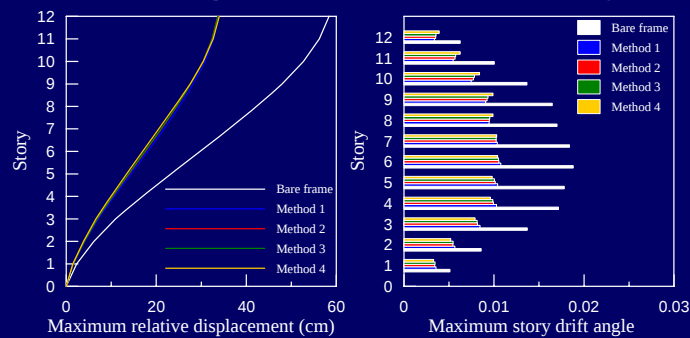
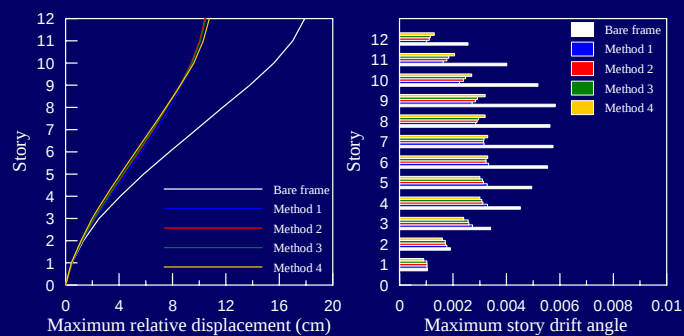
Story	Distribution method 1 (kN)	Distribution method 2 (kN)	Distribution method 3 (kN)	Distribution method 4 (kN)
10	147	26	6	no damper
9	236	79	23	no damper
8	321	150	55	no damper
7	361	212	86	no damper
6	380	267	113	no damper
5	437	362	172	no damper
4	509	477	257	271
3	614	629	391	411
2	1359	1485	1932	2020
1	914	1026	890	935
Total damper force	5278	4713	3925	3637
Total damper number	20	20	20	8

A twelve-story vertically irregular structure (with setback)



Damping Coefficient

Story	Distribution method 1 (kN-sec/m)	Distribution method 2 (kN-sec/m)	Distribution method 3 (kN-sec/m)	Distribution method 4 (kN-sec/m)
12	4997	984	449	no damper
11	4997	1926	1275	no damper
10	4997	2805	2363	no damper
9	4997	3606	3535	no damper
8	4997	4314	4331	4979
7	4997	4928	5389	6195
6	4997	5440	6192	7118
5	4997	5908	6542	7520
4	4997	6255	6862	7888
3	4997	6484	5830	6701
2	4997	6828	5160	5932
1	4997	6957	3187	no damper
Total	59964	56435	51115	46333



Story	Distribution method 1 (kN)	Distribution method 2 (kN)	Distribution method 3 (kN)	Distribution method 4 (kN)
12	167	38	20	no damper
11	287	120	82	no damper
10	404	233	200	no damper
9	497	355	351	no damper
8	529	445	449	510
7	593	567	621	699
6	637	667	760	865
5	646	726	809	927
4	670	793	881	1010
3	580	713	662	765
2	526	675	540	624
1	417	553	273	no damper
Total damper force	5953	5885	5648	5400
Total damper number	24	24	24	14

Conclusions

The proposed method distributing damping coefficients of viscous dampers to efficient stories proportionally to story shear strain energy could be a simple, rational and economic method for the design of viscous dampers, in lieu of the existing complicated systematic procedures for the optimal design of viscous dampers.

中間層隔震建築結構設計實例 (以台中市西屯區四季天韻為例)

科建聯合結構技師事務所
負責人：陳 陸 民 總 經 理
主講人：劉 紹 魁 協 理

內容大綱

- 基地位置及建築概述
- 結構概述
- 隔震設計流程
 - 建築規劃階段
 - 結構分析階段
 - 細部設計階段
 - 結構施工階段
- 隔震設計Q&A

隔震建築案例介紹

惠民段新建工程

工程名稱：四季天韻
工程地址：台中市西屯區
樓層數：(B6F~35F)
樓高：128.7m
業主：興富發建設
碰撞距離：55cm
隔震器數量：32個
阻尼器數量：8個
最大隔震器：140(方形)
隔震器位置：3F
隔震層上方高度：112.2m
建築物短邊尺寸：33.25m
高寬比：3.37
隔震前週期：2.917sec
隔震後週期(設計地震)：5.520sec
隔震後週期(最大地震)：5.797sec



基地位置及建築概述

一、基地位置

➤ 四季天韻：台中市西屯區惠民段117、119等二筆地號



基地位置及建築概述

二、建築概述

➤ 四季天韻：

基地面積：2713.52m²

地下6層(開挖深度20.95m)

地上35層(高度128.7m)

屋突3層(高度8.4m)

樓地板面積：49161.92m²

建築面積：1355.52m²

剪力牆：30cm RC牆

外牆：15cm RC牆

分戶牆：15cm RC牆

室內隔間牆：10cm輕隔間牆

基地位置及建築概述

三、建築用途及樓高

➤ 四季天韻：

B6F：4.1m 停車空間

B3F~B5F：2.8m 停車空間

B2F：3.0m 停車空間

B1F：3.55m 停車空間

1F：7.2m 門廳、辦公室

2F：4.5m 機車停車空間

隔震層：3.5m

3F~35F：3.4m 集合住宅

R1F~R2F：3.0m 梯間、機房、水箱

結構概述

一、結構系統說明

四季天韻：

本工程採隔震結構設計

構架為具RC剪力牆之二元結構系統

主要構件在隔震層上方為填充型鋼柱與鋼梁系統

隔震層下方則為包覆填充型鋼柱與包覆型SRC梁系統

結構概述

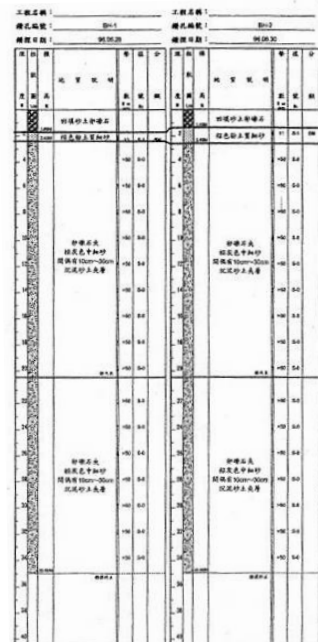
二、結構設計參數

■ 地盤分類

- 地表下2.6m即進入卵礫石層，屬於第一類地盤。
- 工址距車籠埔斷層約9.7km。

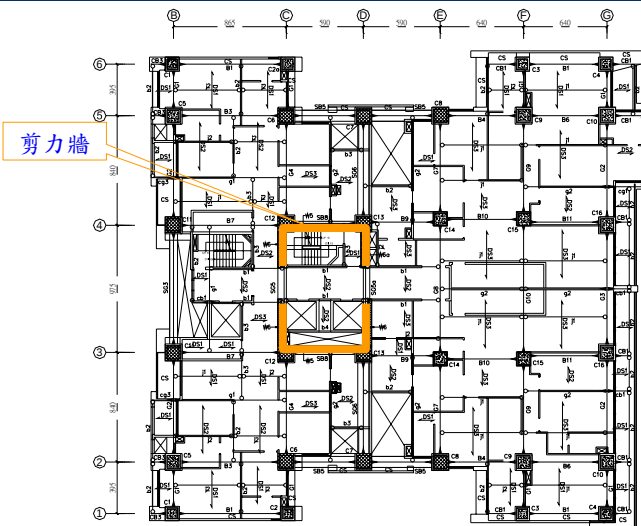
■ 耐震設計參數

- 台中市西屯區 $S_S^D=0.7$ ， $S_1^D=0.4$ ， $S_S^M=0.9$ ， $S_1^M=0.5$ 。
- $S_{DS}=0.721$ ， $T_O^D=0.61\text{sec}$ ， $\text{PGA}=288.4\text{gal}$
- $S_{MS}=0.927$ ， $T_O^M=0.62\text{sec}$ ， $\text{PGA}=370.8\text{gal}$



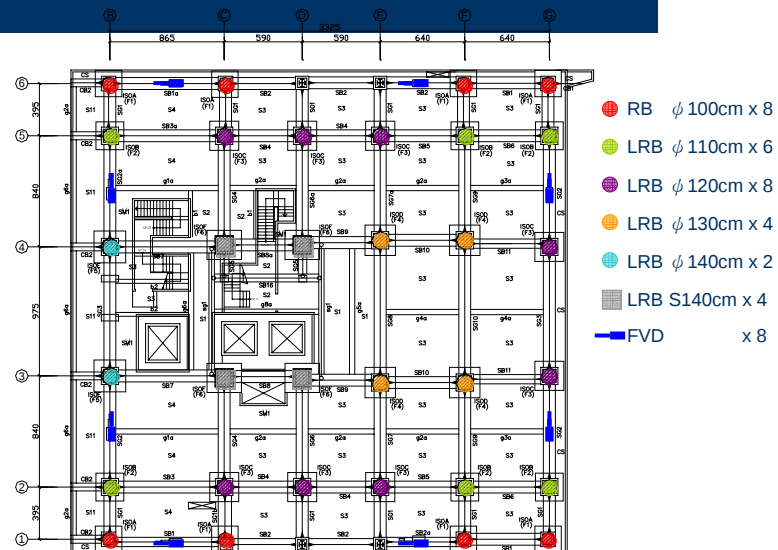
結構概述

三、標準層結構平面

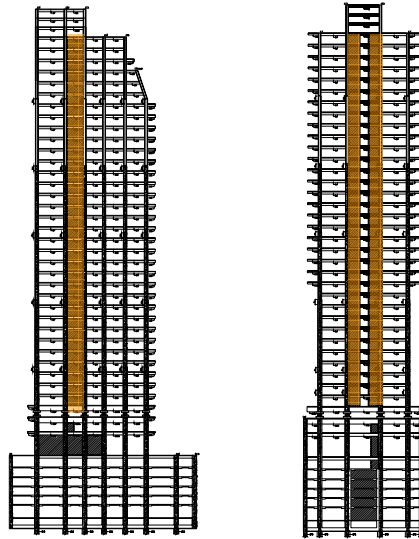


結構概述

四、隔震層結構平面

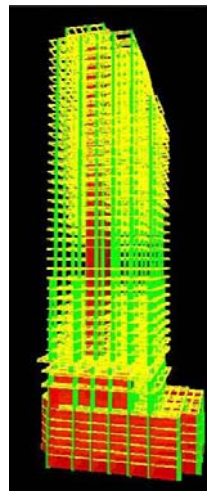
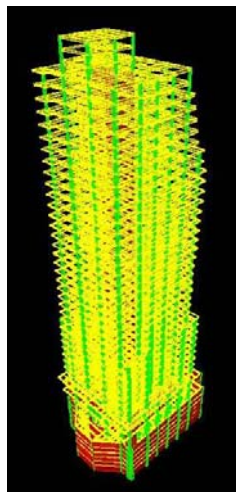


五、結構立面配置

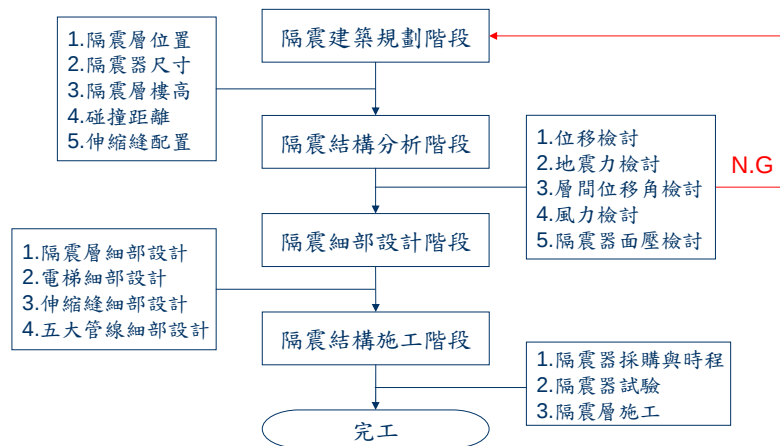


六、結構3D模擬

以ETABS建立結構模型並進行非線性歷時分析



隔震設計流程



隔震設計流程(建築規劃階段)

一、隔震層位置及配置

■ 隔震層位置規劃

- 適當的隔震層位置(本案採中間層隔震，設置於2F頂)
- 隔震層樓高檢討(本案採3.5m)

■ 隔震器配置規劃

- 隔震器的尺寸配置(共8組RB，24組LRB)
- 檢討隔震層剛心與質心之關係

■ 隔震碰撞距離檢討

- 初步分析碰撞距離最大需求(碰撞距離=55cm)
- 檢討建築配置之可行性
- 電梯、樓梯等設施檢討
- 伸縮縫配置檢討

隔震設計流程(建築規劃階段)

二、隔震層立面配置說明

■ 隔震器高度

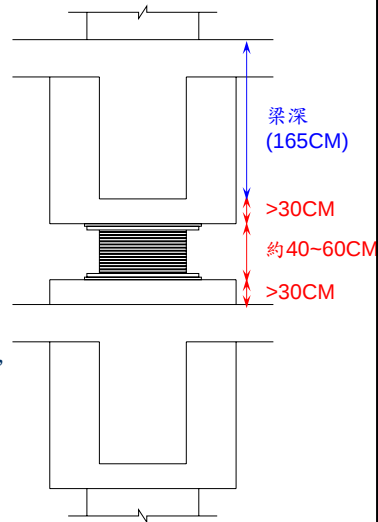
- 隔震器總高度約在40~60CM之間。

■ 隔震器基座

- 隔震器基座高度至少須大於25CM，為考量較佳的施工性，建議至少30CM以上。

■ 隔震層高度

- 由於隔震器及基座厚度至少需120CM，且本案隔震層上梁深165CM，下梁深180CM，故隔震層高度最少需285CM，本案規劃隔震層高度為350CM。

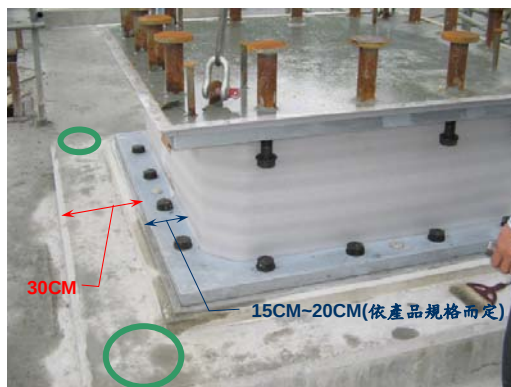


隔震設計流程(建築規劃階段)

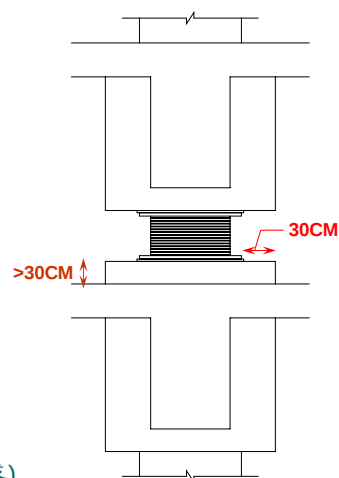
三、隔震基座尺寸說明

■ 隔震基座主要有兩個功能

- 提供隔震器足夠的錨碇強度
- 提供以後更換隔震器使用



提供以後千斤頂使用(四個角落)

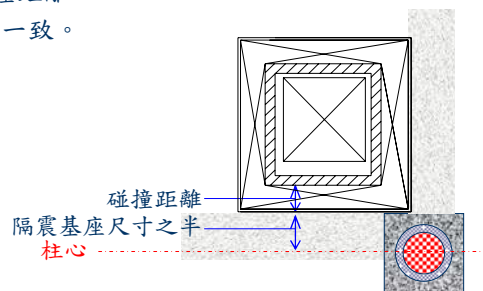


隔震設計流程(建築規劃階段)

四、電梯配置原則說明(1/2)

■ 電梯配置平面原則

- 電梯需避開隔震器基座並預留足夠的碰撞距離。
- 隔震建築規劃階段應避免於柱邊配置電梯間，且柱心與電梯外牆皮距離為隔震基座尺寸之半+碰撞距離。
- 隔震器的中心應與柱心一致。

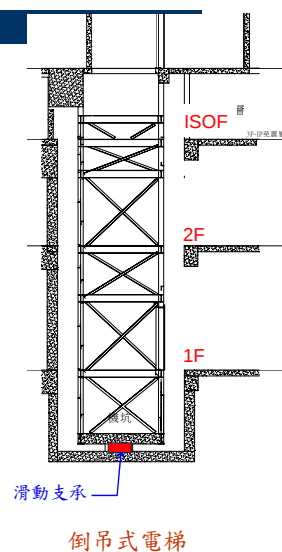


隔震設計流程(建築規劃階段)

四、電梯配置原則說明(2/2)

■ 電梯配置立面原則

- 由隔震層頂版往下倒吊，可採RC或鋼構方式配置，並於電梯機坑下方配置滑動支承。
- 倒吊電梯原則上不宜超過3個樓層。
- 若超過3個樓層則需配置可撓式電梯，但成本相對提高甚多。

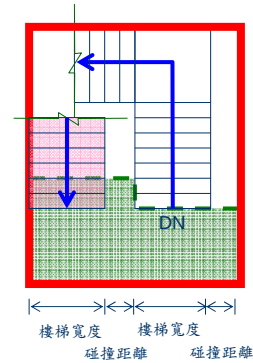


隔震設計流程(建築規劃階段)

五、樓梯配置原則說明(1/3)

■ 樓梯配置原則

- 樓梯分成兩部份，一為隔震層頂版倒吊部份，屬上部結構，一為固定在隔震層部分，屬下部結構。
- 兩部份需依照碰撞距離需求檢討平面配置。
- 隔震層樓梯空間配置應確保當地震發生時不至於影響行人動線為原則。

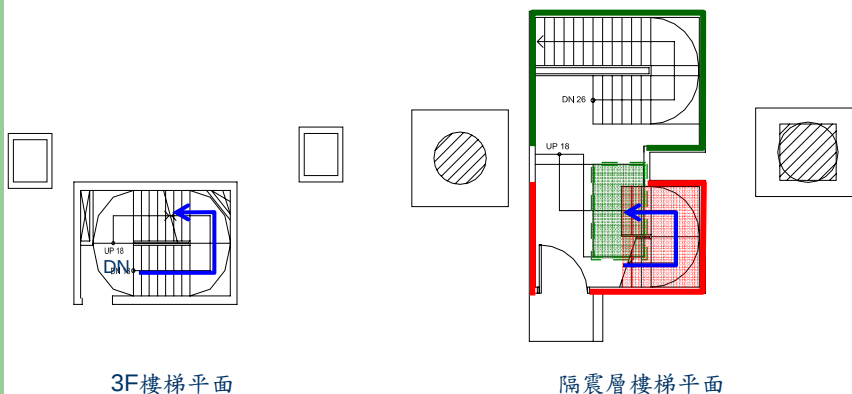


隔震層樓梯平面

隔震設計流程(建築規劃階段)

五、樓梯配置原則說明(2/3)

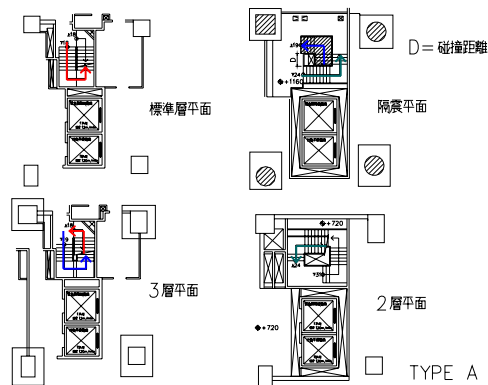
■ 本案設計範例



隔震設計流程(建築規劃階段)

五、樓梯配置原則說明(3/3)

■ 他案設計範例



隔震設計流程(結構分析階段)

一、結構分析內容

■ 動力反應譜分析

- 結構振態分析
- 設計地震力分析
- 最大考量地震力分析
- 設計總位移分析(碰撞距離需滿足分析成果)
- 最大總位移分析(碰撞距離需滿足分析成果)
- 設計地震下等效阻尼比
- 最大考量地震下等效阻尼比
- 層間位移角分析($\alpha_{MAX}=0.005$)

■ 風力分析

- 風力規範檢討
- 風洞試驗檢討

隔震設計流程(結構分析階段)

■ 非線性歷時分析

- 地震力分析
- 隔震層位移分析
- 層間位移角分析
- 樓層加速度分析

■ 隔震器性能檢核

- 隔震器最大剪應變
- 垂直力作用下長期面壓
- 垂直力與水平力共同作用下短期面壓

隔震設計流程(結構分析階段)

二、動力反應譜分析

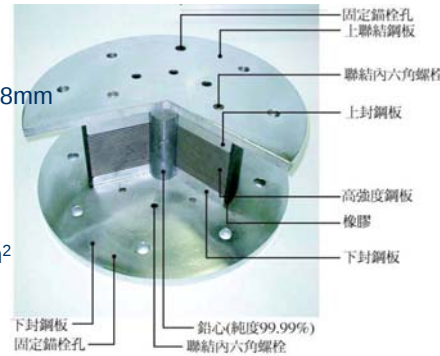
本案共配置6種隔震器

	橡膠材料	橡膠尺寸		鉛心尺寸	數量
		直徑(CM)	高度(CM)	直徑(CM)	
第一種隔震器	G4	100RB	24.8	-	8
第二種隔震器	G4	110	24.8	23	6
第三種隔震器	G4	120	24.8	23	8
第四種隔震器	G4	130	24.8	23	4
第五種隔震器	G4	140	24.8	23	2
第六種隔震器	G4	140方形	24.8	23	4
				共	32

隔震設計流程(結構分析階段)

■ LRB參數介紹(以本案所採用之第三種隔震器為例)

- 橡膠材質：G4
- 隔震器直徑： $D_i=120\text{cm}$
- 隔震器連結鋼板： $D_l=150\text{cm}$
- 鉛心直徑：23cm
- 橡膠厚度： $H=8\text{mm}\times 31=t_r\times n_r=248\text{mm}$
- 中間鋼板：4.4mm
- 降伏後勁度： $K_d=1796\text{kN/m}$
- 降伏前勁度： $K_u=23348\text{kN/m}$
- 特徵力： $Q_d=331.2\text{kN}$
- 隔震器斷面積： $A_r=1089426\text{mm}^2$
- 軸向勁度： $K_v=5696000\text{kN/m}$
- 一次形狀係數： $S_1=D_l/(4t_r)=37.5$
- 二次形狀係數： $S_2=D_l/H=4.8$

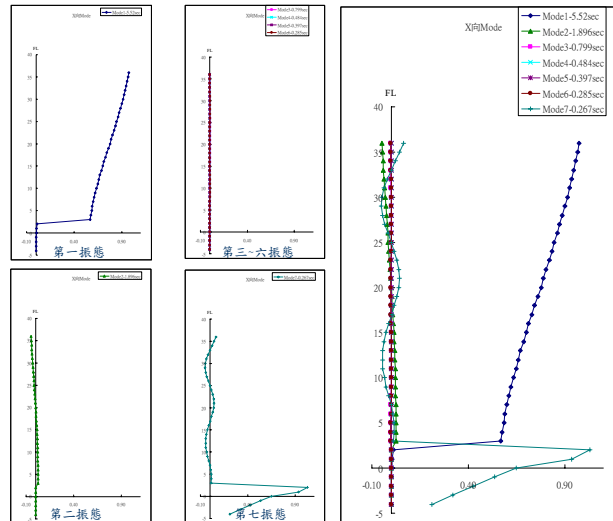


隔震設計流程(結構分析階段)

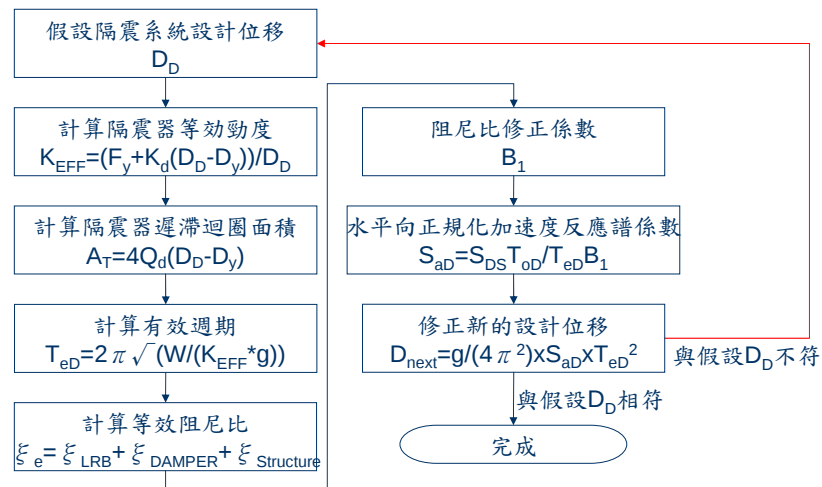
	第一種隔震器	第二種隔震器	第三種隔震器	第四種隔震器	第五種隔震器	第六種隔震器
LRB外徑(cm)	100RB	110	120	130	140	140S
鉛心TYPE	A	C	C	C	C	C
鉛心直徑(cm)	-	23	23	23	23	11.5
降伏後勁度 $K_d(t/m)=$	124	154	183	214	248	312
降伏前勁度 $K_u(t/m)=$		2006	2380	2786	3224	4052
特徵力 $Q(t)=$		33.8	33.8	33.8	33.8	33.8
隔震器總數 N=	8	6	8	4	2	4
降伏力 $F_y(t)=$	0	36.6	36.6	36.6	36.6	36.6
降伏位移 $D_y(m)=$	0	0.0182	0.0154	0.0131	0.0113	0.0090

隔震設計流程(結構分析階段)

振態分析



隔震層等效阻尼比分析流程



隔震設計流程(結構分析階段)

■ 地震力分析

(1)假定隔震系統設計位移

$$D_D = 0.2861\text{m}$$

(2)計算隔震器等效勁度 $K_{\text{eff}} = (F_y + K_d(D_D - D_y))/D_D = Q/D_D + k_d$

$$K_{\text{EFF}} = 8817\text{t/m}$$

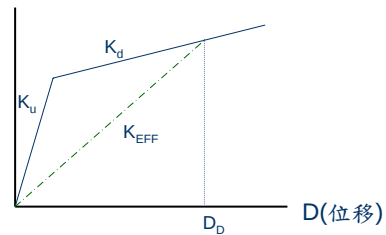
(3)計算隔震器遲滯迴圈面積 $A_T = 4Q(D_D - D_y)$

$$A_T = 880.87\text{t-m}$$

(4)計算有效週期 $T_{\text{ed}} = 2\pi \sqrt{W/(K_{\text{EFF}} \cdot g)}$

$$T_{\text{ed}} = 4.253\text{sec}$$

V(水平力)



水平力於水平位移關係圖

隔震設計流程(結構分析階段)

■ 地震力分析

(5)計算等效阻尼

$$\xi_c = \xi_{\text{LRB}} + \xi_{\text{DAMPER}} + \xi_{\text{Structure}}$$

$$\xi_{\text{LRB}} = A_T / (2\pi \times K_{\text{eff}} \times D_D^2)$$

$$\xi_{\text{LRB}} = 0.1943$$

$$\xi_{\text{DAMPER}} = \Sigma W_D / (2\pi \times K_{\text{eff}} \times D_D^2)$$

$$\text{單一方向} N = 4$$

$$F_D = C V^\alpha = 77.23\text{t}$$

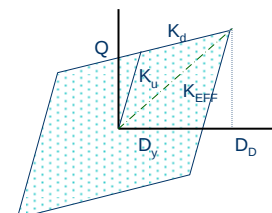
$$\Sigma W_D = N \lambda C \omega^\alpha D_D^{1+\alpha} = 324.77\text{t-m}$$

$$\xi_{\text{DAMPER}} = 0.0716$$

$$\xi_{\text{Structure}} = 0.03$$

$$\xi_c = 0.2959$$

結構體部分



遲滯迴圈等效阻尼計算

隔震設計流程(結構分析階段)

■ 地震力分析

(6)水平向正規化加速度反應譜係數 $S_{aD}=S_{DS}T_o^0/TeD$

$$S_{aD}= 0.103$$

(7)阻尼比修正係數 B_1

$$\xi_e= 0.2959$$

$$B_1= 1.625$$

(8)修正新的設計位移 $D_{next}=g/(4\pi^2)\times S_{aD}/B_1\times T_{eD}^2$

$$D_{next}= 0.2861m$$

與原假定位移 D_D 比較，若在一定誤差以上則將修正後的設計位移重新代入(2)~(8)進行計算
經過數次迭代求得設計地震力下的有效阻尼比及設計位移

隔震設計流程(結構分析階段)

■ 地震力分析

(9)修正結構反應譜

計算隔震層以外各層之水平層間勁度 K_i

結構固有阻尼比 $\xi=3\%$ (S.C結構)

隔震層在設計位移下之有效勁度 K_{EFF}

隔震層等效阻尼比 $\xi_e=29.59\%$

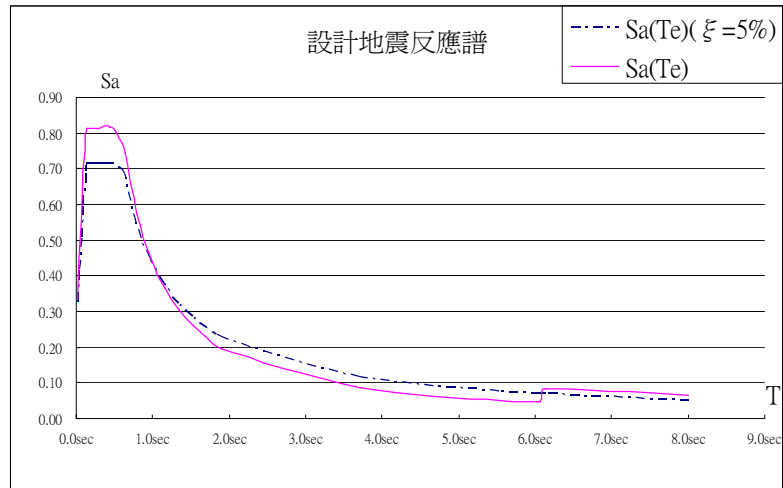
分別計算各振態下振態形狀 ϕ_{ij}

則各振態下之等效阻尼比為 $(\xi_e)_j=\Sigma K_i(\Delta \phi_{ij})^2 \xi / \Sigma K_i(\Delta \phi_{ij})^2$

	Te	Sa(Te)($\xi=5\%$)	ξ_e	BS	B1	Sa(Te)
第一振態	5.520sec	0.080	22.48%	1.5471	1.5322	0.052
第二振態	1.896sec	0.232	8.63%	1.2395	1.1814	0.196
第三振態	0.799sec	0.551	4.34%	0.9561	0.9561	0.576
第四振態	0.484sec	0.721	3.27%	0.8847	0.8847	0.815
第五振態	0.397sec	0.721	3.20%	0.8797	0.8797	0.820
第六振態	0.285sec	0.721	3.29%	0.8858	0.8858	0.814
第七振態	0.267sec	0.721	3.30%	0.8869	0.8869	0.813

隔震設計流程(結構分析階段)

反應譜分析



隔震設計流程(結構分析階段)

動力分析設計地震力豎向分配(1/3)

樓層	H_i	ΣH_i	W_i	ΣW_i	F_{xi}	ΣF_x
	m	m	t	t	t	t
PHF	2.70	135.80	103.76	103.76	23.05	23.05
P3F	2.70	133.10	142.87	246.62	14.95	38.00
R2F	3.00	130.40	143.34	389.96	17.42	55.42
R1F	3.40	127.40	680.63	1070.59	85.99	141.41
35F	3.40	124.00	734.54	1805.13	93.00	234.41
34F	3.40	120.60	777.83	2582.96	86.31	320.72
33F	3.40	117.20	793.47	3376.43	79.67	400.39
32F	3.40	113.80	812.17	4188.60	76.93	477.32
31F	3.40	110.40	839.17	5027.76	76.65	553.97
30F	3.40	107.00	866.97	5894.73	77.12	631.09
29F	3.40	103.60	910.51	6805.24	78.60	709.69
28F	3.40	100.20	921.92	7727.16	76.65	786.34
27F	3.40	96.80	991.42	8718.58	77.94	864.28
26F	3.40	93.40	995.41	9713.99	72.45	936.73
25F	3.40	90.00	1004.88	10718.87	66.85	1003.58

隔震設計流程(結構分析階段)

■ 動力分析設計地震力豎向分配(2/3)

樓層	H _i	Σ H _i	W _i	Σ W _i	F _{xi}	Σ F _x
	m	m	t	t	t	t
24F	3.40	86.60	1044.08	11762.94	65.78	1069.36
23F	3.40	83.20	1077.00	12839.94	67.11	1136.47
22F	3.40	79.80	1073.22	13913.16	65.91	1202.38
21F	3.40	76.40	1079.29	14992.45	65.26	1267.64
20F	3.40	73.00	1080.33	16072.78	63.71	1331.35
19F	3.40	69.60	1080.33	17153.10	61.80	1393.15
18F	3.40	66.20	1103.75	18256.85	61.86	1455.01
17F	3.40	62.80	1128.21	19385.06	61.80	1516.81
16F	3.40	59.40	1121.92	20506.98	58.96	1575.77
15F	3.40	56.00	1121.92	21628.90	56.59	1632.36
14F	3.40	52.60	1130.41	22759.31	56.02	1688.38
13F	3.40	49.20	1139.19	23898.50	59.33	1747.71
12F	3.40	45.80	1159.83	25058.33	61.49	1809.20
11F	3.40	42.40	1329.31	26387.64	71.31	1880.51
10F	3.40	39.00	1348.21	27735.84	73.67	1954.18

隔震設計流程(結構分析階段)

■ 動力分析設計地震力豎向分配(3/3)

樓層	H _i	Σ H _i	W _i	Σ W _i	F _{xi}	Σ F _x
	m	m	t	t	t	t
9F	3.40	35.60	1346.57	29082.41	73.39	2027.57
8F	3.40	32.20	1348.20	30430.61	72.30	2099.87
7F	3.40	28.80	1348.20	31778.81	70.50	2170.37
6F	3.40	25.40	1393.23	33172.04	72.03	2242.40
5F	3.40	22.00	1430.05	34602.09	74.47	2316.87
4F	3.40	18.60	1430.05	36032.14	75.60	2392.47
3F	3.50	15.20	2933.28	38965.42	195.90	2588.37
ISO	4.50	11.70	3532.72	42498.14	5208.78	7797.15
2F	7.20	7.20	2917.87	45416.00	5455.98	13253.13

■ 隔震層上部結構設計橫力 $V=0.066W$

■ 基面設計橫力 $V=0.292W/0.8=0.365W$

隔震設計流程(結構分析階段)

- 另考量製程、溫度、老化等現象所造成隔震器設計參數的偏差進行分析

- 溫度偏差假設變化範圍在0°C~40°C之間
- 老化偏差則以60年的生命週期為基準

- 隔震器溫度偏差： $K_d(t) = K_d(t_0) \times \exp(-0.00271(t-t_0))$
 $Q_d(t) = Q_d(t_0) \times \exp(-0.00879(t-t_0))$
 t_0 ：基準溫度(20°C)

	Kd	Qd
製程偏差	15.0%	15.0%
	-15.0%	-15.0%
溫度偏差	5.6%	19.2%
	-5.3%	-16.1%
老化偏差(60年)	10.0%	5.0%
偏差合計	30.6%	39.2%
	-20.3%	-31.1%

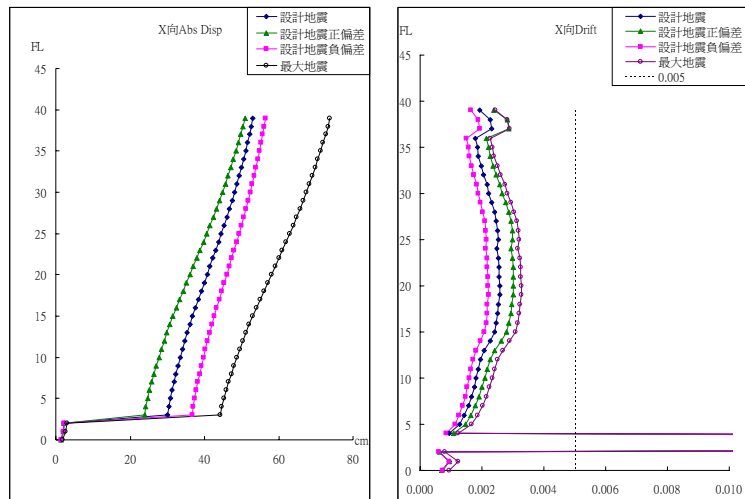
隔震設計流程(結構分析階段)

- 結構分析成果

	原設計參數	設計參數 最大偏差	設計參數 最小偏差		原設計參數
設計地震力 (隔震層上部)	0.066W	0.075W	0.059W	最大地震力 (隔震層上部)	0.086W
設計地震力 (基面)	0.365W	0.359W	0.368W	最大地震力 (基面)	0.473W
隔震層 設計位移	27.8cm	21.6cm	34.4cm	隔震層 最大位移	41.4cm
設計地震 層間位移角	0.00257	0.00302	0.00221	最大地震 層間位移角	0.00328
隔震層 阻尼比	29.59%	31.57%	26.88%	隔震層 阻尼比	24.29%
第一振態 等效阻尼比	22.48%	21.28%	22.63%	第一振態 等效阻尼比	19.03%

隔震設計流程(結構分析階段)

反應譜分析成果



隔震設計流程(結構分析階段)

二、風力分析

- 風力作用下結構週期 $T_x=3.31\text{sec}$ ， $T_y=3.35\text{sec}$
- 風力規範
 - 隔震層以上設計風力 $W_x=780.9\text{t}$ ， $W_y=908.6\text{t}$
 - 風力規範35F角隅加速度尖峰值 3.8gal
- 風洞試驗
 - 最大風力 $W_{\text{Max}}=873.0\text{t}$
 - 35F角隅加速度尖峰值 2.4gal
- 風力作用下隔震層位移 $D_w=1.38\text{cm}$ ($\gamma=5.56\%$)
- 隔震層鉛心降服位移 $D_y=1.82\text{cm}$ ($\gamma=7.34\%$)

隔震設計流程(結構分析階段)

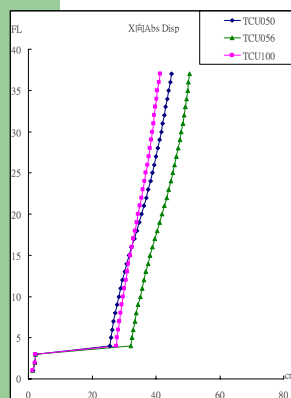
三、非線性歷時分析

- 選擇工址附近三組測站：TCU050(西屯國小)，TCU056(黎明國小)，TCU100(永安國小)
- 將水平地震紀錄調整至5%阻尼反應譜
- 依歷時分析成果檢核隔震器性能

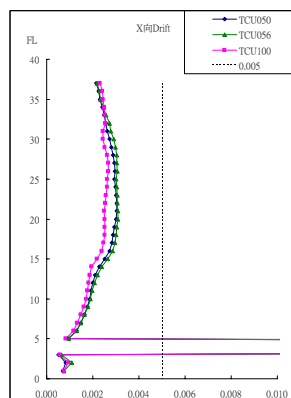
	TCU050	TCU056	TCU100
最大地震力 (隔震層上部)	0.057W	0.069W	0.062W
隔震層最大位移	23.36cm	29.9cm	25.31cm
最大地震層間位移角	0.00303	0.00310	0.00269
PGA	394gal	412gal	433gal
隔震層加速度	1410gal(3.58)	1682gal(4.09)	1485gal(3.43)
3F加速度	180gal(1/2.19)	163gal(1/2.53)	203gal(1/2.13)
35F加速度	248gal(1/1.59)	248gal(1/1.66)	285gal(1/1.52)

隔震設計流程(結構分析階段)

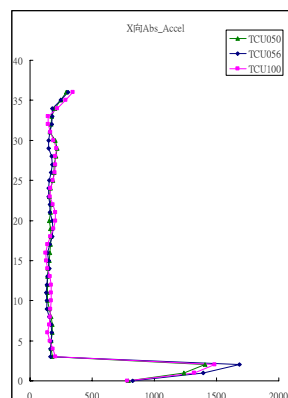
■ 歷時分析成果



歷時分析位移結果



歷時分析層間位移角結果

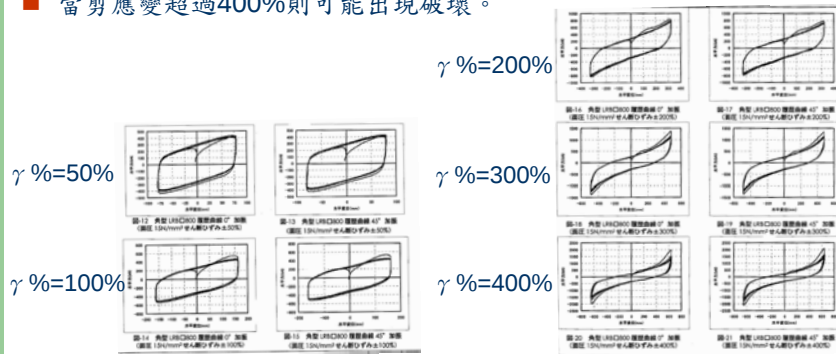


歷時分析樓層加速度結果

隔震設計流程(結構分析階段)

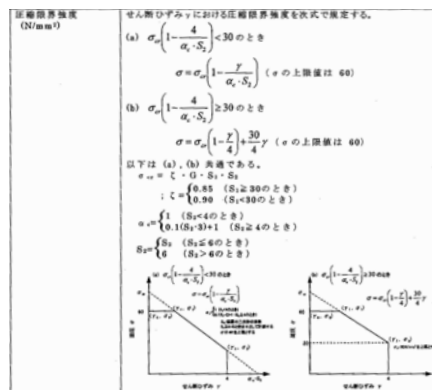
四、隔震器性能檢核

- 隔震器最大剪應變不得超過250%，本案橡膠厚度 $H=24.8\text{cm}$ ，故隔震層最大位移不得超過62cm。
- 當剪應變超過250%雖不致發生材料破壞的情況，但將出現橡膠硬化的情況，使隔震器無法提供飽滿的遲滯迴圈。
- 當剪應變超過400%則可能出現破壞。



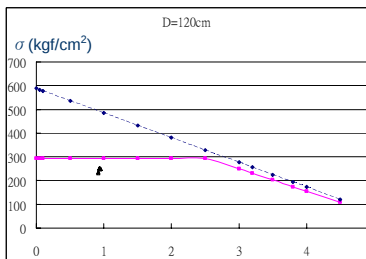
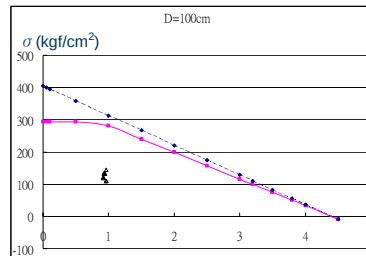
隔震設計流程(結構分析階段)

- 垂直力作用下(DL+0.8LL)長期面壓應小於材料強度(本案 $\sigma_{\max}=150\text{kg/cm}^2$)
- 垂直力與水平力共同作用下(DL+0.5LL+E, DL+E)短期面壓 ($\sigma(\gamma)$)：隨著剪應變增加而變小，且不得大於300kg/cm²
- 拉面壓不得大於10kg/cm²



隔震設計流程(結構分析階段)

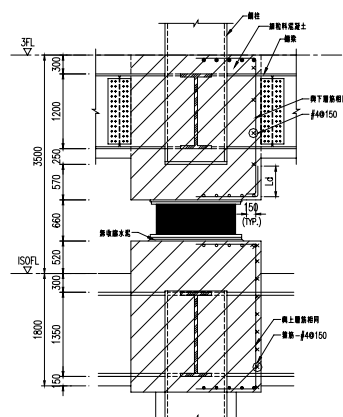
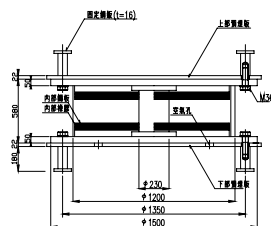
Link	直徑 (cm)	面壓 σ (kgf/cm ²)		
		長期載重	短期載重	
L1	100RB	55.48	109.33	-2.78
L2	100RB	79.68	145.30	7.62
L3	100RB	73.18	132.27	7.55
L4	100RB	60.52	121.93	-6.63
L5	110	141.31	245.22	25.19
L6	120	129.59	249.37	-0.93
L7	120	144.48	249.69	27.11
L8	120	142.71	251.63	20.94
L9	110	130.54	203.35	44.81
L10	110	127.07	220.48	21.31
L11	140	134.53	223.00	35.82
L12	140S	136.98	269.22	-7.25
L13	140S	137.36	240.99	21.30
L14	130	129.84	195.13	50.85
L15	130	137.04	202.53	56.59
L16	120	145.44	235.01	41.01



隔震設計流程(結構細設階段)

一、隔震層細部設計

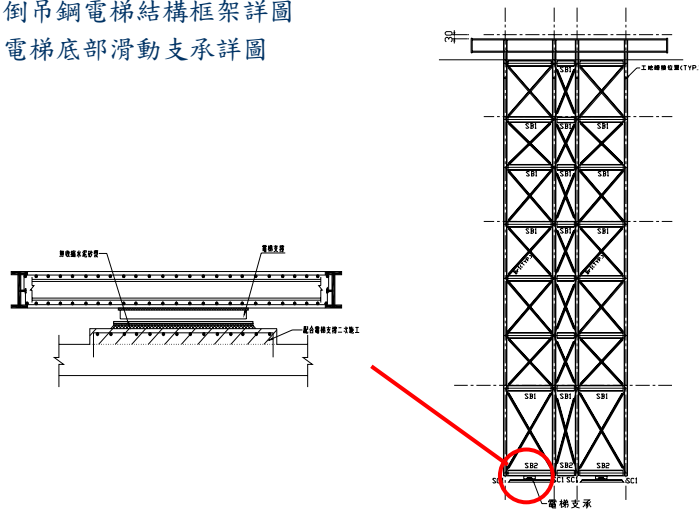
- 隔震器尺寸及規格詳圖設計
- 隔震基座詳圖設計
- 隔震層大梁配筋設計
- 隔震層樓梯詳圖設計



隔震設計流程(結構細設階段)

二、電梯細部設計

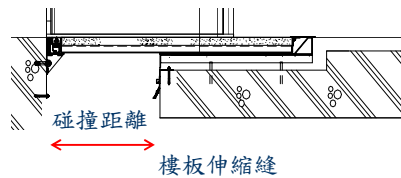
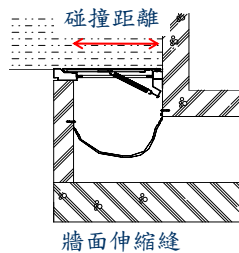
- 倒吊鋼電梯結構框架詳圖
- 電梯底部滑動支承詳圖



隔震設計流程(結構細設階段)

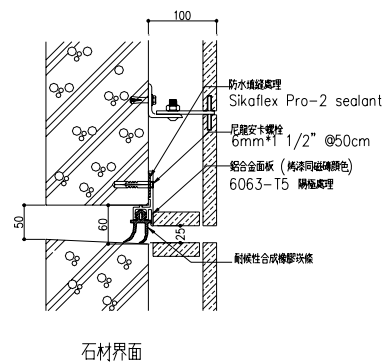
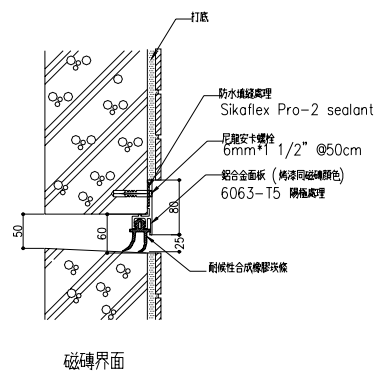
三、伸縮縫細部設計

- 樓板伸縮縫詳圖
- 電梯出入口伸縮縫詳圖
- 天花伸縮縫詳圖



隔震設計流程(結構細設階段)

■ 牆面伸縮縫詳圖(外牆、防火區隔需求)



四、五大管線細部設計

- 壓力式可撓管(給水、消防)
- 重力式可撓管(排水)
- 瓦斯可撓管



隔震設計流程(結構施工階段)

三、隔震層施工步驟(2/5)



墩座鋼筋綁紮



下錨錠板吊裝



下錨錠板高程測量



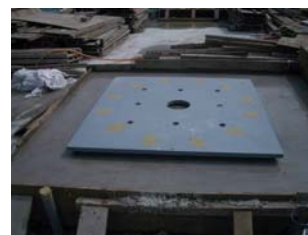
下錨錠板水平位置測量

隔震設計流程(結構施工階段)

三、隔震層施工步驟(3/5)



下墩座組模



高性能混凝土澆置



無收縮灌注組模



無收縮灌注完成

隔震設計流程(結構施工階段)

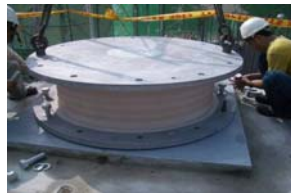
三、隔震層施工步驟(4/5)



進場檢測



隔震器吊裝



隔震器吊裝



隔震器吊裝

隔震設計流程(結構施工階段)

三、隔震層施工步驟(5/5)



螺栓鎖固



螺栓扭力確認



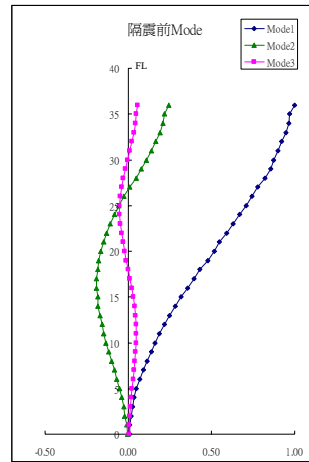
水平確認



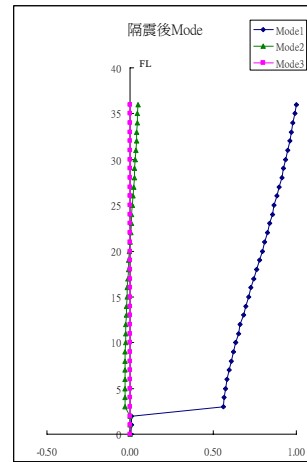
鐵板保護(完成)

隔震前後結構週期差異

Mode Shape



隔震前($T_1=2.917\text{sec}$)

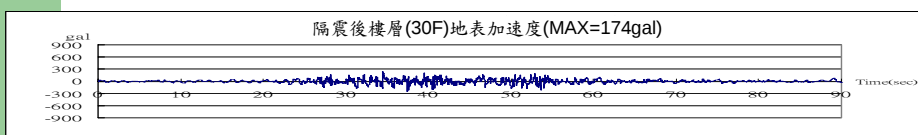
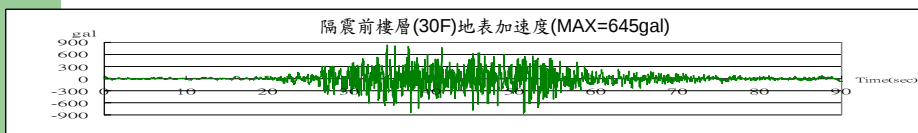
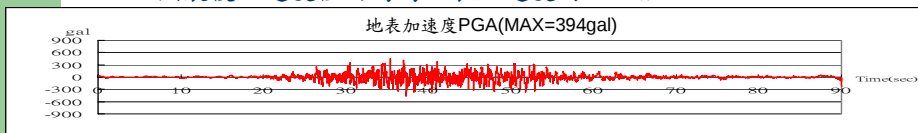


隔震後($T_1=5.520\text{sec}$)

隔震前後樓層加速度差異

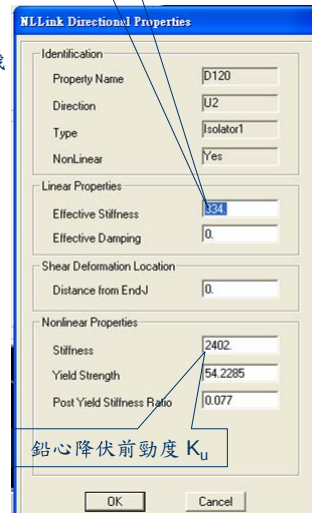
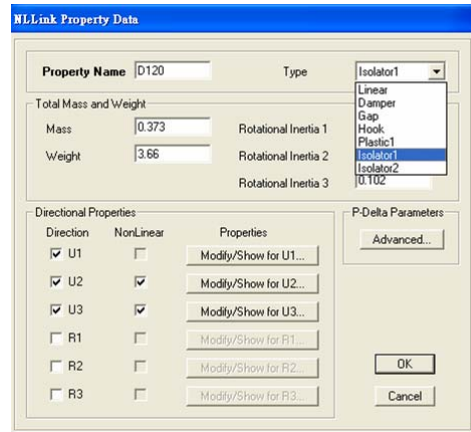
假設除了隔震器以外的結構構件均為彈線性。

- 隔震前加速度約放大為地表加速度的1.64倍。
- 隔震後加速度僅約為地表加速度的0.44倍。



隔震器參數模擬應用

- 隔震器參數於Etabs中的應用方式
 - 選擇NLLink中的Isolator1進行參數定義

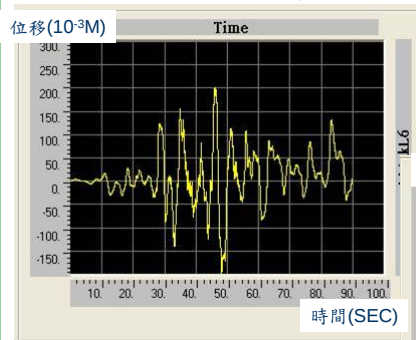


有效勁度 K_{EFF}

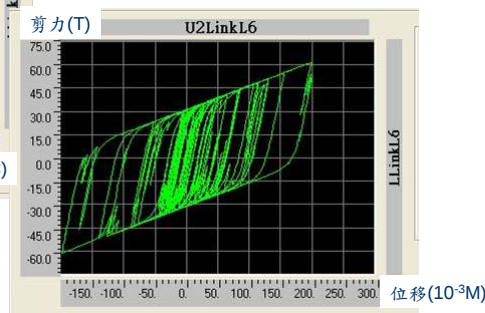
鉛心降伏前勁度 K_U

隔震器模擬應用

- 隔震器於歷時分析下之成果



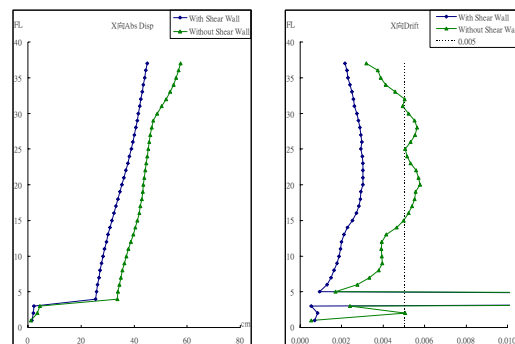
隔震器歷時VS位移



隔震器遲滯迴圈

隔震結構勁度之影響

- 耐震規範中隔震設計地震力的計算是建立在剛體運動的基礎上。
- 考慮結構本身的柔性將會造成等效阻尼比減小。
- 本案未考慮結構本身的柔性時，設計地震下等效阻尼比為29.59%，考慮結構柔性後將等效阻尼比修正為22.48%。
- 假設本案例中所有RC剪力牆均取消的情況下則等效阻尼比修正為17.72%。



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