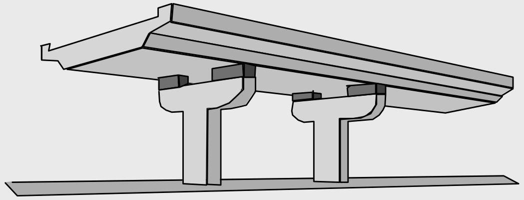
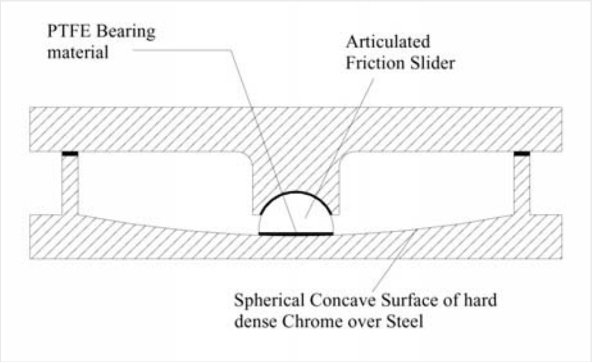
Chapter 7
Seismic Isolation of Bridges
(No. 3)



2006 Course
Kazuhiko Kawashima
Tokyo Institute of Technology

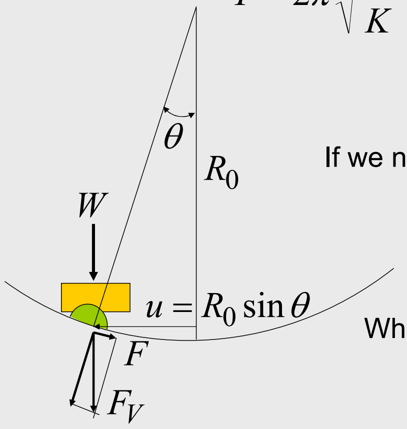
Friction Pendulum System

Friction Pendulum System



After Professor M. Calvi

FPS



$$T = 2\pi\sqrt{\frac{M}{K}} \qquad M = \frac{W}{g} = \frac{F_V}{g}$$

$$F = Ku$$

If we neglect the friction force

$$F = F_V \sin\theta$$

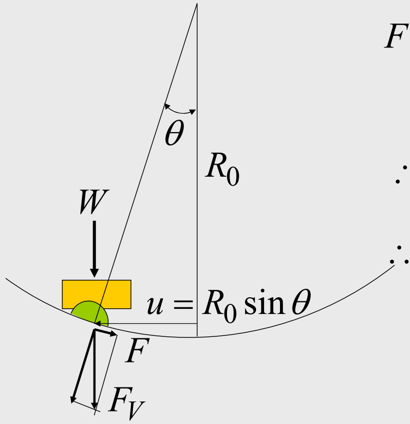
$$u = R_0 \sin\theta$$

When θ is small,

$$F \approx F_V\theta$$

$$u \approx R_0\theta$$

FPS



$$F = Ku$$

$$F \approx F_V \theta$$

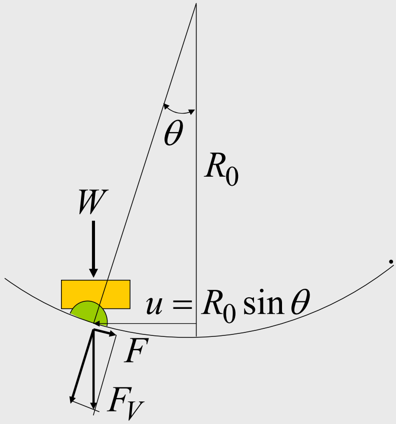
$$u \approx R_0 \theta$$

$\therefore$

$$F_V \theta = K R_0 \theta$$

$\therefore$

$$K = \frac{F_V}{R_0}$$



$$T = 2\pi\sqrt{\frac{M}{K}}$$

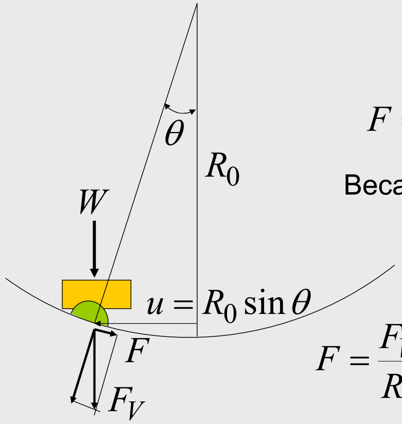
$$M = \frac{W}{g} = \frac{F_V}{g}$$

$$K = \frac{F_V}{R_0}$$

$\therefore$

$$T = 2\pi\sqrt{\frac{R_0}{g}}$$

Lateral Force vs. Lateral Displacement Hysteresis



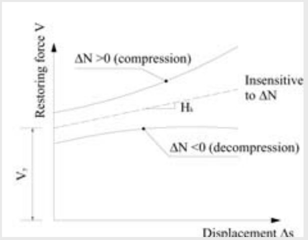
$$F = F_V \sin\theta$$

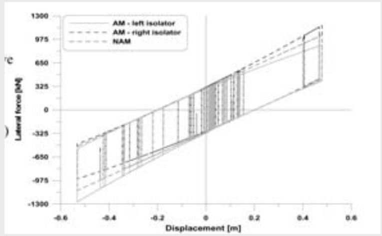
Because,

$$\sin\theta = \frac{u}{R_0}$$

$$F = \frac{F_V}{R_0} u + sign(\dot{u})\mu F_V$$

$$F = \frac{F_V}{R_0} u + sign(\dot{u})\mu F_V$$



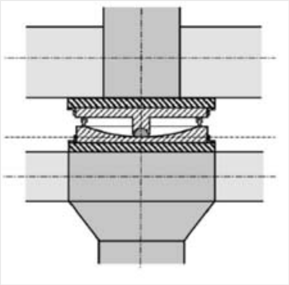


After Professor M. Calvi

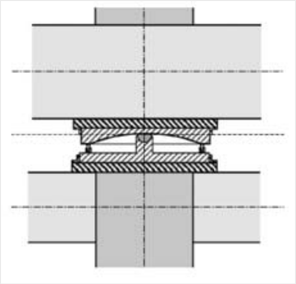
Characteristics of FPS

- ●FPS is governed by 2 parameters
  - ✓Friction coefficient at the sliding interface
  - ✓Radius of the spherical surface
- ●Neglecting variation of the friction coefficient with velocity and pressure that slightly effect the peak response of the system, the only one parameter is the radius of the spherical surface
- ●Residual displacements are reduced due to the self-centering action induced by the concave spherical surface.

Upward mounting



Downward mounting



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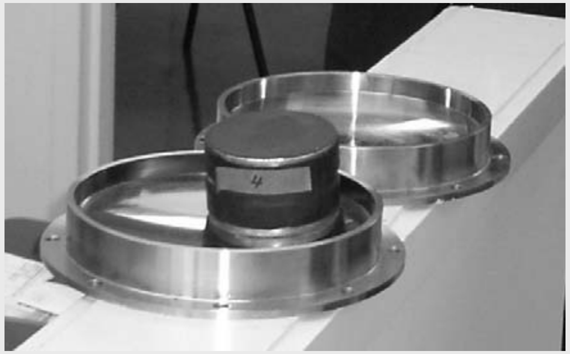
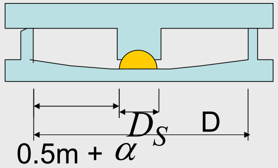
FPS Being Developed at University of California, Berkeley



Courtesy of Prof. Steven Mahin, UCB

Problems for Implementation

- ●Large diameter FPS needed to accommodate +/- 0.5 m displacement

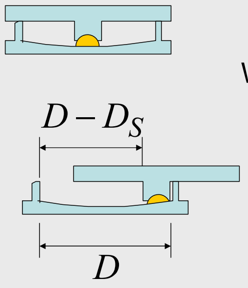


If we consider $\alpha = 0.15m$ and $D_S = 0.2m$

$$D = 2(0.5 + \alpha) + D_S$$
$$= 1.5m$$

Consequently, the out-diameter of the FPS is nearly 1.8m

In fact, much large diameter is needed



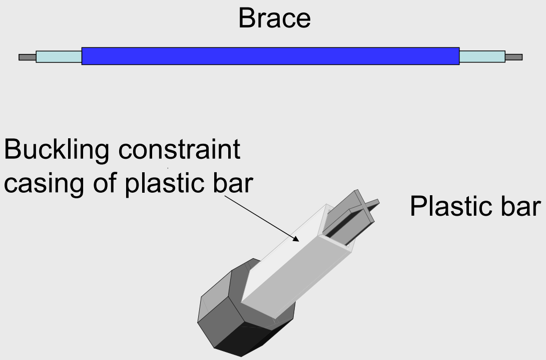
Width of the upper steel is

$$D_U = 2(D-D_S)+D_S$$
$$= 2D-D_S$$

If D=1.5m, D_U becomes 2.8m

Damper Braces

Damper Brace

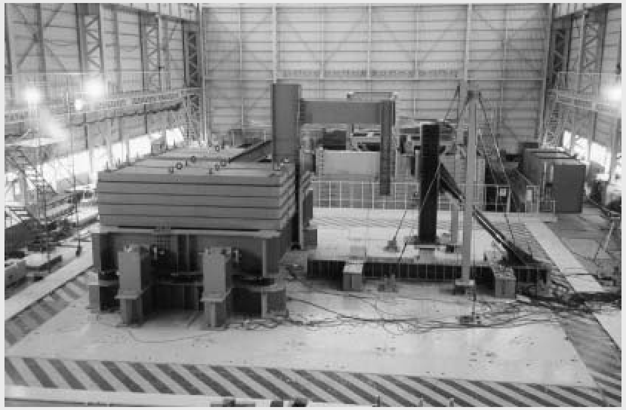


Morishita et al (2004)

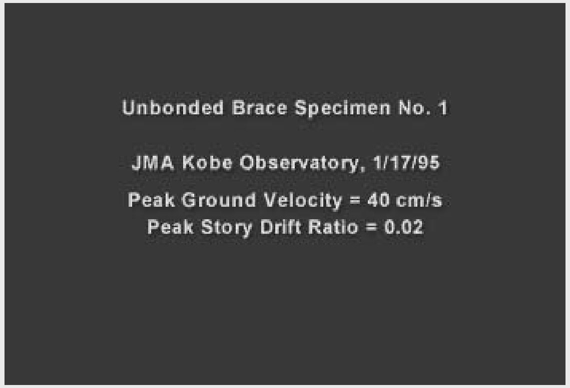
Unbond Brace Dampers widely used for Buildings

Courtesy of Shin-Nippon Steel Co. Ltd

Shake Table Test for Effectiveness of Unbonded Brace Damper



Courtesy of Shin-Nippon Steel Co. Ltd



Unbonded Brace Specimen No. 1

JMA Kobe Observatory, 1/17/95

Peak Ground Velocity = 40 cm/s
Peak Story Drift Ratio = 0.02

Courtesy of Shin-Nippon Steel Co. Ltd

Damper Brace

Morishita et al (2004)

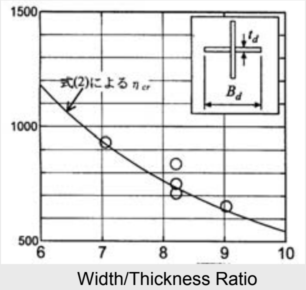
Energy Dissipation of Unbond Damper Braces

- Width-thickness ratio

$$r = \frac{B_d}{2t_d}$$

- Accumulated plastic deformation

$$\eta = \sum_i \frac{\delta_{pi}}{\delta_y}$$



where,

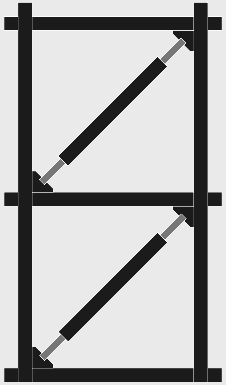
δ_{pi} : plastic displacement of i-th loading

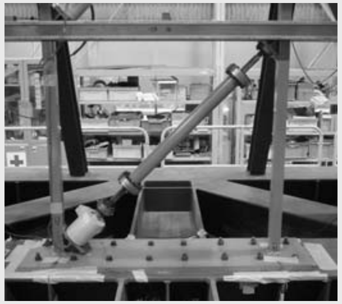
δ_y : yield displacement

$$\eta_{cr} = 17{,}800 r^{-1.52}$$

Morishita et al (2004)

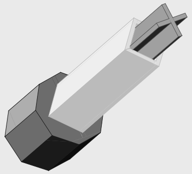
Shake Table Verification for Critical Plastic Deformation

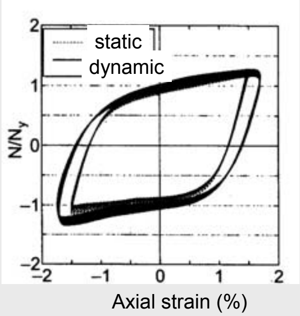




Morishita et al (2004)

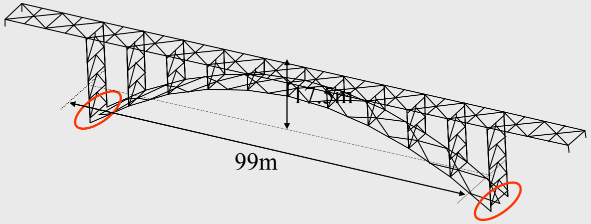
Hysteretic Energy Dissipation by Damper Brace



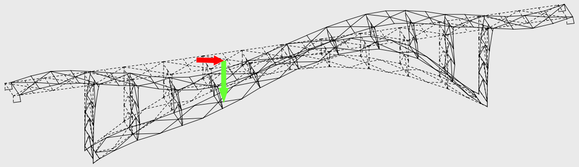


Morishita et al (2004)

Application of Damper Braces for Seismic Retrofit of an Arch Bridge

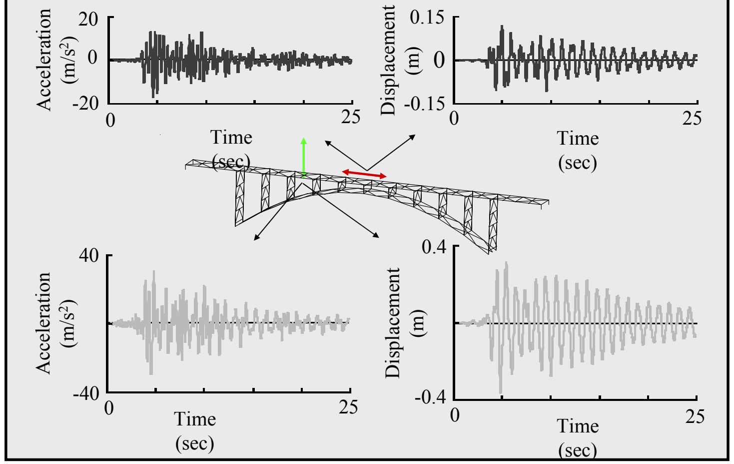


Coupling of Longitudinal and Vertical Modes

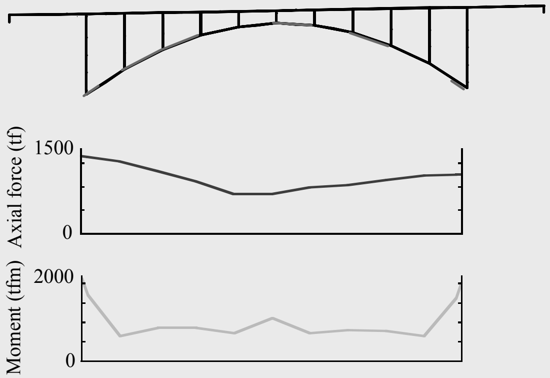


Fundamental Natural Period T=1.1 sec

Seismic Response of As-Built Bridge under JMA Kobe Ground Acceleration, 1995 Kobe EQ



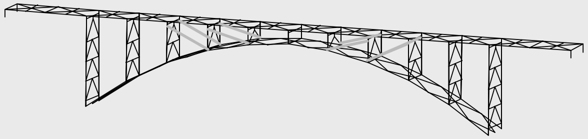
Arch Members which undergo Inelastic Deformation



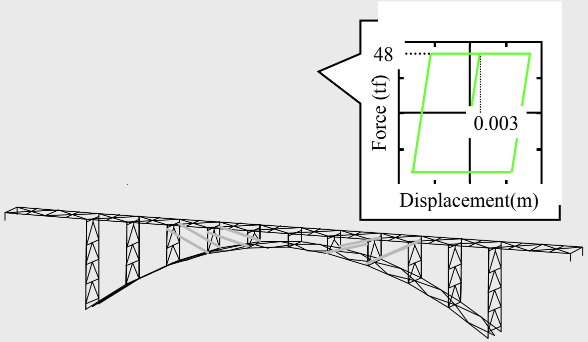
Location of Damper Braces

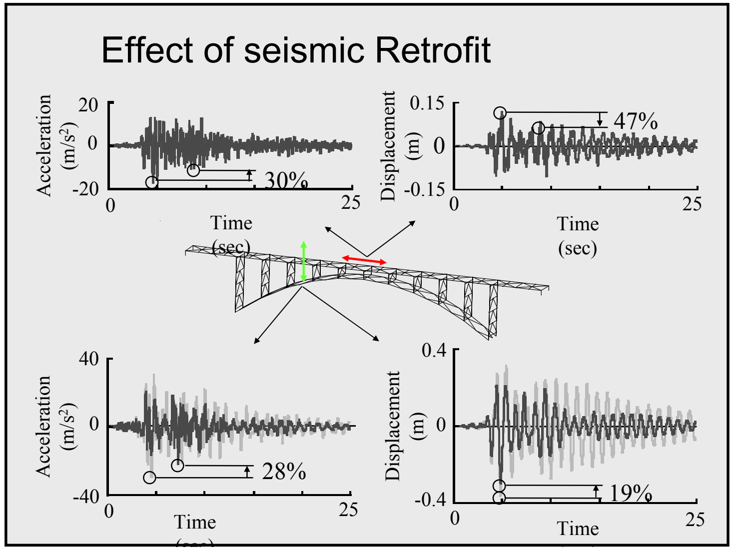
Criteria for selecting members where Damper braces are installed

$\varepsilon > \varepsilon_a = 1/100$	Axial strain between 2 nodes
$\Delta u > \Delta u_a = 1/10m$	Relative displacement between 2 nodes
$l_D < 12m$	Length of member

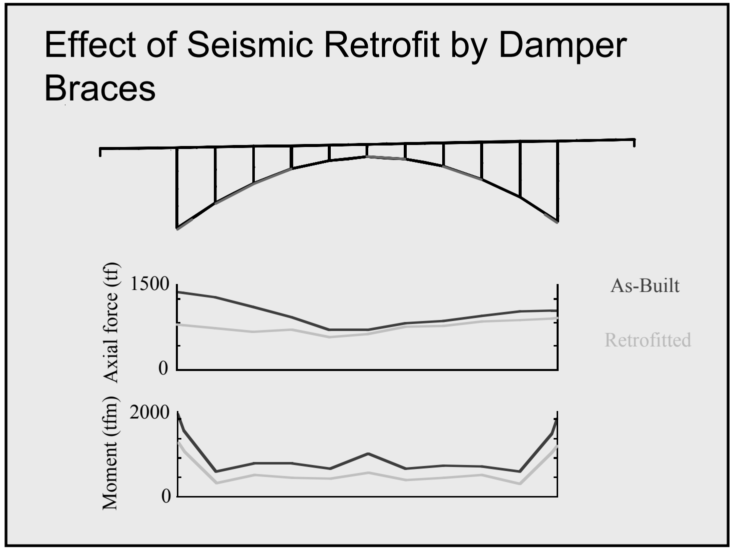


Analytical Idealization of Hysteretic Behavior of Damper Brace

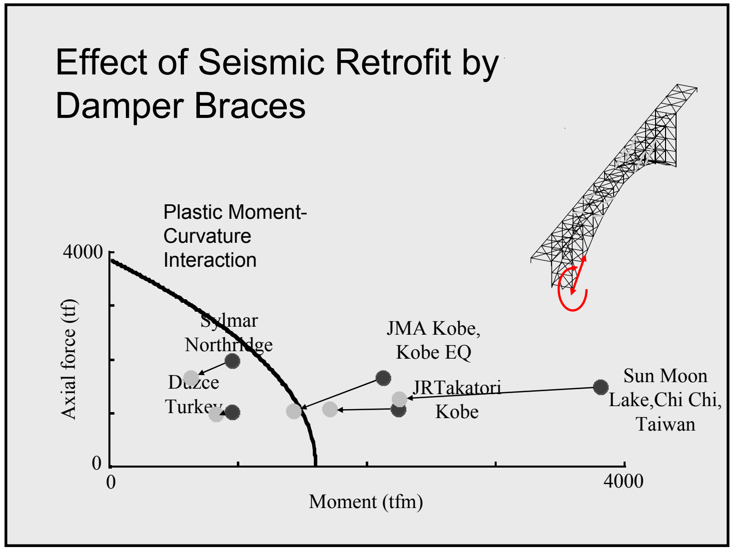




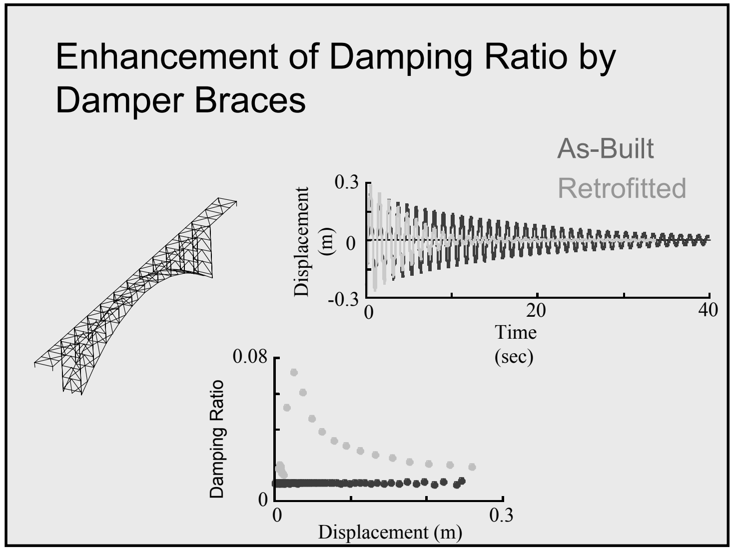
Effect of seismic Retrofit



Effect of Seismic Retrofit by Damper Braces



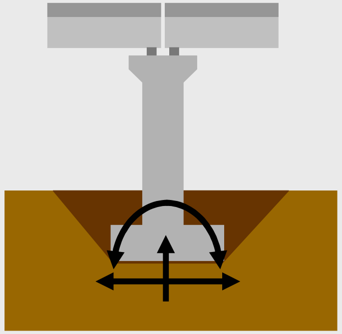
Effect of Seismic Retrofit by Damper Braces



Enhancement of Damping Ratio by Damper Braces

Rocking Seismic Isolation of Bridges

Requirements of Foundations in Seismic Design



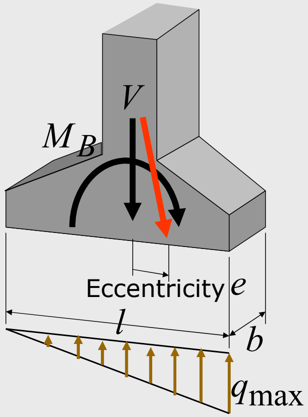
Static Seismic Design

- Bearing capacity
- Sliding
- Rocking

Dynamic Response

- Sliding + Rocking
- Rocking + Jump

Requirements for Rocking Response in the Static Design



Eccentricity

$$e = \frac{M_B}{V} < e_a$$

Allowable Eccentricity

$$e_a = \begin{cases} l/3 & \text{— Static} \\ l/6 & \text{— Static +Seismic} \end{cases}$$

Bearing Capacity

$$q_{max} = \frac{V}{lb} + \frac{6M_B}{lb^2} < q_a$$

Akashi Straight Bridge
The World Longest Bridge

In the static design, overturning was the major factor for sizing those foundations

Is it true that such a large foundation overturns under a seismic excitation??

- Mass
- Natural period
- Frequency content of a ground motion

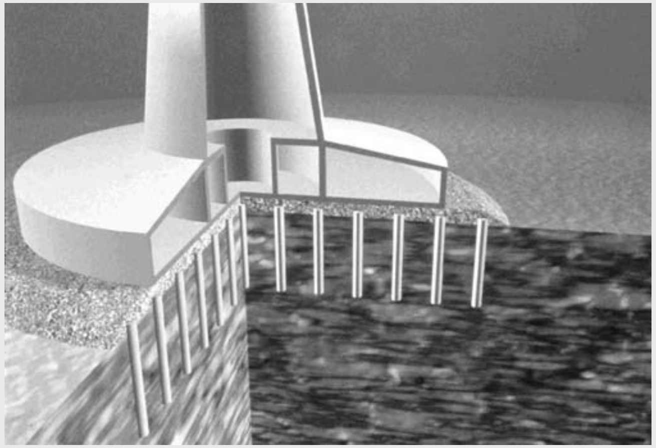
Overturning of Foundation-Tower System of Akashi Straight Bridge

- Static Analysis on Overturning of Foundation-Tower System was eliminated from seismic design
- It was decided that the static overturning analysis is unrealistic
- Decision of design was made based on nonlinear dynamic response analysis and a preliminary static design based on critical velocity which results in overturning

Seismic Rocking Isolation

Rion Antirion Bridge, Greece

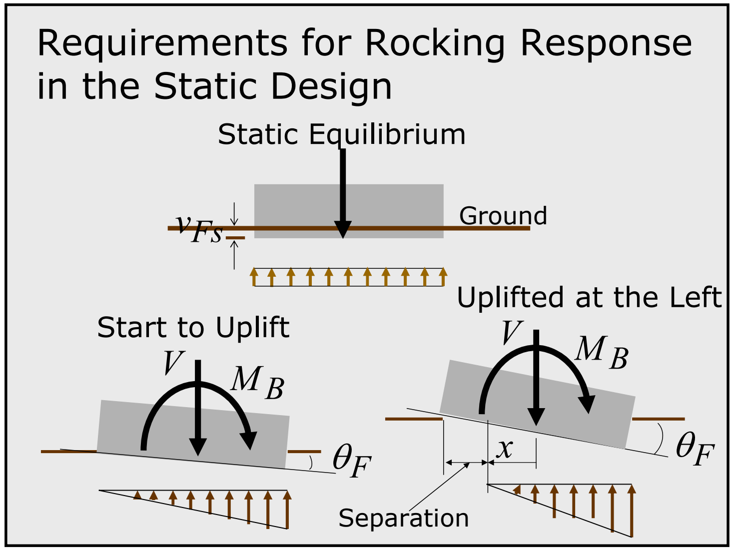
Courtesy of Dr. Alain Pecker



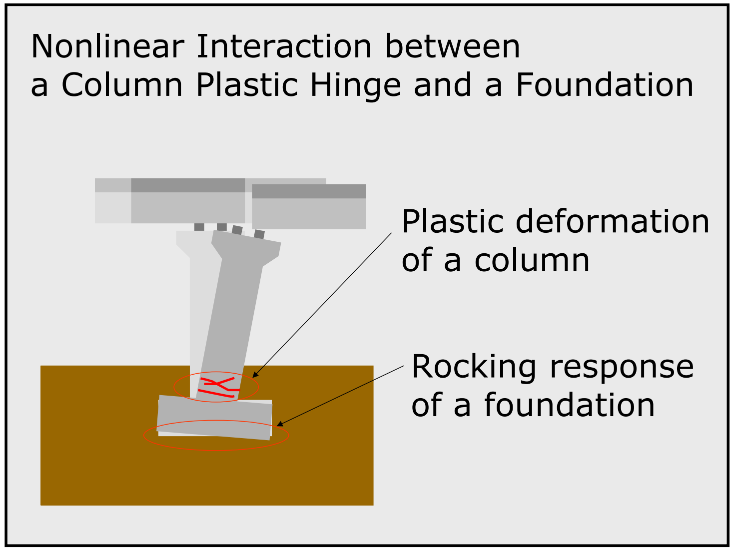
Concept of Rocking Isolation of Rion Anti-Rion Bridge

- ●Fault dislocation as large as 2 m is anticipated although the location of fault is not known.
- ●Rocking isolation reduces bridge response.

Rion Antirion Bridge

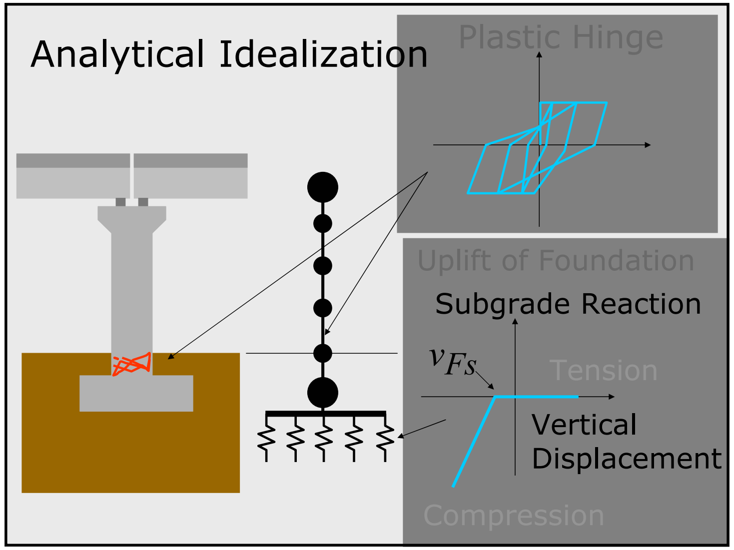


Requirements for Rocking Response in the Static Design



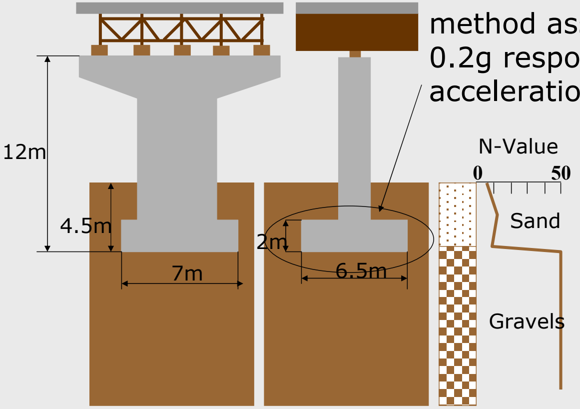
Nonlinear Interaction between a Column Plastic Hinge and a Foundation

Analytical Idealization



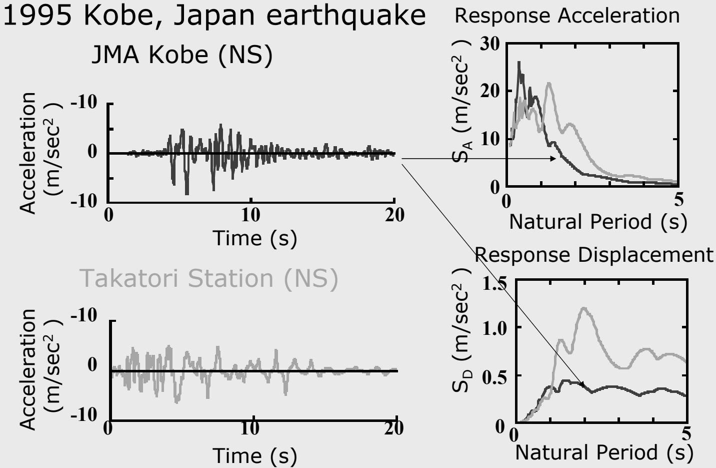
Bridge Analyzed

Designed based on the static design method assuming 0.2g response acceleration

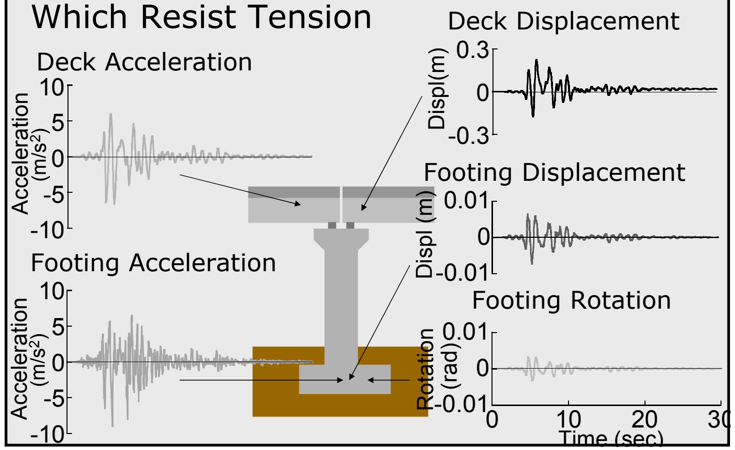


Ground Accelerations Subjected to Bridge

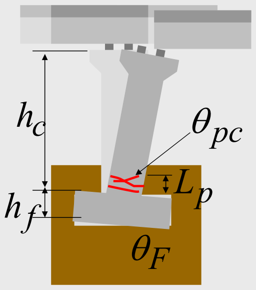
1995 Kobe, Japan earthquake



Seismic Response of the Bridge When Supported by Soil Springs Which Resist Tension

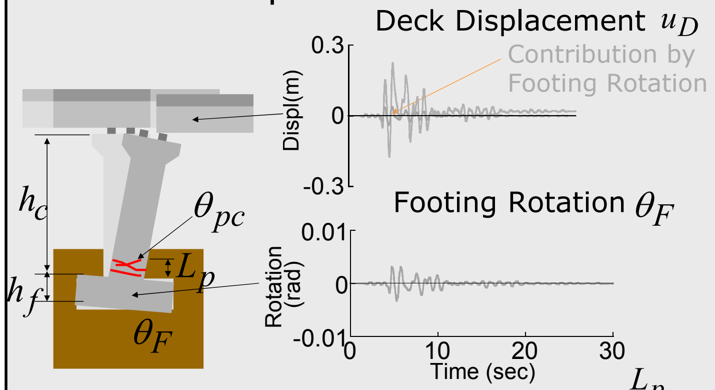


Contribution of Footing Displacement and Column Displacement to Deck Response



$$u_D = u_F + \theta_F(h_c + h_f) + u_{Cf} + \theta_{pc}(h_c - \frac{L_p}{2})$$

Contribution of Footing Rotation to the Deck Displacement

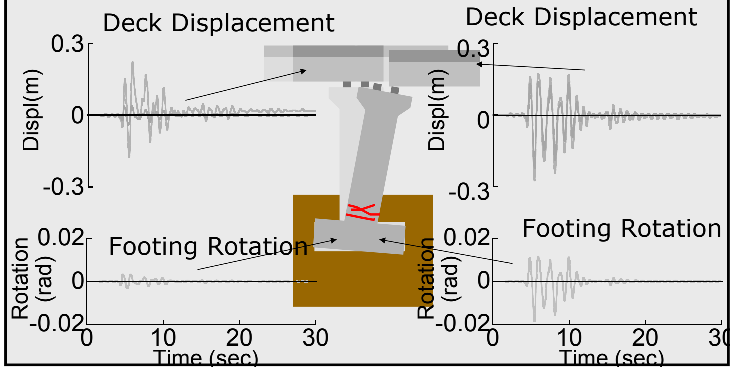


$$u_D = u_F + \theta_F(h_c + h_f) + u_{cf} + \theta_{pc}(h_c - \frac{L_p}{2})$$

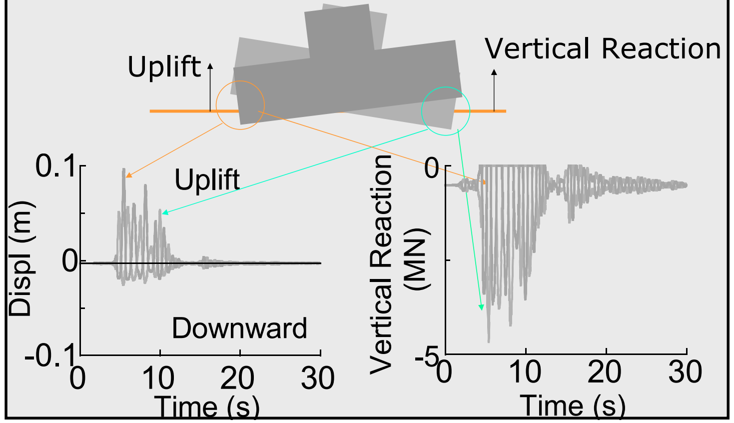
Effect of the Uplift of Foundation

Underlying ground resists tension

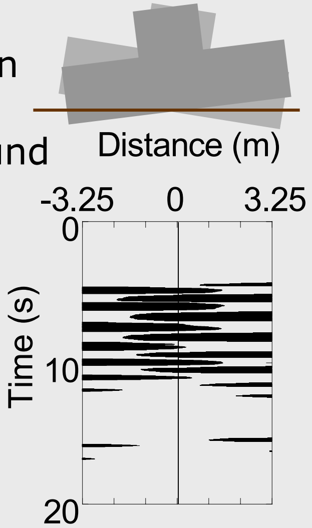
Separations are allowed between footing and underlying ground



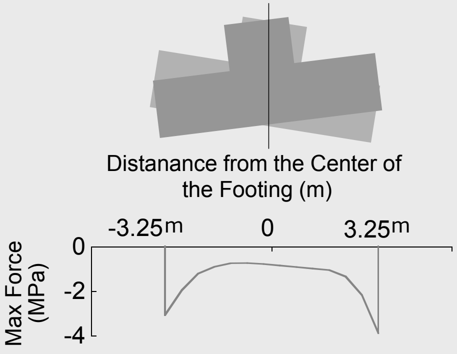
Uplift of the Foundation from the Underlying Ground



Separations between the Foundation and the Underlying Ground



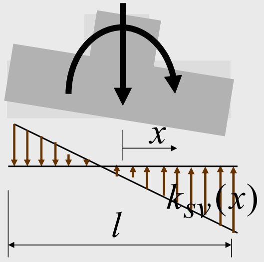
Peak Subgrade Reaction under the Footing



- Effect of local softening
- Effect of yield of the underlying ground

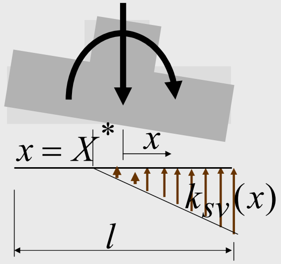
Moment vs. Rotation Relation of the Foundation

Underlying ground resists tension



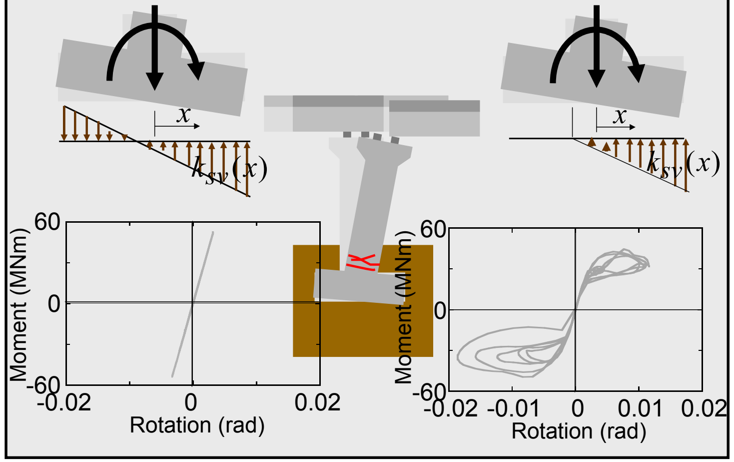
$$K_{F\theta} = \int_{-l/2}^{l/2} k_{sv}(x) \cdot x^2 dx$$

Separations are allowed between the footing and the underlying ground



$$K_{F\theta} = \int_{X^*}^{l/2} k_{sv}(x) \cdot x^2 dx$$

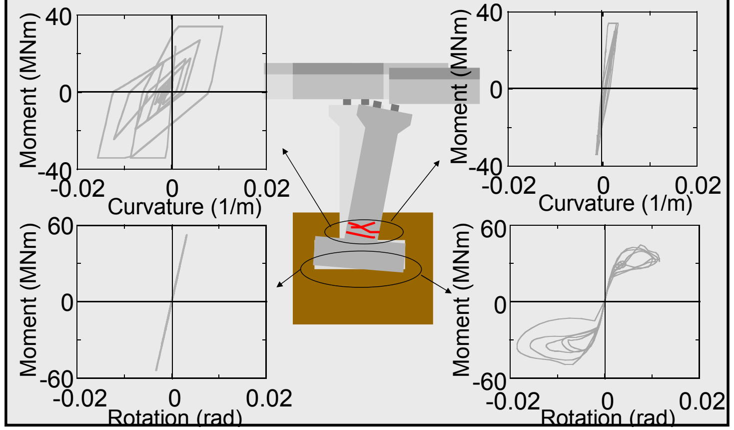
Moment vs. Rotation of the Footing



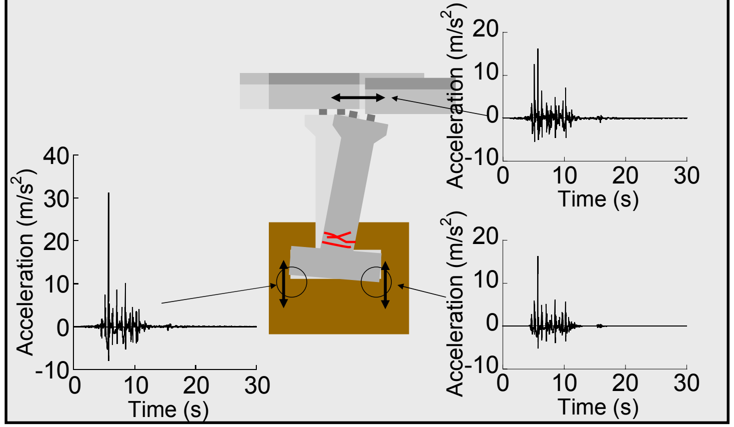
Column and Foundation Interaction

Underlying ground resists tension

Separations of the footing from the underlying ground

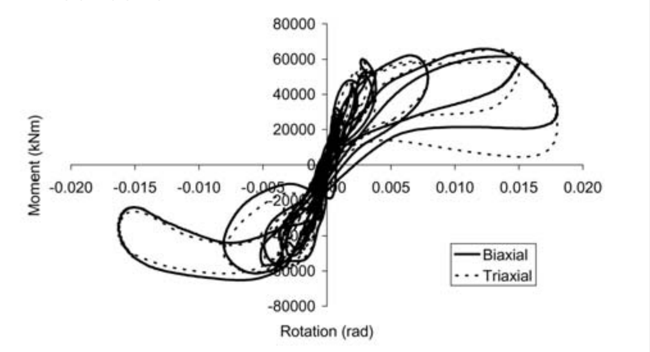


Collision between the Footing and the Underlying Ground



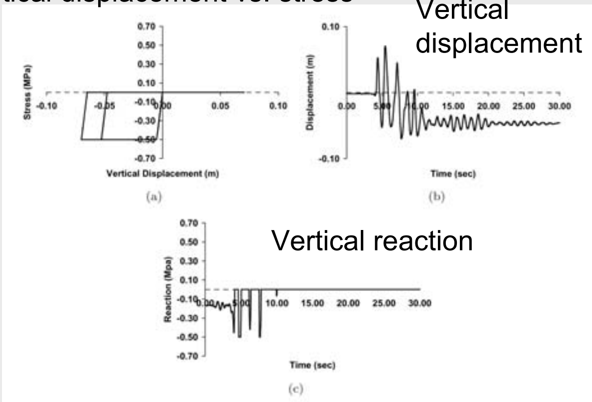
Effect of Bilateral Excitation

Moment vs. Rotation Hysteresis of the Foundation



Effect of Yield of Underlying Ground

Vertical displacement vs. stress



Vertical displacement

Vertical reaction