Seismic Design of Bridges

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Course Outline

1. **Design philosophy**
2. **Design response spectra and design procedures**
3. **Modeling of bridges**
4. **Design of RC columns**
5. **Foundation stability and design of foundations**
6. **Design of movement joints**

Chapter 1: Design Philosophy

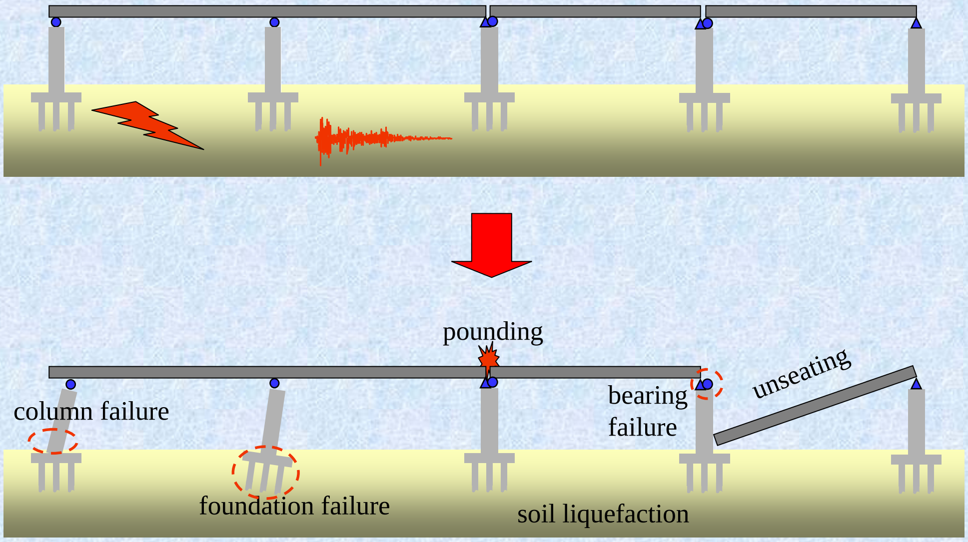
1. Lessons learned from past earthquakes

> ***"Those who ignore the lessons of history are doomed to repeat its mistakes."***
> From Seismic Design and Retrofit of Bridges by Priestley et al. (1996)

2. Performance criteria

> ***"How do you want the structure to perform in an earthquake? How much danger can you accept?"***
> Roberts, J. (1999)

Lessons Learned from Past Earthquakes

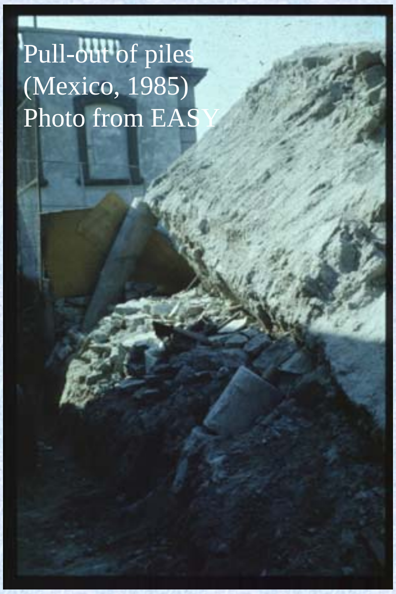


Lessons Learned: Liquefaction

Large settlements of ground near crane girders on piles. Kobe 1995
Photo from EASY

Niigata, Japan earthquake, June 16, 1964
Photo from NISEE

Lessons Learned: Foundation Failure



Pull-out of piles
(Mexico, 1985)
Photo from EASY

Cracked pile and extension of nominal vertical reinforcing. (Kobe, 1995)
Photo from NISEE

Pile stayed in place while soil oscillated, leaving imprint in soil about 30cm.
Photo from NISEE

Lessons Learned: Flexural Failure of Column

Hanshin Expressway, Kobe 1995
Photo from NISEE

Lessons Learned: Shear Failure of Column

Railway bridge, Kobe 1995
Photo from EASY

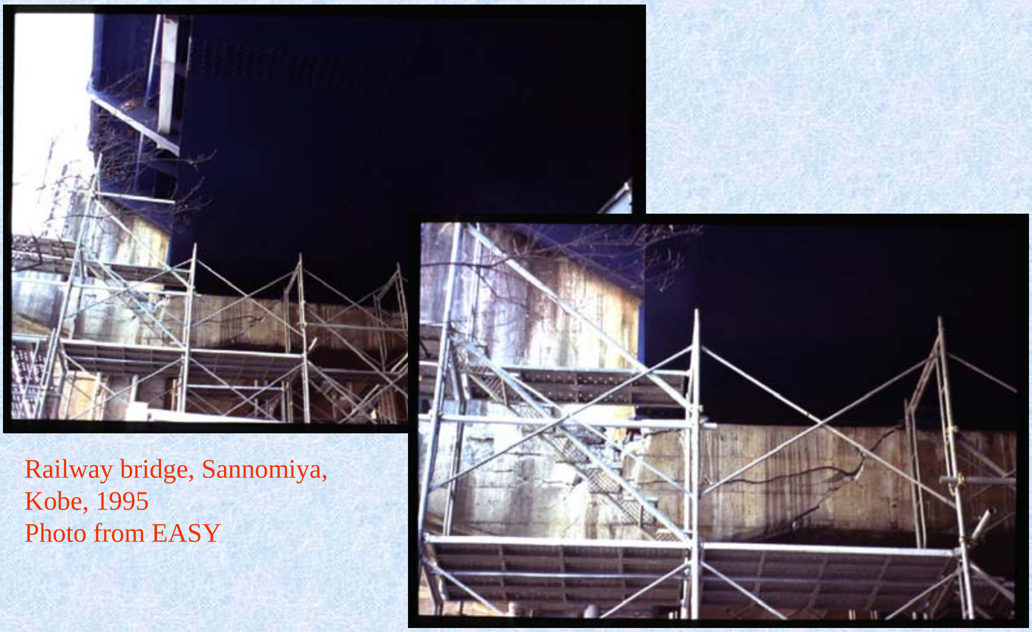
Lessons Learned: Splice Failure of Column

Hanshin Expressway, Kobe 1995
Photo from NISEE

Lessons Learned: Splice Failure of Column

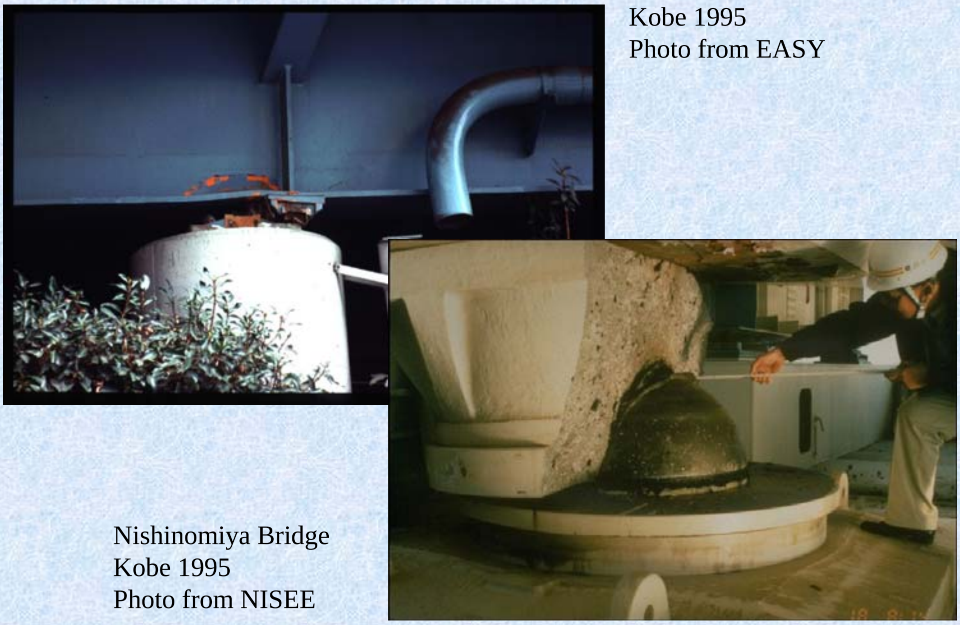
Different type of column failure with many failed splices
Hanshin Expressway, Kobe 1995
Photo from NISEE

Lessons Learned: Shear Failure of Cap Beam



Railway bridge, Sannomiya,
Kobe, 1995
Photo from EASY

Lessons Learned: Bearing Failures



Kobe 1995
Photo from EASY

Nishinomiya Bridge
Kobe 1995
Photo from NISEE

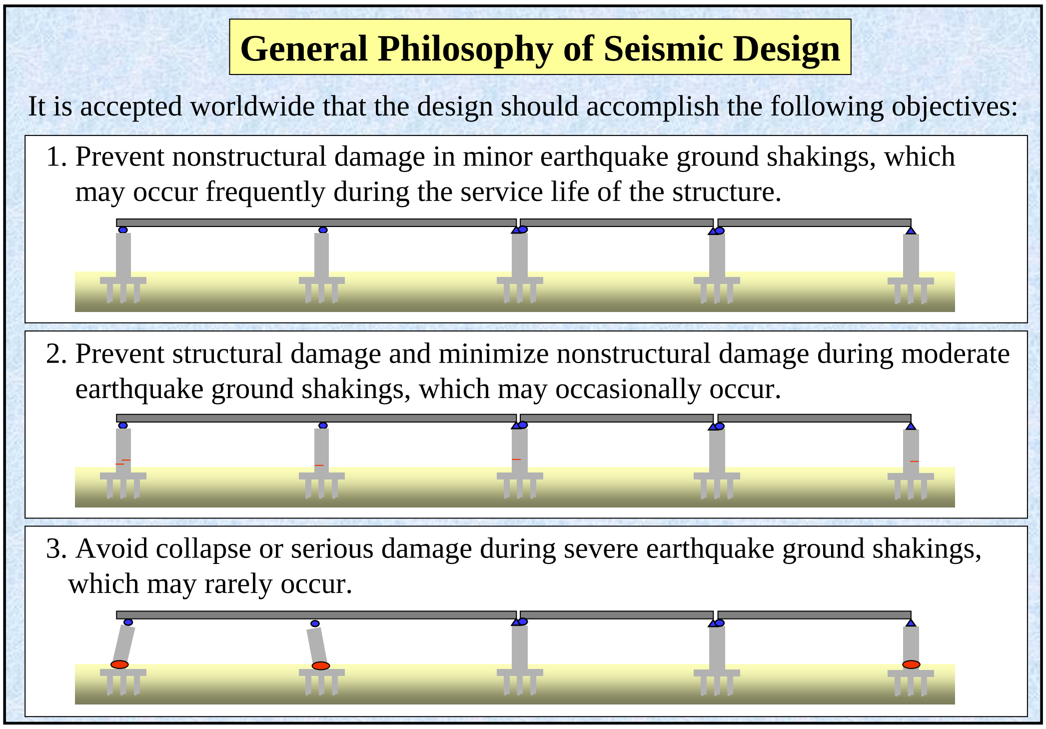
Lessons Learned: Pounding

Interstate-5 at Santa Clara River. Joint was open about 1/2 inch.
Northridge EQ 1994
Photo from NISEE

Steel deck girder hit into the abutment and locally buckled. The abutment failed in shear.
Kobe 1995
Photo from EASY

Lessons Learned: Unseating

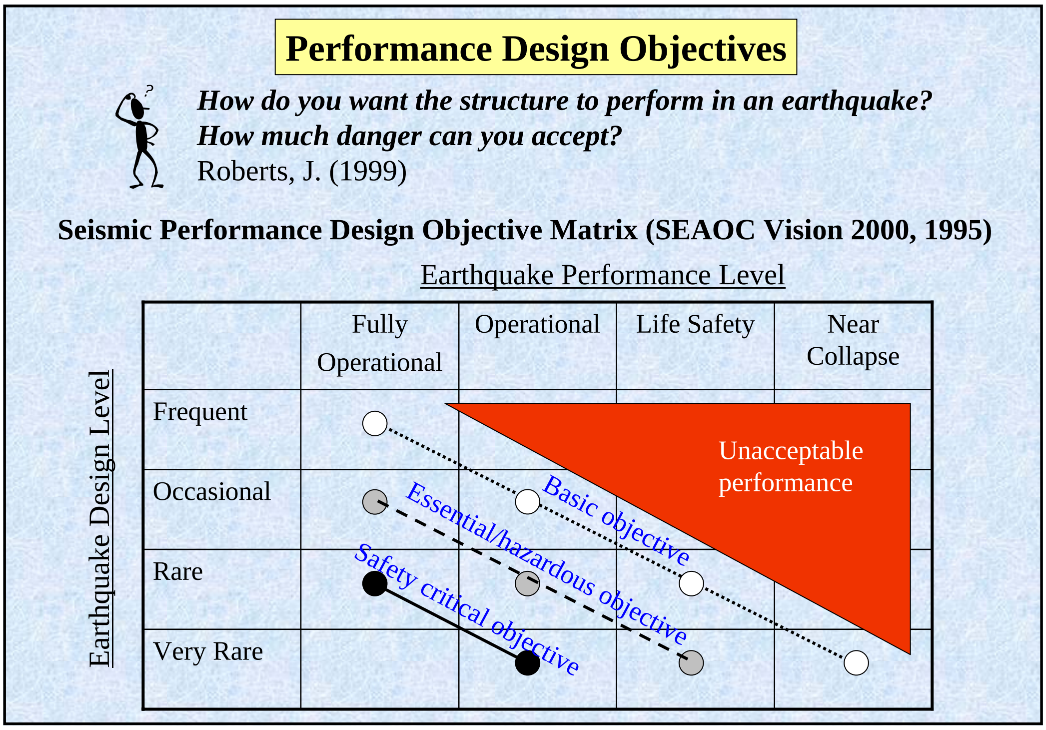
Kobe 1995
Photo from NISEE



General Philosophy of Seismic Design

It is accepted worldwide that the design should accomplish the following objectives:

1. Prevent nonstructural damage in minor earthquake ground shakings, which may occur frequently during the service life of the structure.
2. Prevent structural damage and minimize nonstructural damage during moderate earthquake ground shakings, which may occasionally occur.
3. Avoid collapse or serious damage during severe earthquake ground shakings, which may rarely occur.



Performance Design Objectives

How do you want the structure to perform in an earthquake?
How much danger can you accept?
Roberts, J. (1999)

Seismic Performance Design Objective Matrix (SEAOC Vision 2000, 1995)

Earthquake Performance Level

	Fully Operational	Operational	Life Safety	Near Collapse
Frequent				
Occasional				
Rare				
Very Rare				

Seismic Design Codes

Japan Road Association: Design Specifications of Highway Bridges – Part V Seismic Design, 2002 (**JRA**)

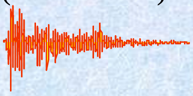
AASHTO: Standard Specifications for Highway Bridges, 1996 (**AASHTO**)

Applied Technology Council: Improved Seismic Design Criteria for California Bridges – Provisional Recommendations (ATC-32), 1996 (**ATC-32**)

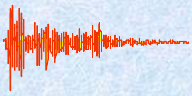
Transit New Zealand: Bridge Manual, 1995 (**TNZ**)

CEN: Eurocode 8 - Design Provisions for Earthquake Resistance of Structures, 1994 (**EC8**)

Performance Criteria: JRA

Ground Motion		**Ordinary Bridge**	**Important Bridge**
GM with high possibility of occurrence (Level-1 GM) 		Functional (1)	Functional (1)
GM with low possibility of occurrence (Level-2 GM) 	Type-I GM (Kanto EQ)	Prevent critical damage (3)	Retain limited damage (2)
	Type-II GM (Kobe EQ)		

Performance Criteria: ATC-32

Ground Motion	Level of Post-EQ Service		Level of Damage	
	Ordinary Bridges	**Important Bridge**	**Ordinary Bridges**	**Important Bridge**
Functional-Evaluation GM 	Immediate	Immediate	Reparable	Minimum
Safety-Evaluation GM 	Limited	Immediate	Significant	Reparable

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1. **Design philosophy**
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3. **Modeling of bridges**
4. **Design of RC columns**
5. **Foundation stability and design of foundations**
6. **Design of bearings and movement joints**
7. **Capacity design of bridges**

Chapter 2: Design Response Spectra and Design Procedures

1. Elastic and inelastic response spectra

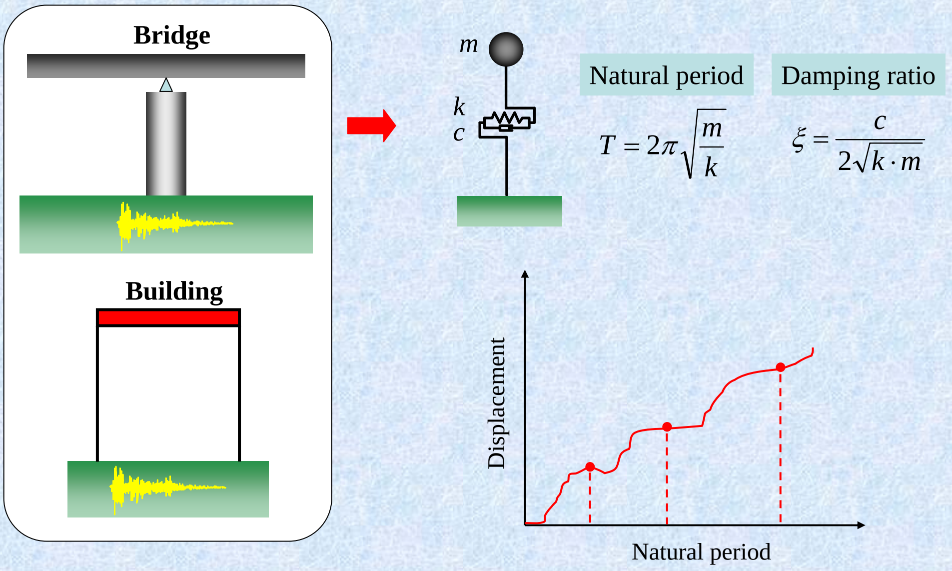
2. Force reduction factor
   - Equal-energy approximation
   - Equal-displacement approximation

3. Design response spectra (review of various design codes)

4. Design procedures

5. Load combination

Elastic Response Spectra



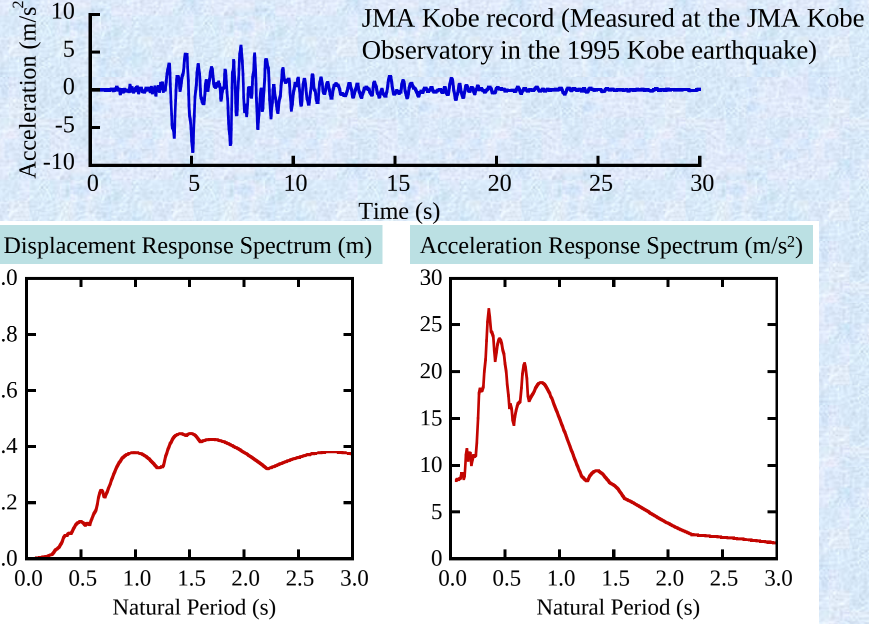
Natural period

$$T = 2\pi\sqrt{\frac{m}{k}}$$

Damping ratio

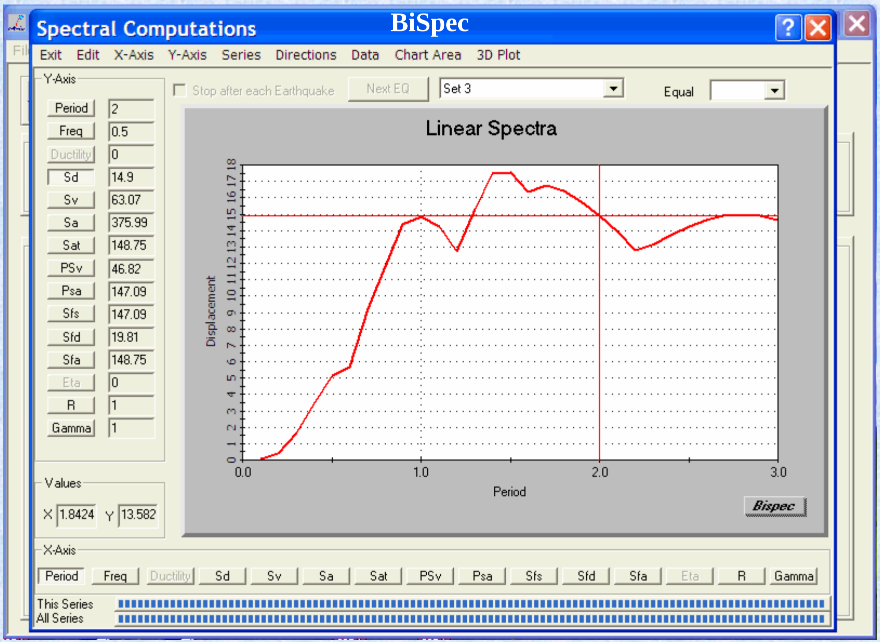
$$\xi = \frac{c}{2\sqrt{k \cdot m}}$$

Example of Elastic Response Spectra



JMA Kobe record (Measured at the JMA Kobe Observatory in the 1995 Kobe earthquake)

Visualization of Elastic Response Spectra

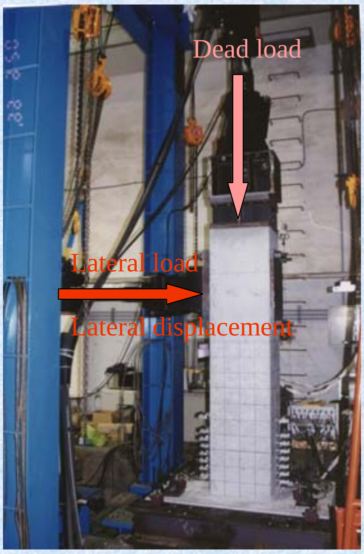


Set 3

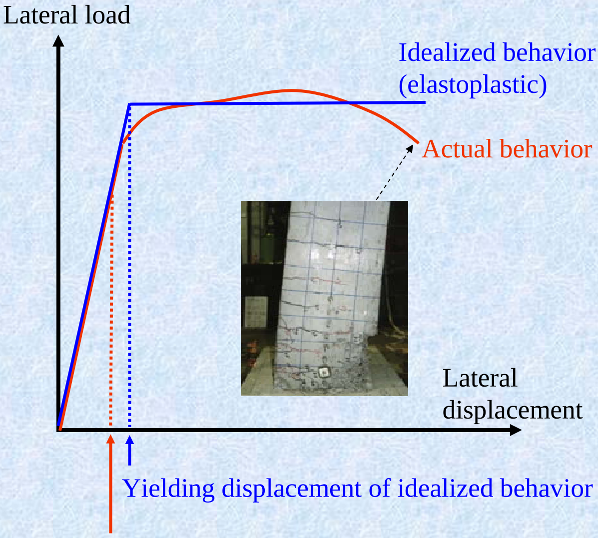
Period 2
Freq 0.5
Ductility 0
Sd 14.9
Sv 63.07
Sa 375.99
Sat 148.75
PSv 46.82
Psa 147.09
Sfs 147.09
Sfd 19.81
Sfa 148.75
Eta 0
R 1
Gamma 1

Values
X 1.8424 Y 13.582

Nonlinear Inelastic Behavior



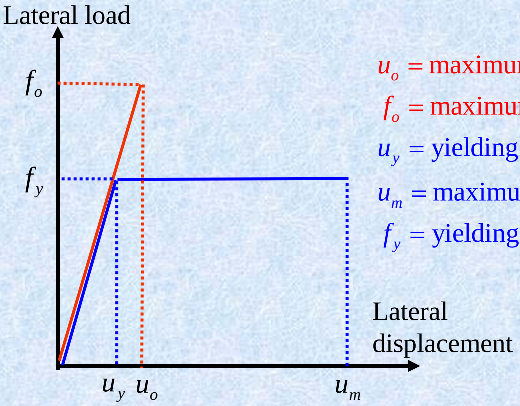
Photos from Dr. Jun-ichi Sakai (TIT)



Actual yielding displacement

Definition of Key Parameters of Elastoplastic System

Consider an elastic system and an elastoplastic system subjected to an earthquake. The following figure show the envelop curve.



u_o = maximum displacement of an elastic system

f_o = maximum force of an elastic system

u_y = yielding displacement of an elastoplastic system

u_m = maximum displacement of an elastoplastic system

f_y = yielding force of an elastoplastic system

$$\text{ductility factor } (\mu) = \frac{u_m}{u_y}$$

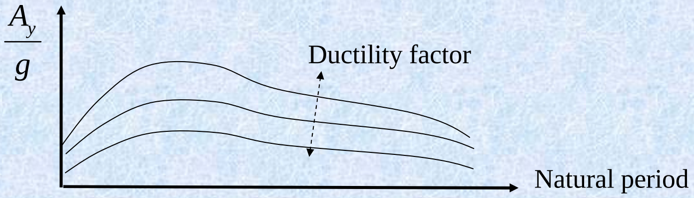
$$\text{Force reduction factor } (R) = \frac{f_o}{f_y}$$

Constant-Ductility Response Spectrum

For design purposes, it is desired to determine the **yield strength** of the system for a certain **design ductility factor**. It can be accomplished by resorting to "a constant-ductility response spectrum." The conventional elastic response spectrum can be considered as a constant-ductility response spectrum with a ductility factor of 1.

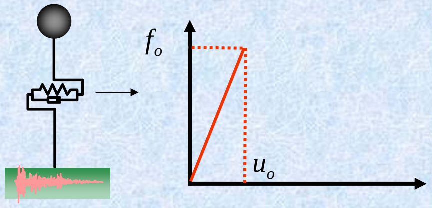
How should we present the response spectrum?

$$\text{Yield strength } f_y = \overset{\text{stiffness}}{k}u_y = (\overset{\text{mass}}{m}\underset{\text{natural angular frequency}}{\omega^2})u_y = m(\omega^2 u_y) = m\overset{\text{pseudo-acceleration}}{A_y} = \boxed{\frac{A_y}{g}}\underset{\text{weight}}{w}$$

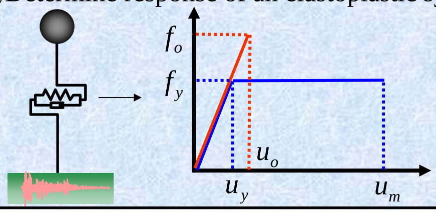


How to Construct Constant-Ductility Response Spectrum?

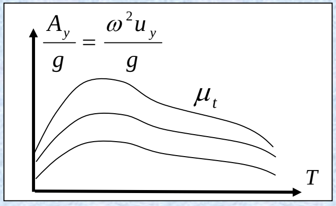
1. Define a ground motion.
2. Fix a mass and a damping ratio (typically 0.05).
3. Set a natural period *T*.
4. Determine response of a linear system.



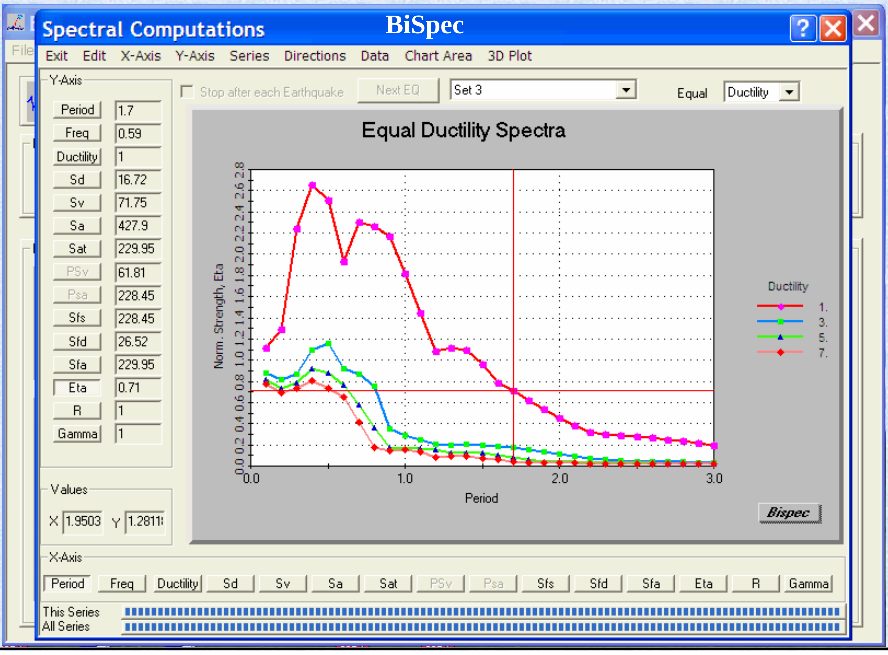
5. Set a *target* ductility factor μ_t
6. Determine response of an elastoplastic system with $\underset{\text{assumed}}{f_y} < f_o$



7. Compute a *response* ductility factor $\mu_r = \frac{u_m}{u_y}$
8. If $|\mu_r - \mu_t| > tolerance \approx 0$, repeat Step 6 by changing f_y until $|\mu_r - \mu_t| \le tolerance$. Then keep u_y and f_y
9. Repeat Step 5 for a different *target* ductility factor.
10. Repeat Step 3 for a different natural period.
11. Plot a constant-ductility response spectrum.



Visualization of Constant-Ductility Response Spectrum



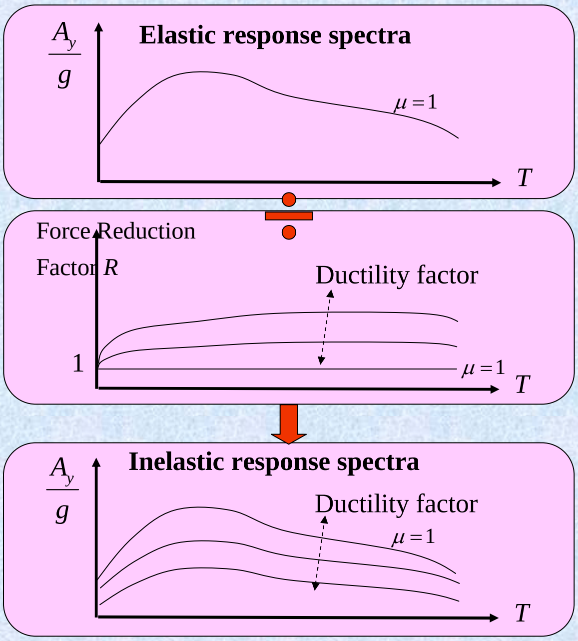
Set 3

Equal Ductility

Period	1.7
Freq	0.59
Ductility	1
Sd	16.72
Sv	71.75
Sa	427.9
Sat	229.95
PSv	61.81
Psa	228.45
Sfs	228.45
Sfd	26.52
Sfa	229.95
Eta	0.71
R	1
Gamma	1

X 1.9503 Y 1.2811

Force Reduction Factor



Elastic response spectra

Force Reduction Factor R

Ductility factor

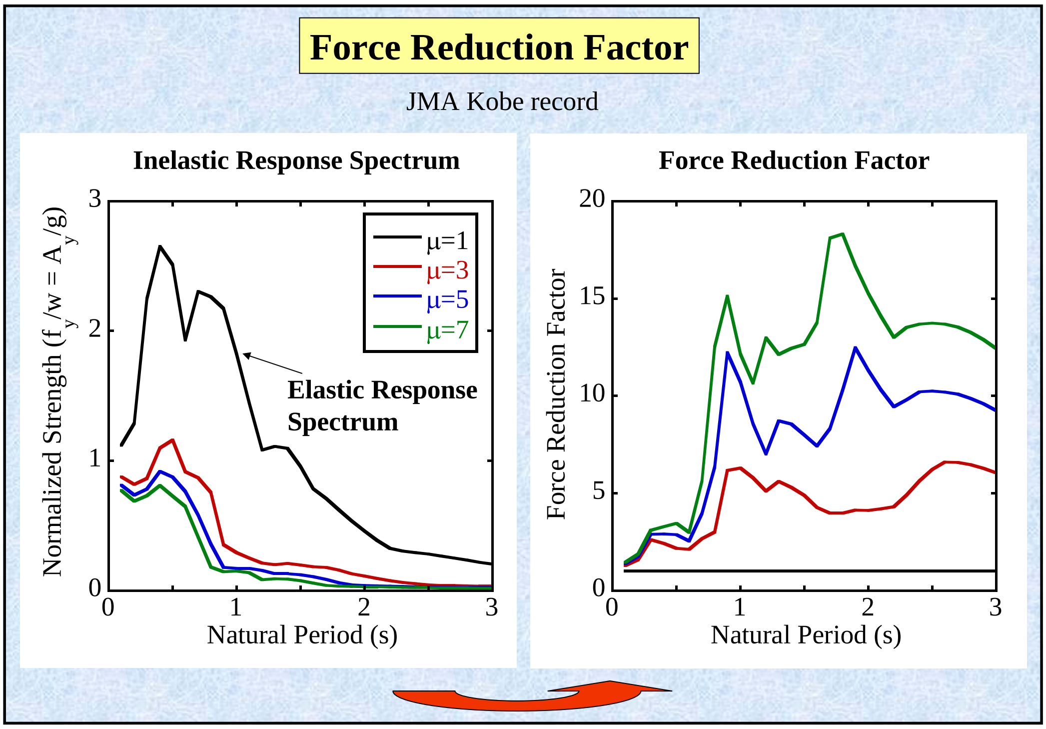
Inelastic response spectra

Ductility factor

Instead of directly computing an **inelastic response spectrum**, we can use an **elastic response spectrum** (due to its simplicity) with a **force reduction factor** (dependent on a natural period and a ductility factor).

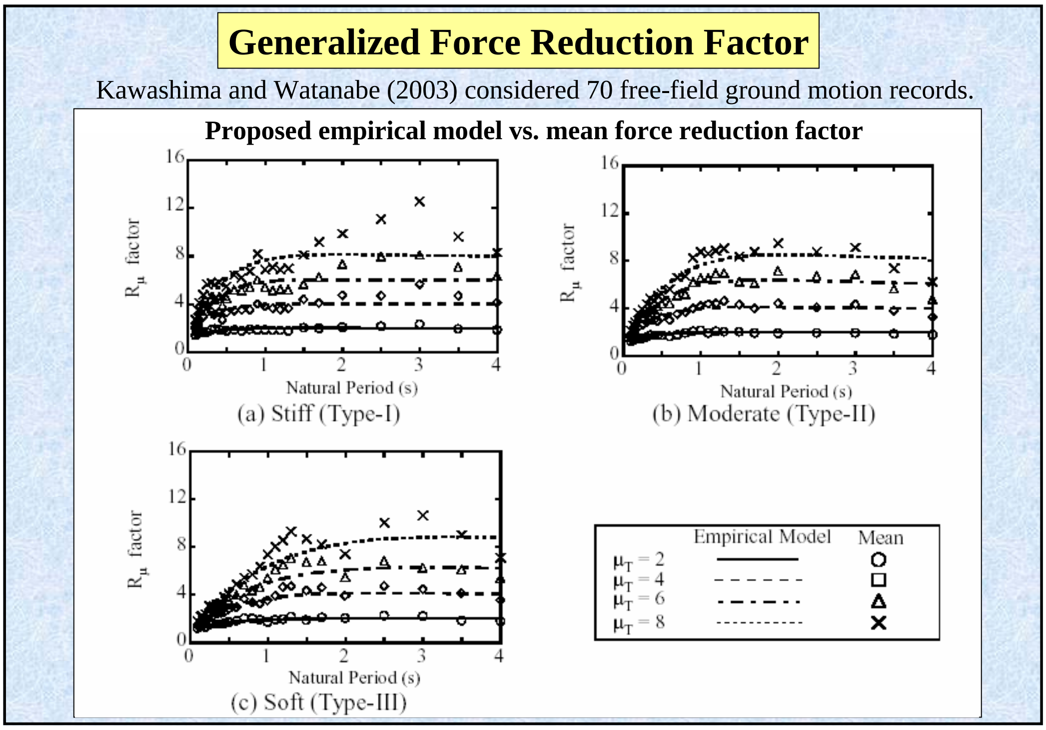
$$A_y(\mu,T)=\frac{A_y(\mu=1,T)}{R(\mu,T)}$$

$$f_y(\mu,T)=\frac{f_y(\mu=1,T)}{R(\mu,T)}$$



Force Reduction Factor

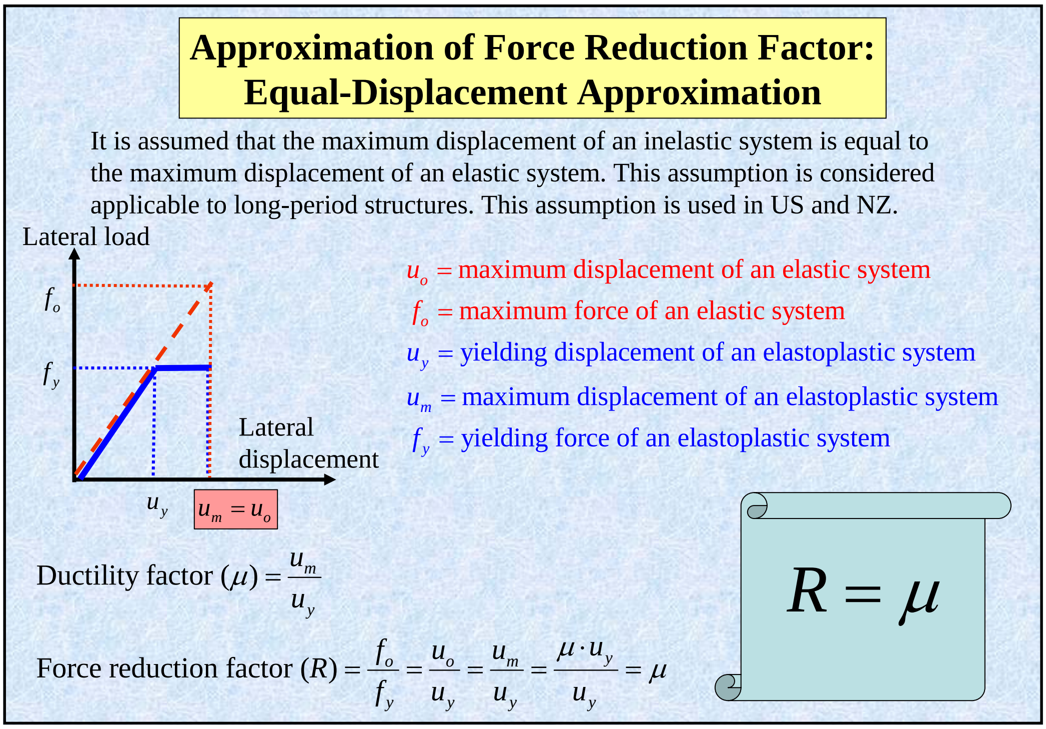
JMA Kobe record



Generalized Force Reduction Factor

Kawashima and Watanabe (2003) considered 70 free-field ground motion records.

Proposed empirical model vs. mean force reduction factor



Approximation of Force Reduction Factor: Equal-Displacement Approximation

It is assumed that the maximum displacement of an inelastic system is equal to the maximum displacement of an elastic system. This assumption is considered applicable to long-period structures. This assumption is used in US and NZ.

u_o = maximum displacement of an elastic system

f_o = maximum force of an elastic system

u_y = yielding displacement of an elastoplastic system

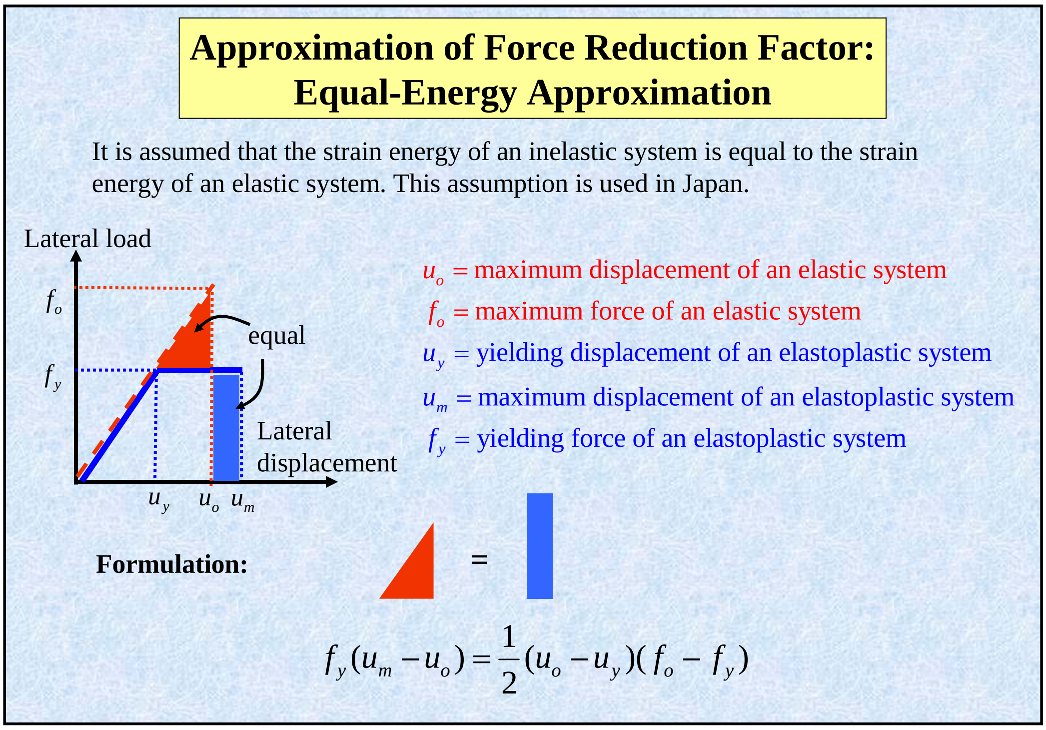
u_m = maximum displacement of an elastoplastic system

f_y = yielding force of an elastoplastic system

$$\text{Ductility factor } (\mu) = \frac{u_m}{u_y}$$

$$\text{Force reduction factor } (R) = \frac{f_o}{f_y} = \frac{u_o}{u_y} = \frac{u_m}{u_y} = \frac{\mu \cdot u_y}{u_y} = \mu$$

$$R = \mu$$



Approximation of Force Reduction Factor: Equal-Energy Approximation

It is assumed that the strain energy of an inelastic system is equal to the strain energy of an elastic system. This assumption is used in Japan.

u_o = maximum displacement of an elastic system

f_o = maximum force of an elastic system

u_y = yielding displacement of an elastoplastic system

u_m = maximum displacement of an elastoplastic system

f_y = yielding force of an elastoplastic system

Formulation: ◢ = ▮

$$f_y(u_m - u_o) = \frac{1}{2}(u_o - u_y)(f_o - f_y)$$

Approximation of Force Reduction Factor: Equal-Energy Approximation

$$f_y(u_m - u_o) = \frac{1}{2}(u_o - u_y)(f_o - f_y)$$

$$f_y(\mu u_y - u_o) = \frac{1}{2}\frac{u_y}{f_y}(f_o - f_y)(f_o - f_y)$$

$$f_y(\mu u_y - \frac{f_o u_y}{f_y}) = \frac{1}{2}\frac{u_y}{f_y}(f_o - f_y)(f_o - f_y)$$

$$f_y u_y(\mu - \frac{f_o}{f_y}) = \frac{1}{2}\frac{u_y}{f_y}(f_o - f_y)(f_o - f_y)$$

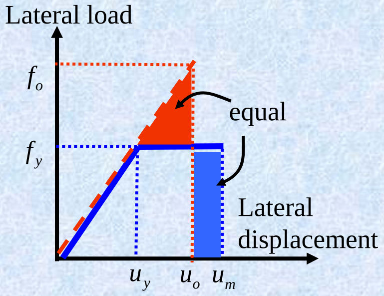
$$\mu - \frac{f_o}{f_y} = \frac{1}{2}\frac{1}{f_y^2}(f_o - f_y)(f_o - f_y)$$

$$\mu - \frac{f_o}{f_y} = \frac{1}{2}(\frac{f_o}{f_y} - 1)(\frac{f_o}{f_y} - 1)$$

$$\mu - R = \frac{1}{2}(R-1)(R-1)$$

$$2\mu - 2R = R^2 - 2R + 1$$

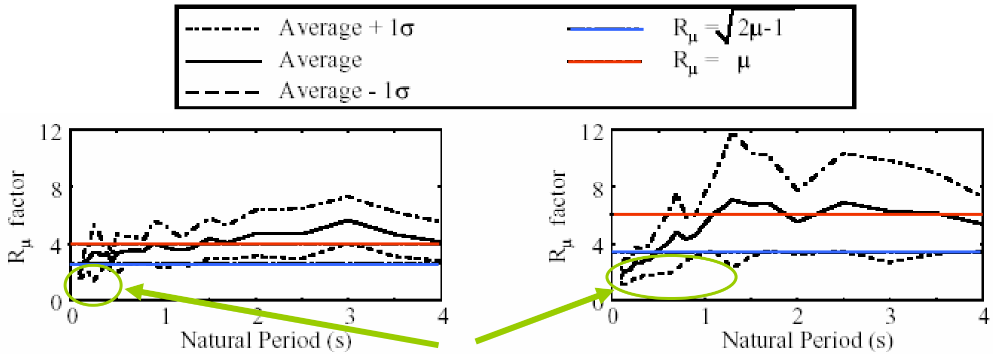
$$R = \sqrt{2\mu - 1}$$



$$R = \sqrt{2\mu - 1}$$

Comparison of Force Reduction Factors

Mean and Mean +/- One Standard Deviation of the Force Reduction Factors for 70 Ground Motions and Soft Soil (Kawashima and Watanabe (2003)



Not conservative at short periods

(1) μ_T=4

(2) μ_T=6

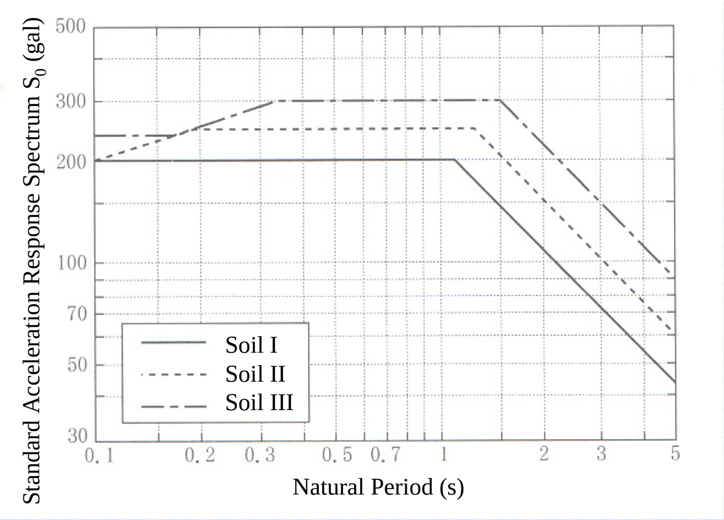
Taking account of the considerable scattering of the force reduction factors depending on the ground motions, it is conservative to assume the equal energy assumption instead of the equal displacement assumption for the evaluation of the force reduction factors

Design Response Spectra: JRA

Function-Evaluation (Level-1) $S = c_z c_D S_0$

damping modification factor

zone factor

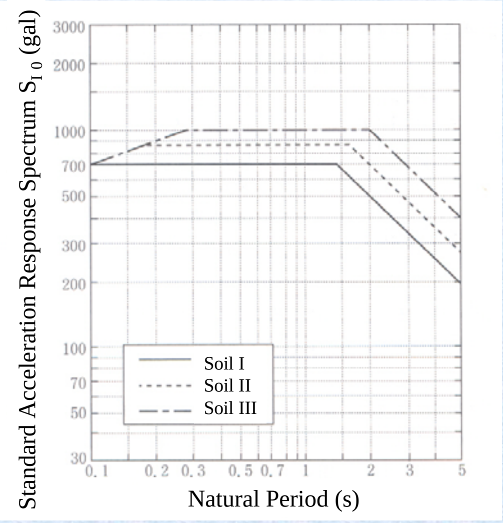


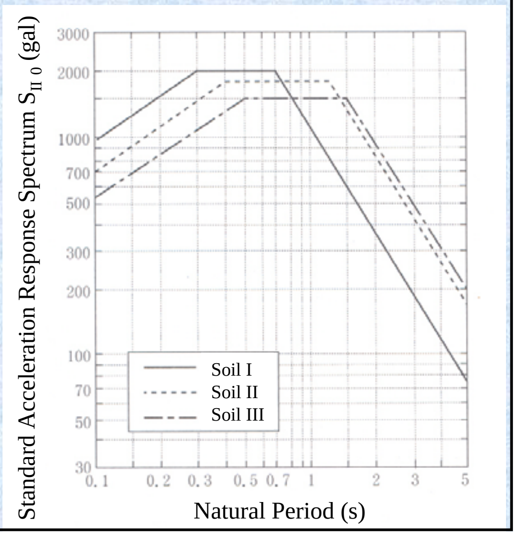
Design Response Spectra: JRA

Safety-Evaluation (Level-2)

Type-I ground motion $S_{\mathrm{I}} = c_z c_D S_{\mathrm{I}\,0}$ ⟵ Represent the 1923 Kanto EQ

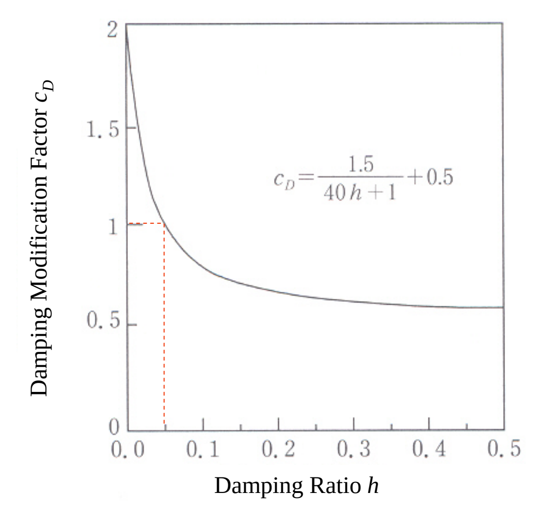
Type-II ground motion $S_{\mathrm{II}} = c_z c_D S_{\mathrm{II}\,0}$ ⟵ Represent the 1995 Kobe EQ





Design Response Spectra: JRA

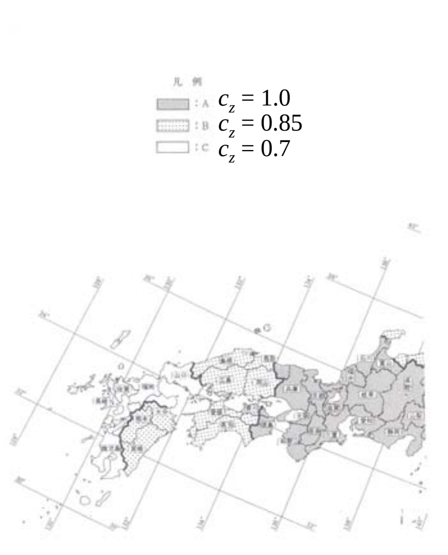
Damping modification factor c_D



Standard response spectra are presented for the 5% damping ratio.

Design Response Spectra: JRA

Zone factor c_z



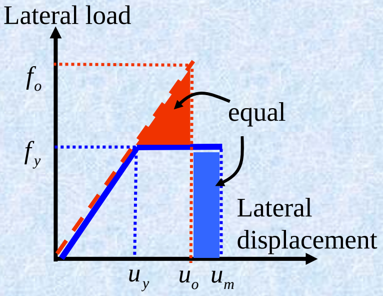
Design Response Spectra: JRA

Lateral strength capacity $$P \ge \frac{\frac{S_{\mathrm{I\ or\ II}}}{g}W}{R}$$

weight

response modification factor

A response modification factor (force-reduction factor) is determined based on the equal-energy approximation.



$$R = \sqrt{2\mu_a - 1}$$

design ductility factor

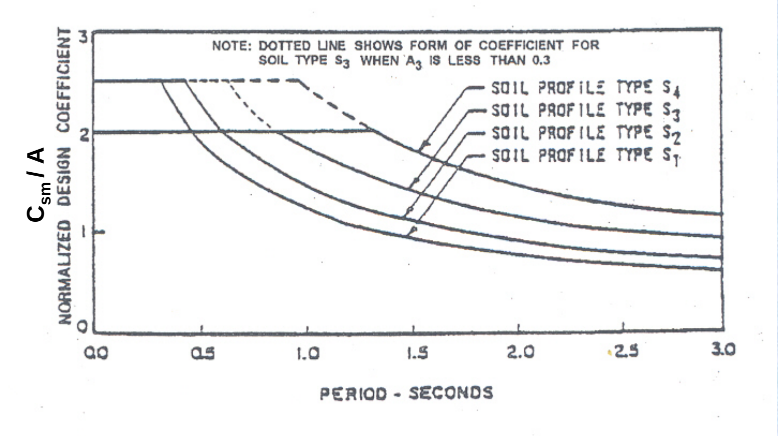
Design Response Spectra: AASHTO

Elastic seismic response coefficient $$C_s = \frac{1.2AS}{T^{2/3}} \le 2.5A$$

site coefficient (= 1.0, 1.2, 1.5, 2.0)

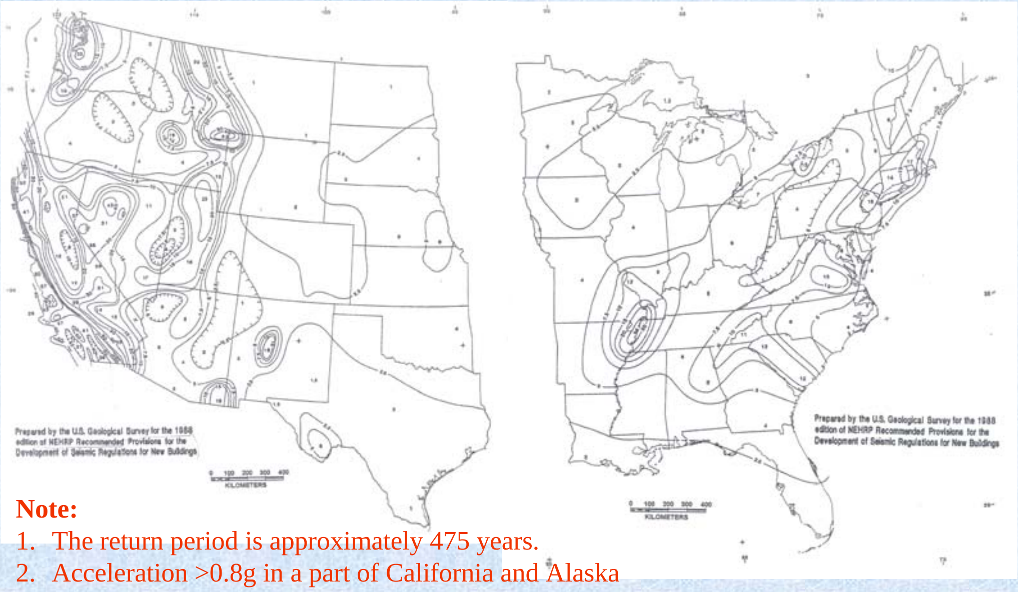
ground acceleration (g)

period



Design Response Spectra: AASHTO

Ground acceleration (called “acceleration coefficient”) in % of g



Note:

1. The return period is approximately 475 years.
2. Acceleration >0.8g in a part of California and Alaska

Design Response Spectra: AASHTO

Response modification factor (R-factor)

Substructure	R
Wall-type pier	2
RC pile bents • vertical piles only • one or more batter piles	 3 2
Single columns	3
Steel or composite steel and concrete pile bents • vertical piles only • one or more batter piles	 5 3
Multiple column bents	5

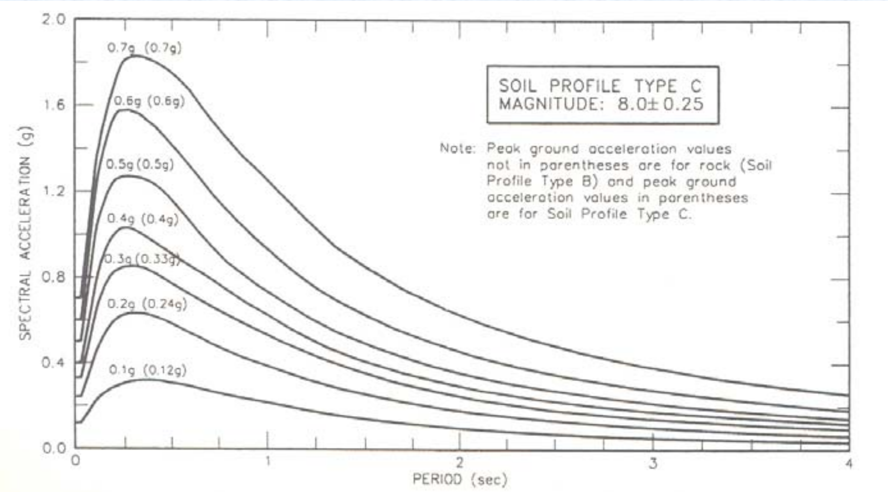
Design Response Spectra: ATC-32

Elastic acceleration response spectrum on a rock site

Elastic response spectrum = ARS

peak rock acceleration (g)

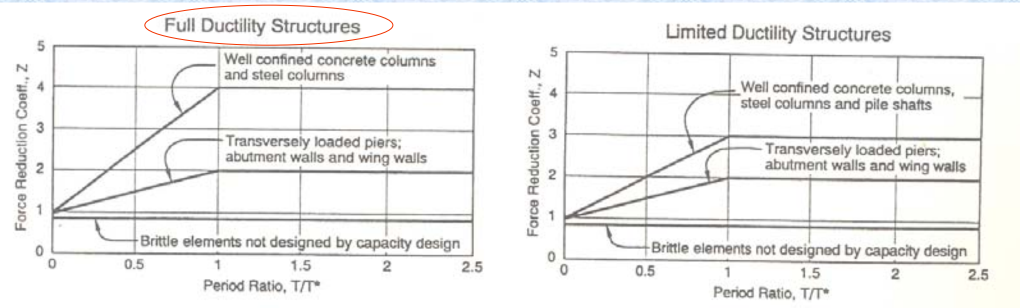
site modification factor



Design Response Spectra: ATC-32

Response modification factor (Z-factor)

Important bridges must be designed as full ductility structures.



Design Response Spectra: TNZ

basic seismic coefficient

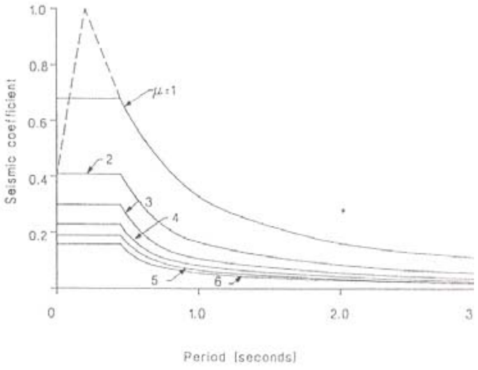
zone factor

Seismic coefficient (V/W) = C_μ Z R S_p $\geq$ 0.05

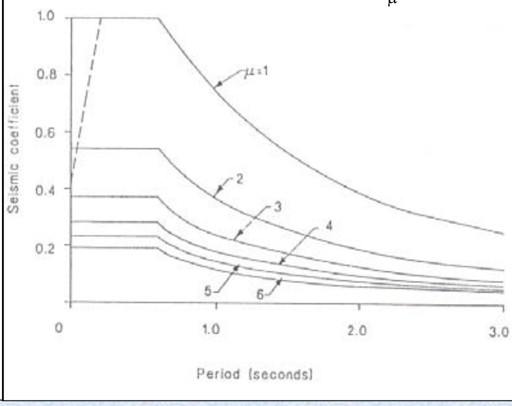
risk factor

structural performance factor

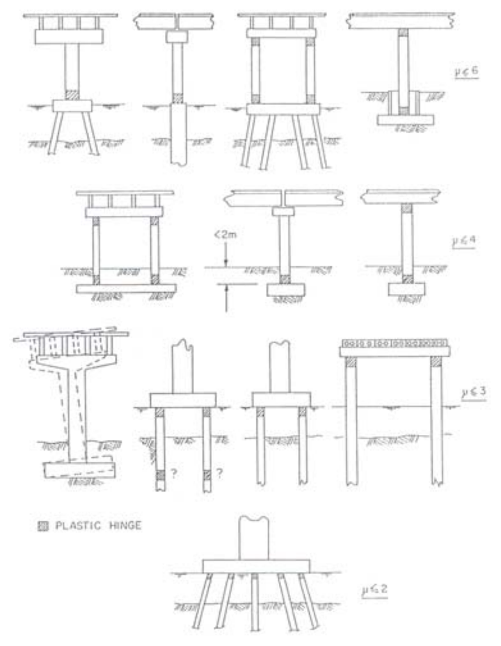
Basic acceleration coefficient C_μ for stiff soil



Basic acceleration coefficient C_μ for stiff soil



Design Response Spectra: TNZ



Design Response Spectra: TNZ

Maximum value = 1.2
Minimum value = 0.6
Interpolate linearly between contours

Zone factor Z

Z = 0.6 – 1.2

Design Response Spectra: TNZ

Risk factor R

Importance Category	R
Bridges carrying more than 2500 vpd Bridges carrying or crossing motorways and railways	1.3
Bridges carrying between 250 and 2500 vpd	1.15
Bridges carrying less than 250 vpd Non permanent bridges	1.0

The return periods of design earthquakes are about 900, 650, and 450 years for bridges with risk factors of 1.3, 1.15, and 1.0, respectively.

Structural performance factor S_p

Site Subsoil Category	S_p
Rock or very stiff sites	0.9
Intermediate soil sites	0.8
Flexible or deep soil sites	0.67

This factor accounts for damping arising from radiation and inelastic behavior in the foundation.

Design Response Spectra: EC8

spectral acceleration amplification factor (=2.5)

ground acceleration (g)

$$S = k_I \cdot k_S \cdot a_g \cdot \begin{cases} 1+\frac{T}{T_B}(k_D \cdot \beta_0 - 1) & ; \quad 0 \le T \le T_B \\ k_D \cdot \beta_0 & ; \quad T_B \le T \le T_C \\ k_D \cdot \beta_0 \cdot \left(\frac{T_C}{T}\right) & ; \quad T_C \le T \le 3 \\ k_D \cdot \beta_0 \cdot \left(\frac{T_C}{3}\right) \cdot \left(\frac{3}{T}\right)^2 & ; \quad 3 \le T \end{cases}$$

important factor
(=1.3, 1.0, 0.7)

site modification factor
(=1.0, 1.0, 0.9)

damping modification factor

Values of T_B and T_C

Soil Classification	T_B (s)	T_C (s)
A	0.1	0.4
B	0.15	0.6
C	0.2	0.8

Damping modification factor

$$k_D = \sqrt{\frac{0.07}{0.02+h}} \geq 0.7$$

damping ratio

Design Response Spectra: EC8

Response modification factor (q-factor)

Substructure	**Seismic Behavior**	
	Limited Ductile	**Ductile**
RC columns • slender (H/L ≥ 3.5) • short (H/L=1)	 1.5 1.0	 3.5 1.0
Abutment	1.0	1.0

Comparison of Design Response Spectra

Factors / Codes	Zone factor	Importance factor	Site modification factor	Damping modification factor	Response modification factor
JRA (2002)	c_z = 1.0, 0.85, 0.7	- 2 categories - For computing ductility → R-factor	- 3 types - Response spectra	$c_D = \frac{1.5}{40h+1} + 0.5$	$R = \sqrt{2\mu_a - 1}$
AASHTO (1996)	Specify ground acceleration	-2 categories - For computing R-factor	- 4 types - S = 1.0, 1.2, 1,5, 2.0	No	- R-factor - From table
ATC-32 (1996)	Specify ground acceleration	- 2 categories - For computing Z-factor	- 6 types - Response spectra	No	- Z-factor - From chart
TNZ (1995)	Z = 1.2 – 0.6	- 3 categories - R = 1.3, 1.15, 1.0	- 3 types - Response spectra - S_p = 0.9, 0.8, 0.67	No	Use inelastic response spectra ($R = \mu$)
EC8 (1994)	Specify ground acceleration	- 3 categories - k_I = 1.3, 1.0, 0.7	- 3 types - k_S = 1.0, 1.0, 0.9	$k_D = \sqrt{\frac{0.07}{0.02+h}} \geq 0.7$	- q-factor - From table

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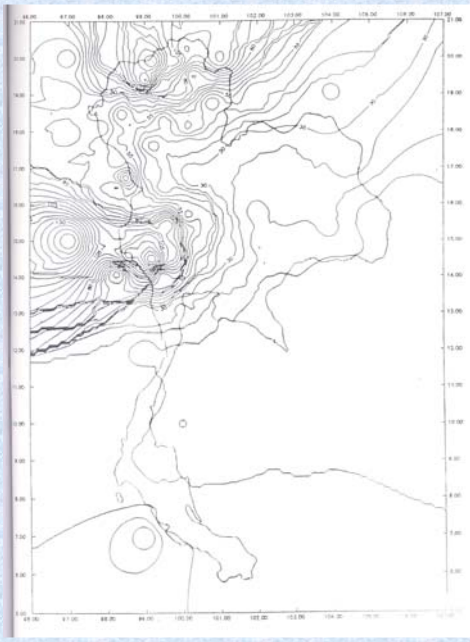
Design Response Spectra of Thailand



The contour map of PGA was proposed by Professor Panitan Lukkunaprasit.
Maximum ground acceleration is about 0.15 g.

✕

During the lack of acceleration data, basic response spectra specified in AASHTO or EC8 should be cautiously use.

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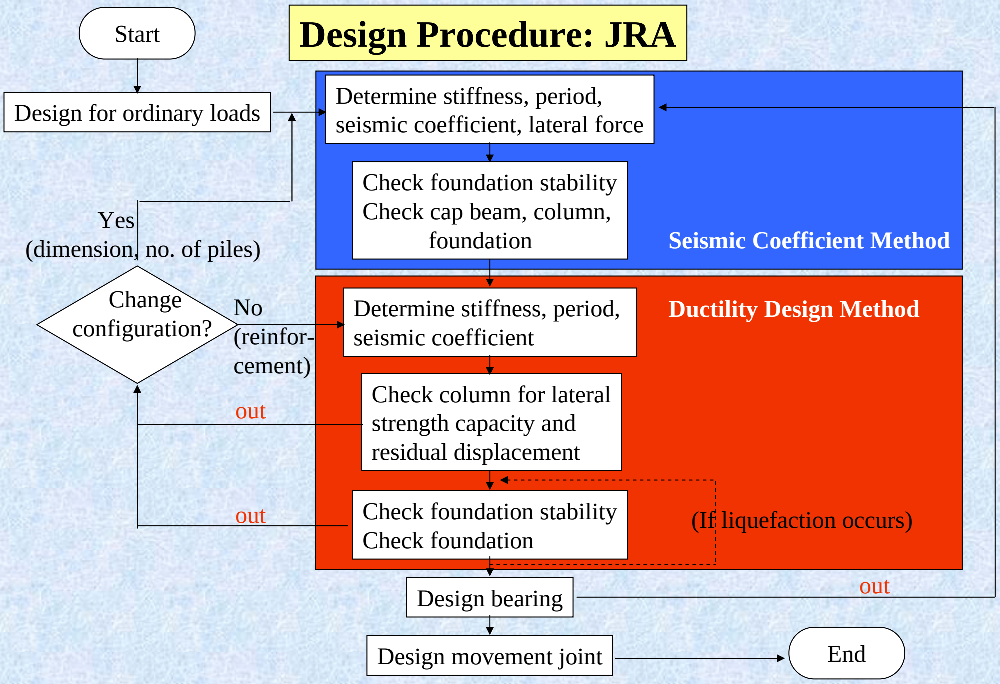
Design response spectra

Design Procedure: JRA



Start

Design for ordinary loads

Determine stiffness, period, seismic coefficient, lateral force

Check foundation stability
Check cap beam, column, foundation

Seismic Coefficient Method

Yes
(dimension, no. of piles)

Change configuration?

No
(reinfor-cement)

Determine stiffness, period, seismic coefficient

Ductility Design Method

Check column for lateral strength capacity and residual displacement

out

out

Check foundation stability
Check foundation

(If liquefaction occurs)

Design bearing

out

Design movement joint

End

Load Combination

Code	Load Combination
JRA	D+PS+CR+SH+E+HP+B+**EQ**
AASHTO	D+E+B+SF+**EQ**
TNZ	1.00{kD+1.35(E+HP+B)+SG+ST+**EQ**+0.33TP} 1.35(D+E+HP+B+SG+0.33**EQ**+1.1CN) (k=1.3 or 0.8, whichever is more severe, to allow for vertical acceleration)
EC8	$\underline{D}$+PS+**EQ**+ψL (ψ=0 for bridges with normal traffic, ψ=0.2 for bridges with heavy traffic, ψ=0.3 for railway bridges)