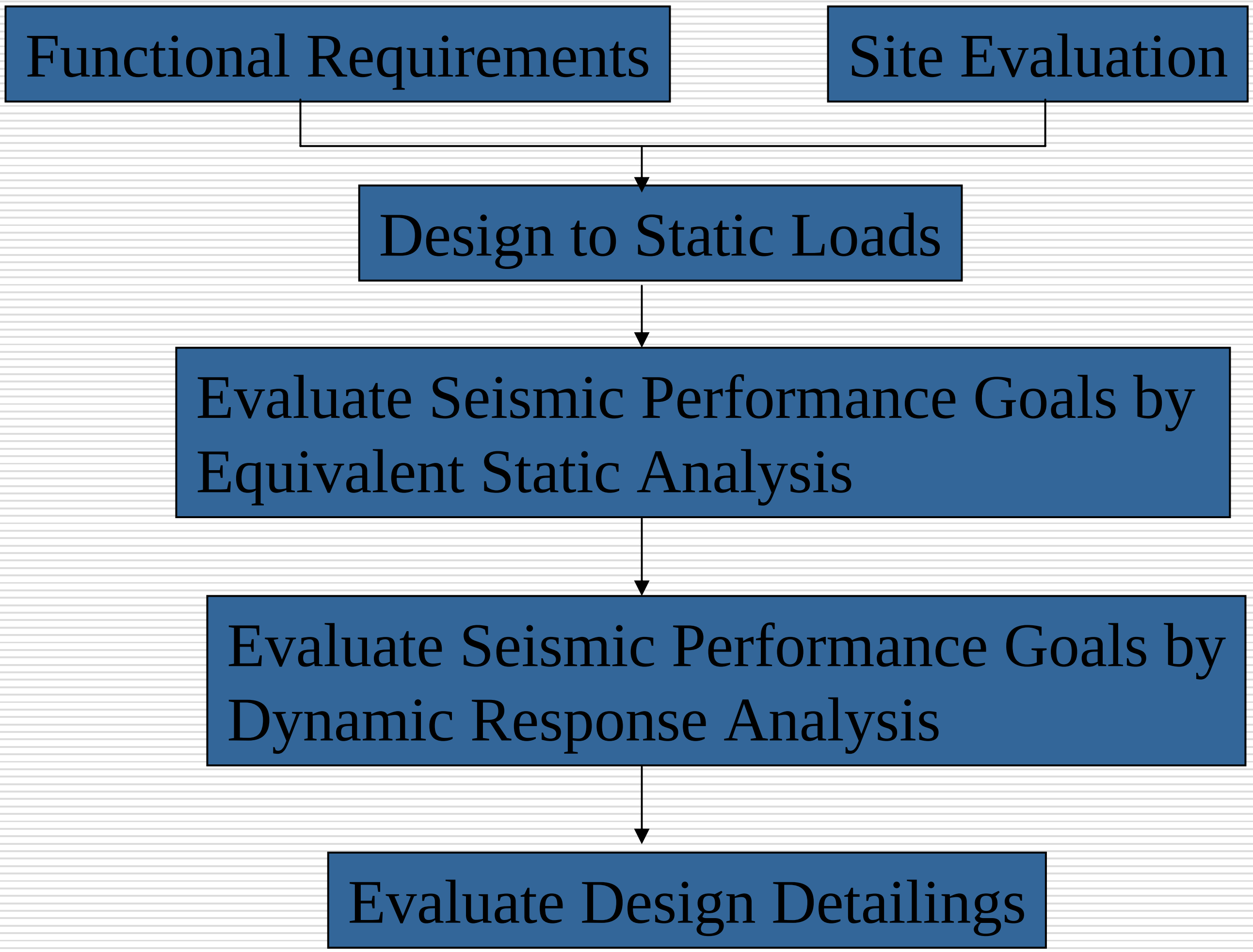
Chapter 6
Seismic Design of Bridges

Kazuhiko Kawashima
Tokyo Institute of Technology

Seismic Design

- Loading environment (dead, live, wind, earthquake etc)
- Performance criteria for gravity (deflection, stresses) and environmental loads (damage, displacement, collapse)
- Geometric (space) requirements
- Time available for construction
- Soil condition
- Cost
- ..

Process of Seismic Design



Requirements in Seismic Design

Load Resistance Factor Design (LRFD、荷重係数設計法)

$$\gamma \cdot D_{ave} \leq \phi \cdot C_{ave}$$

Capacity

Capacity Factor

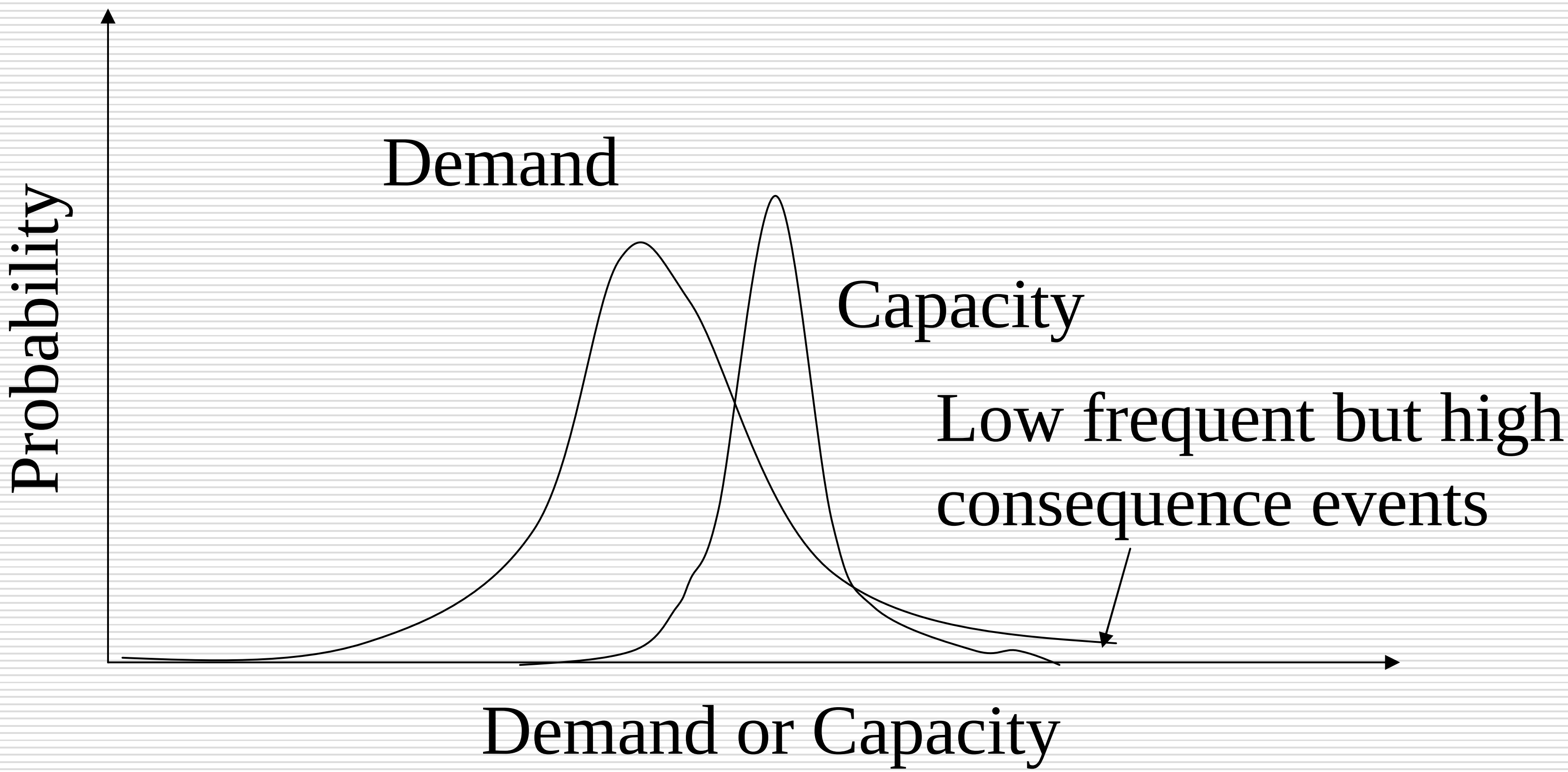
Demand

Load Factor

$$\gamma_1 \cdot \gamma_2 ......\gamma_n \cdot D_{ave} \leq \phi_1 \cdot \phi_2 ......\phi_n \cdot C_{ave}$$

A limit states format is currently based on performance-based design

Variation of Demand and Capacity



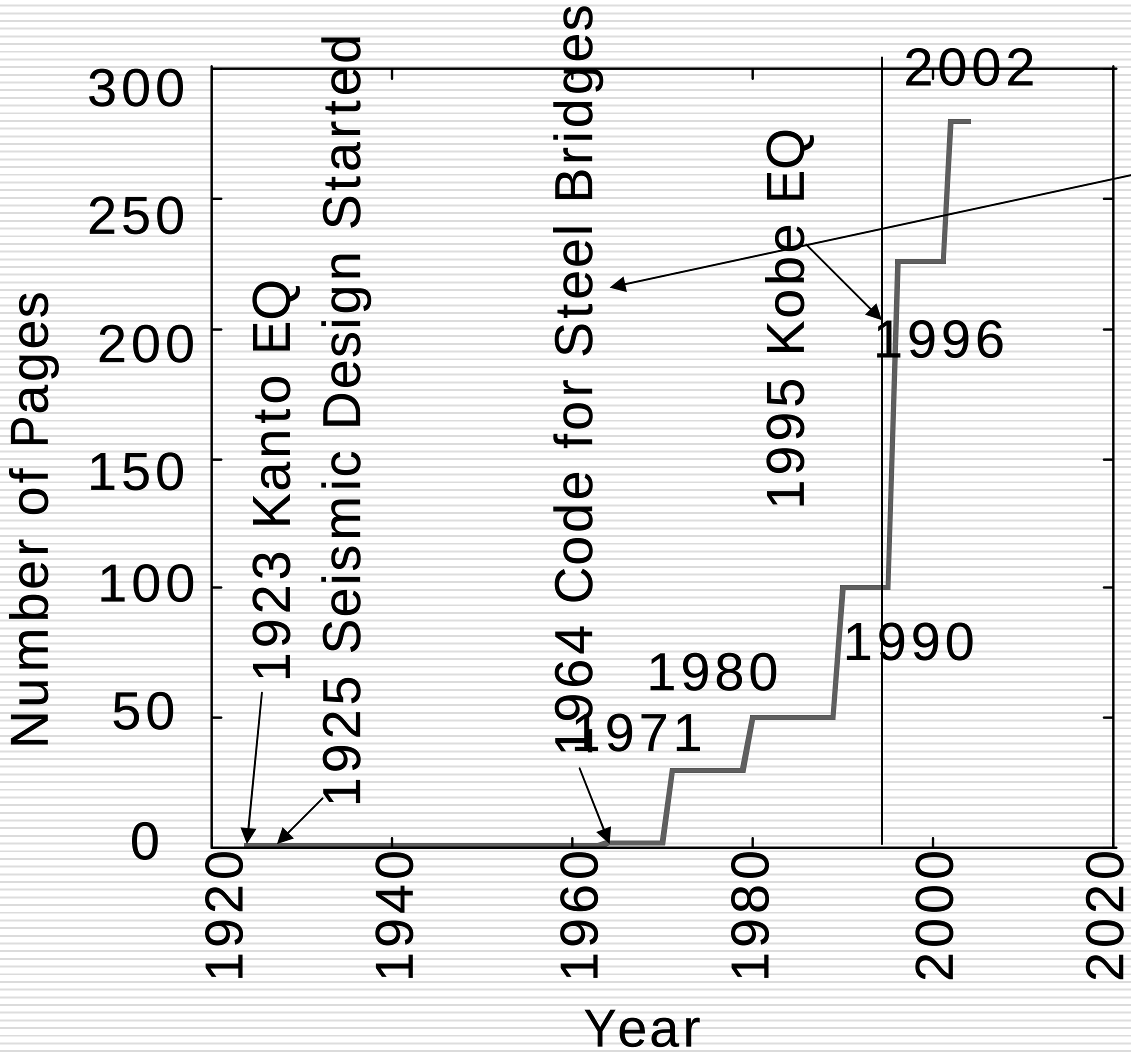
Seismic Design Process with Emphasis on Japanese Seismic Design

Japanese Codes for Design of Highway Bridges

Japan Road Association

- Design Specifications of Highway Bridges
  - ✓Part I Common Part
  - ✓Part II Steel Bridges
  - ✓Part III Concrete Bridges
  - ✓Part IV Foundations
  - ✓Part V Seismic Design
- The code applies to highway bridges with a center span no longer than 200m
  - ✓ 1980
  - ✓ 1990
  - ✓ 1996
  - ✓ 2002

Number of Pages related to Seismic Design of Highway Bridges in Japan



Most bridges which collapse during 1995 Kobe earthquake were designed according to this code.

1971Code and the Latest Code (2002)

Part V Seismic Design, Design Specifications of Highway Bridges, Japan Road Association

1. General
2. Principles of seismic design
3. Loads considered in seismic design
4. Design ground motions
5. Evaluation of seismic performance
6. Performance evaluation by static analysis
7. Performance evaluation by dynamic analysis
8. Effect of unstable soils
9. Menshin design (Seismic Isolation)

Part V Seismic Design, Design Specifications of Highway Bridges, Japan Road Association (continued)

10. Strength & Ductility of RC piers
11. Demand & capacity of steel piers
12. Demand & capacity of foundations
13. Demand & capacity of abutment foundations on liquefiable ground
14. Demand & capacity of superstructures
15. Seismic design of bearings
16. Unseating prevention devices

Seismic Performance Goals

Seismic Performance Criteria

James Roberts (1999), Previous Caltran's Chief Engineer

- How do you want the structure to perform in an earthquake?
- How much danger can you accept?
- What are the reasonable alternate routes?

Japan Road Association

Ground Motion		Ordinary Bridges	Important Bridges
Function Evaluation GMs		Functional	Functional
Safety Evaluation GMs	Type-I (Kanto)	Prevent critical damage	Retain limited damage
	Type-II (Kobe)		

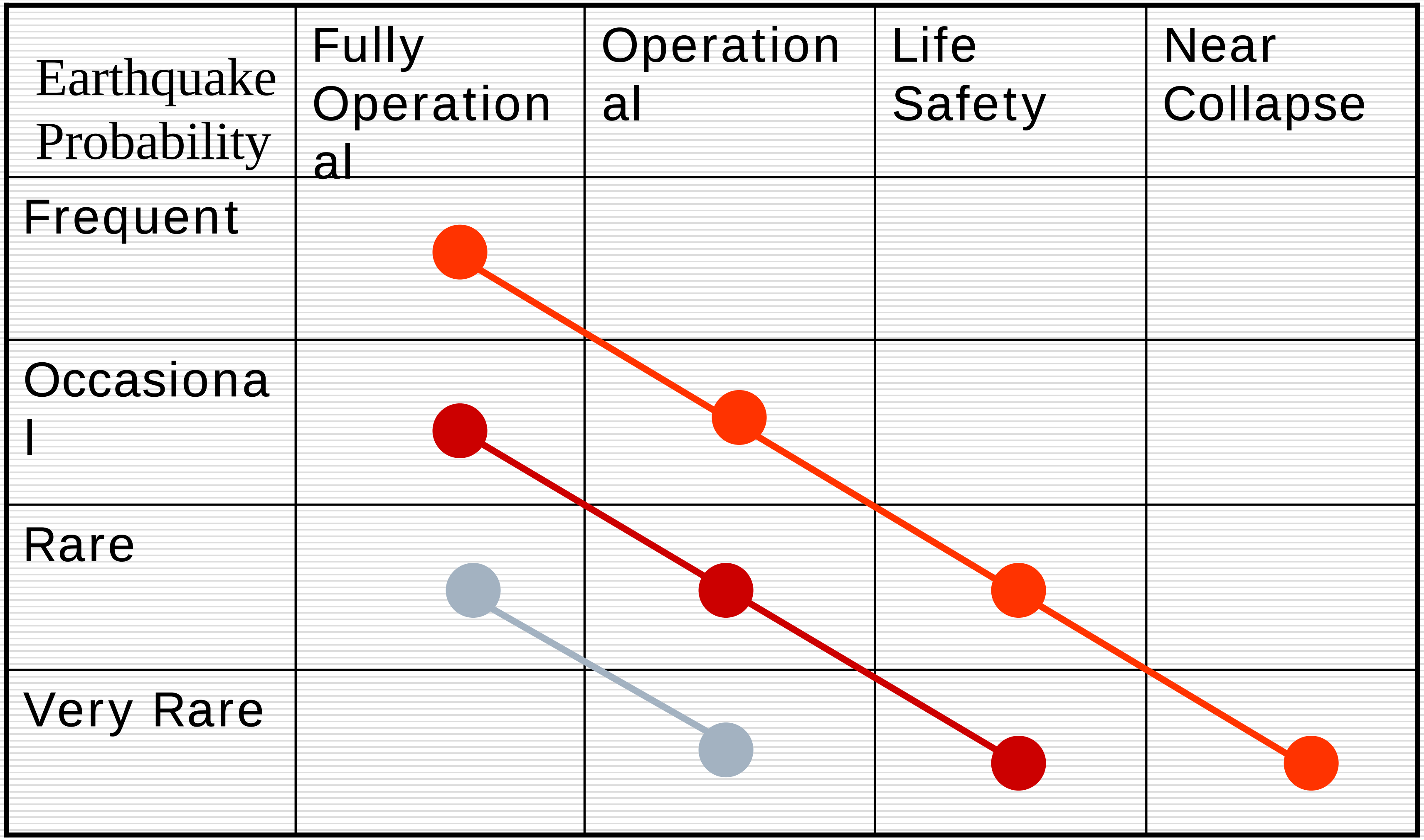
ATC and Caltrans (1999)

Ground Motions	Service Level		Damage Level	
	Ordinary	Important	Ordinary	Important
Function Evaluation	Immediate	Immediate	Reparable damage	Minimum damage
Safety Evaluation	Limited	Immediate	Significant damage	Repairable damage

Seismic Performance Goals

SEAOC Vision 2000

Performance Objective



Definition of Limit States

Limit State	Description of Damage
Fully Operational	No damage, continuous service
Operational	Most operations and functions can resume immediately. Repair is required to restore some non-essential services. Damage is light. Structure is safe for immediate occupancy. Essential operations are protected.
Life safety	Damage is moderate. Selected building systems, features or contents may be protected from damage. Life safety of generally protected. Structure is damaged but remains stable. Falling hazards remain secure. Repair possible.
Near Collapse	Structural collapse is prevented. Nonstructural elements may fall. Repair generally not be possible.
Collapse	Complete structural collapse

Probability of Occurrence of Seismic Ground Motions

Probability of occurrence of a ground motion with its intensity Y larger than y within a life time of a structure T_D

$$P(Y > y, T_D) = 1 - \{1 - P(Y > y, t = 1)\}^{T_D}$$

Return Period = Averaged Recurrence Interval

$$T_R(y) = \frac{1}{P(Y > y, t = 1)}$$

$\therefore$

$$P(Y > y, T_D) = 1 - \left\{1 - \frac{1}{T_R(y)}\right\}^{T_D}$$

$\therefore$ Probability of not being exceeded

$$P(Y < y, T_D) = 1 - P(Y > y, T_D) = \left\{1 - \frac{1}{T_R(y)}\right\}^{T_D}$$

An Exmple of Definition of Probability of Occurrence of Earthquakes

Classification of Earthquakes	Recurrent Interval	Probability of Occurrence
Frequent	43 years	70% in 50 years
Occasional	72 years	50% in 50 years
Rare	475 years	10% in 50 years
Very Rare	970 years	10% in 100 years

Recommended Maximum Transient & Permanent Drift

Vision 2000, SEAOC

Limit State	Maximum Transient Drift (%)	Maximum Permanent Drift (%)
Fully Operational	0.2	Negligible
Operational	0.5	Negligible
Life Safety	1.5	0.5
Near Collapse	2.5	2.5
Collapse	> 2.5	> 2.5

Seismic Design Force

Seismic Design Force

- Seismic design force should be determined based on the seismic environment (seismicity, fault length and rupture, etc) around the construction sites
- Seismic hazard map in terms of PGA is frequently used to scale the seismic design force.
- Response accelerations are widely used to provide the seismic design force of bridges. Multiplying it to mass, lateral force can be directly evaluated.

Seismic Design Force (continued)

- Standard response spectra are generally modified to take account of regional seismicity as

$$S_A(T,\xi)_{design} = c_z \cdot S_A(T,\xi)_{s\tan dard}$$

- Importance of structures are sometimes reflected to evaluate the design seismic force. There are two groups in this treatment as
  - ✓Larger design seismic force is considered for a structure with higher importance
  - ✓Since seismic force which applies to a structure does not change depending on the importance of the structure, the importance is considered in the evaluation of the capacity

Japan Road Association (1996 &2002)

Function Evaluation

$$S_F = k_Z \cdot k_D \cdot \begin{cases} S_1 \cdot T^{1/3} & (0 < T \le T_1) \\ S_2 & (T_1 \le T \le T_2) \\ S_3 / T & (T_2 \le T) \end{cases} \qquad (6.1)$$

Safety Evaluation

Type I GM

$$S_{SI} = k_Z \cdot k_D \cdot \begin{cases} S_4 T^{1/3} & (0 < T \le T_3) \\ S_5 & (T_3 \le T \le T_4) \\ S_6 / T & (T_4 \le T) \end{cases} \qquad (6.2)$$

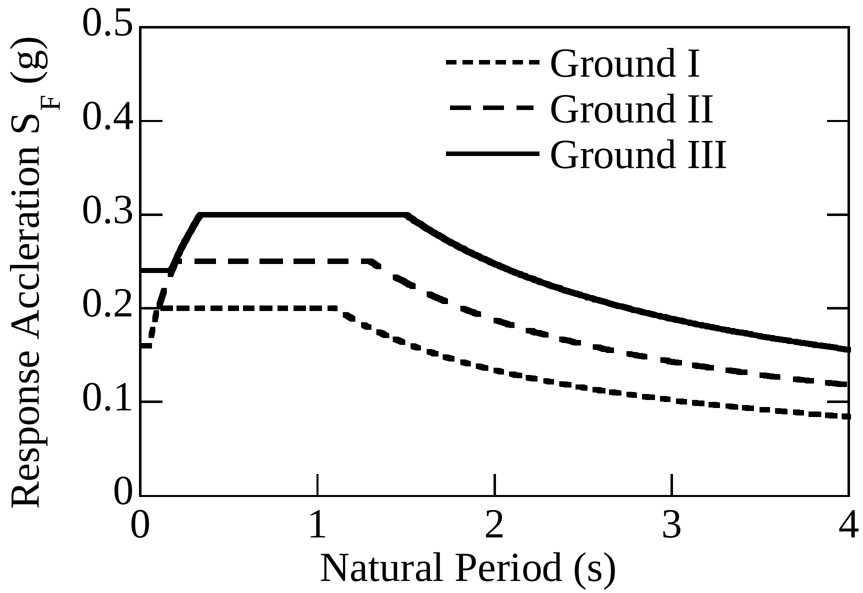
Type II GM

$$S_{SII} = k_Z \cdot k_d \cdot \begin{cases} S_7 \cdot T^{2/3} & (0 < T \le T_5) \\ S_8 & (T_5 < T \le T_6) \\ S_9 / T^{5/3} & (T_6 < T) \end{cases} \qquad (6.3)$$

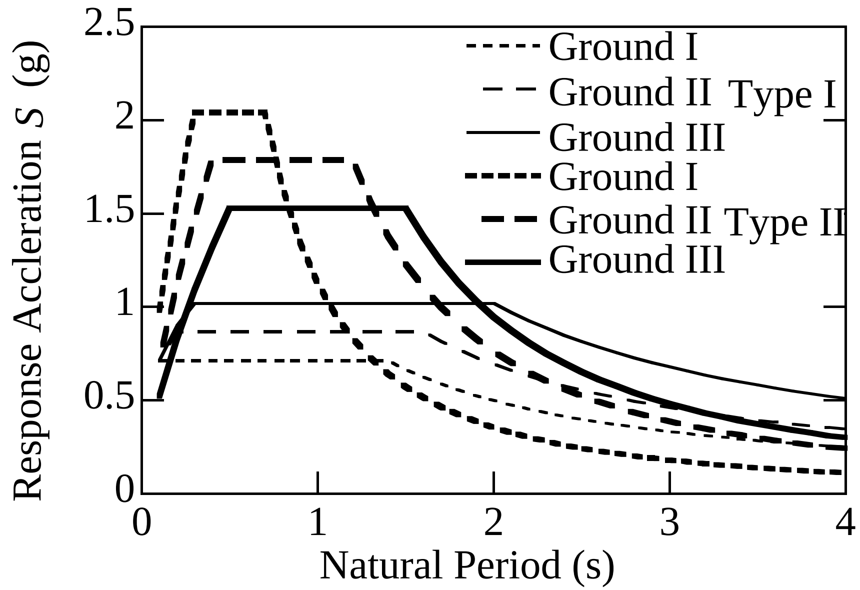
Design Specifications of Highway Bridges, Japan Road Association

$\xi = 0.05$

Function Evaluation GM



Safety Evaluation GM



Seismic Design Force

EC code

Elastic response accelerations for the referenced return period depending on soil condition

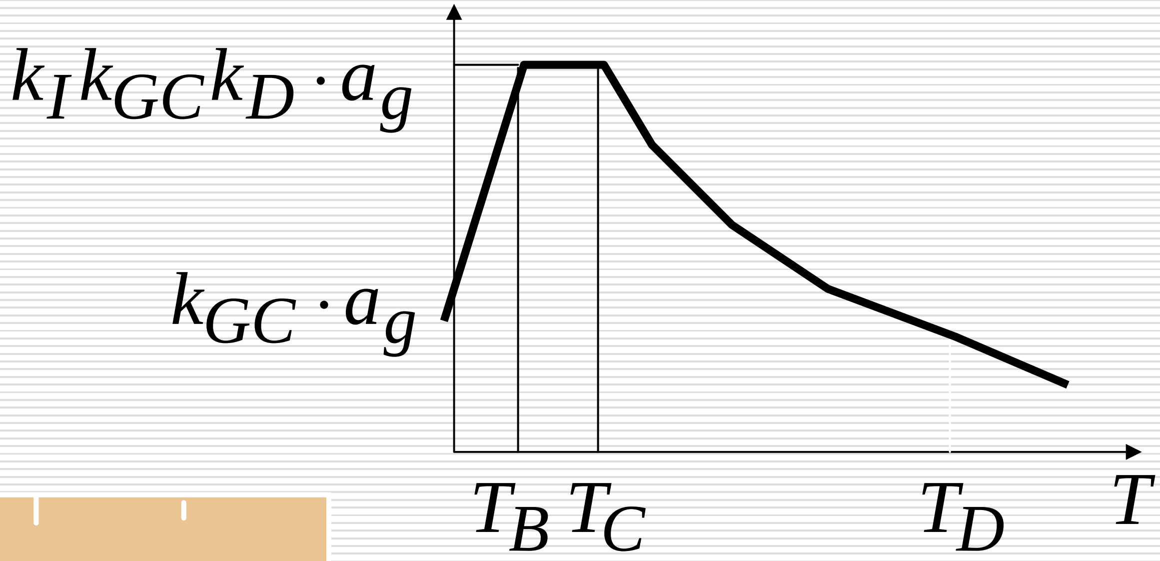
$$S = k_I \cdot k_{GC} \cdot a_g \cdot \begin{cases} 1 + \dfrac{T}{T_B}(k_D \beta_0 - 1) & 0 \le T \le T_B \\ k_D \cdot \beta_0 & T_B \le T \le T_C \\ k_D \cdot \beta_0 \cdot \left(\dfrac{T_C}{T} \right) & T_C \le T \le 3s \\ k_D \cdot \beta_0 \cdot \left(\dfrac{T_C}{3} \right) \cdot \left(\dfrac{3}{T} \right)^2 & 3s \le T \end{cases} \qquad (6.44)$$

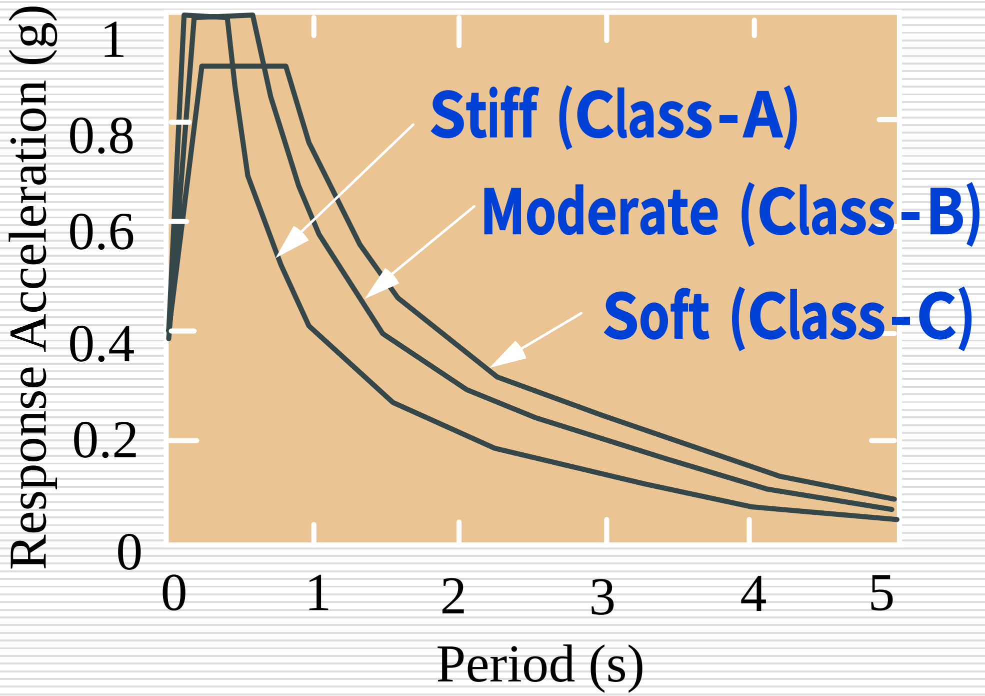
k_{GC}: 0.9 & 1.0

k_I: 1.3, 1.0, 0.7

Seismic Loads (continued)

Eurocode 8
Part " Bridges"
1994





Transit New Zealand Bridge Manual (1995)

$$S_i = k_Z \cdot k_I \cdot k_{GC} \cdot S_\mu(T)$$

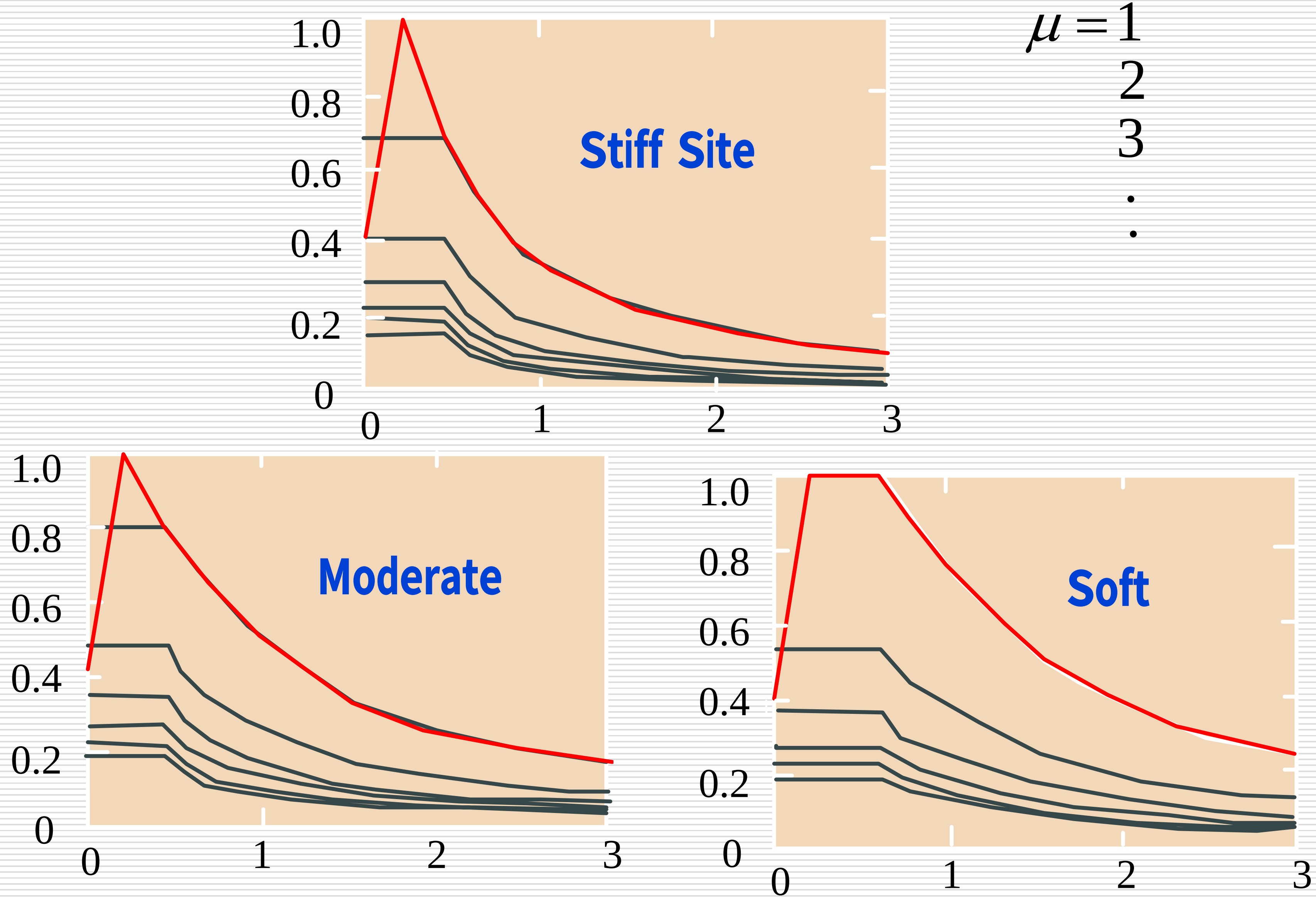
$\mu = 1$

2

3

.

.



AASHOTO

$$S = \frac{1.2 \cdot a_g \cdot k_{GC}}{T^{2/3}} \le 2.5 a_g$$

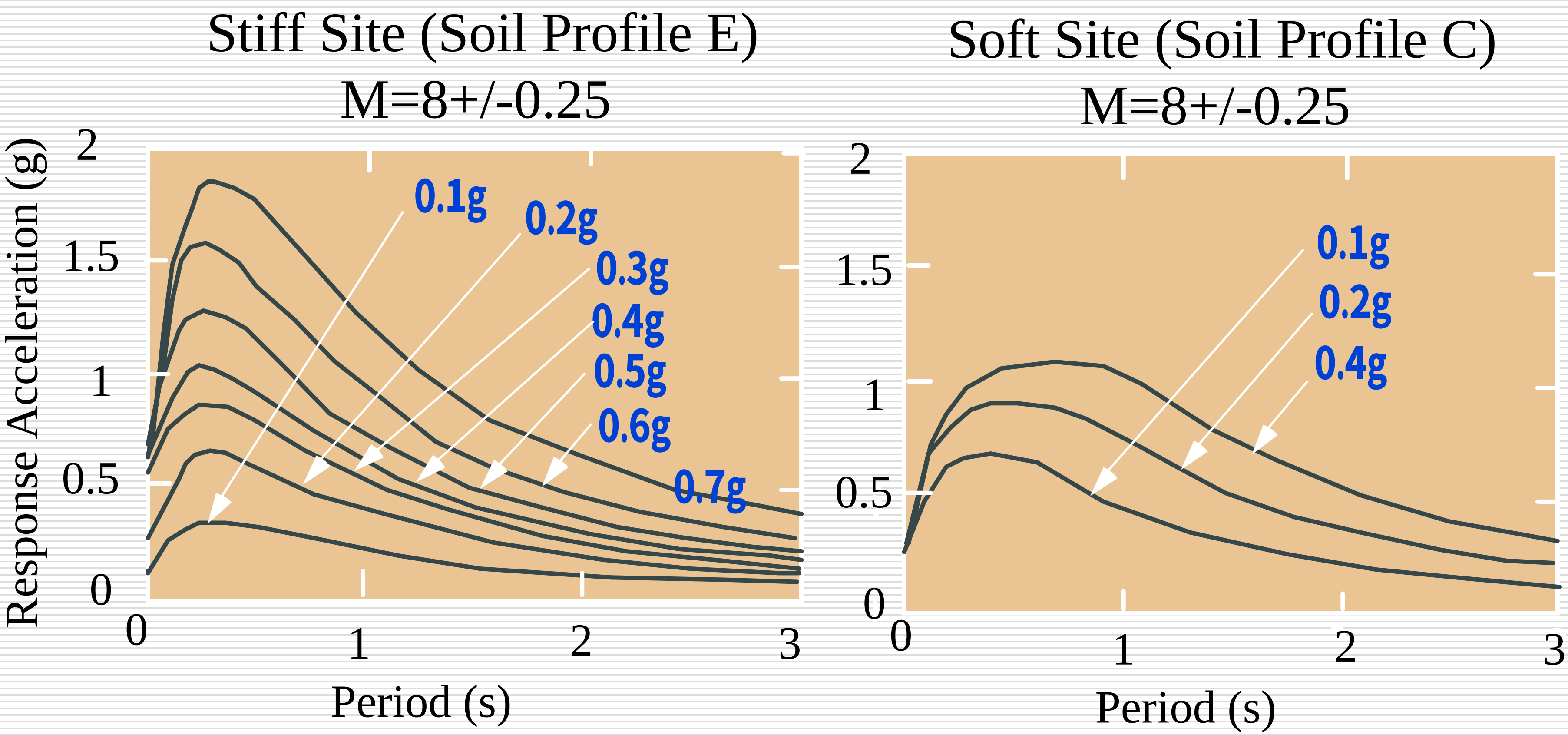
1.0, 1.2, 1.5, 2.0

Given as a contour for T_R=475 years

ATC-32 & Caltrans (1999)

Deterministic & probabilistic evaluation

T_R=1,000-2,000 years

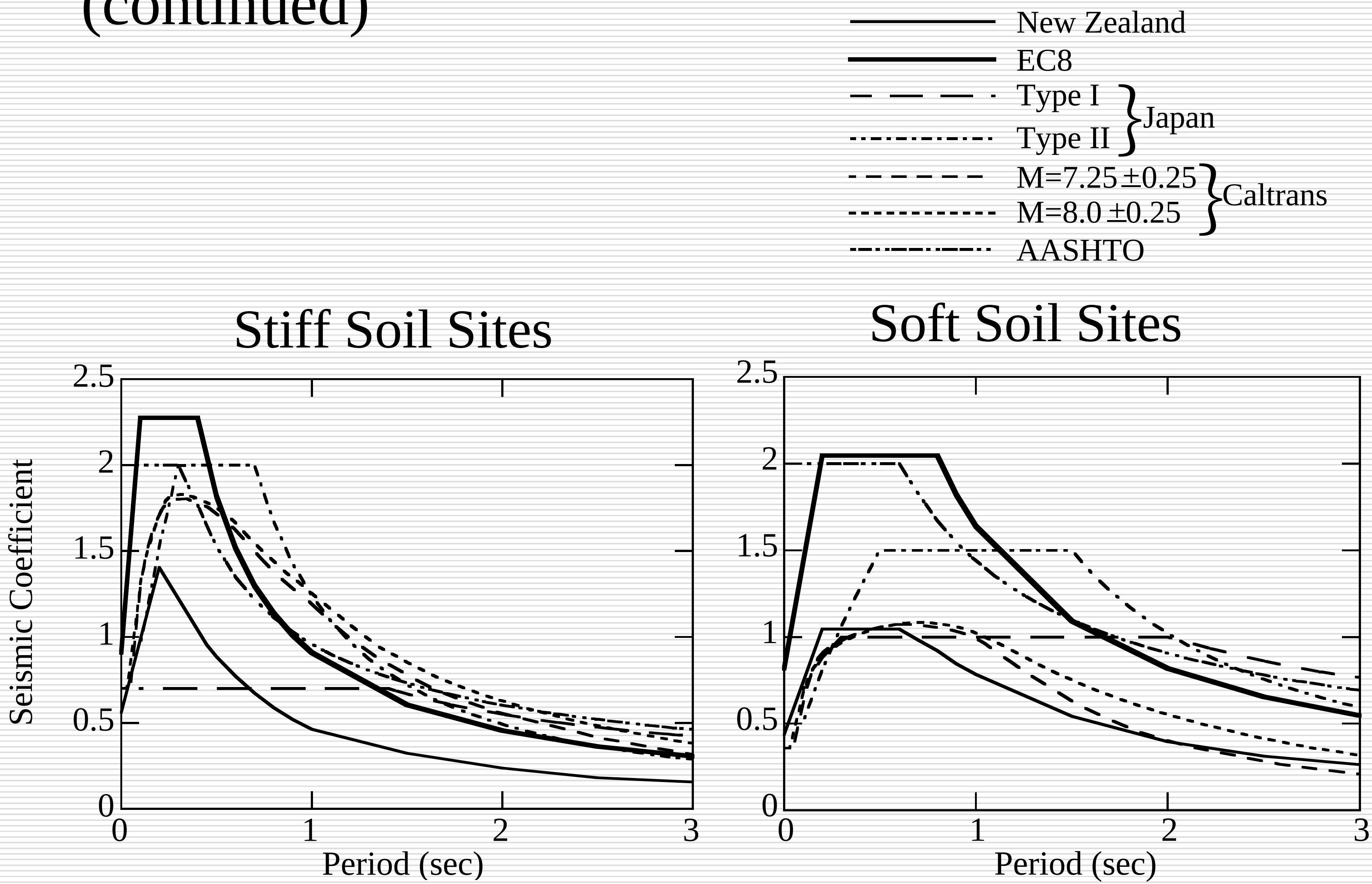


Comparison of Design Response Accelerations

AASHOTO (1995)	ATC (1996) & Caltrans (1999)	Eurcode 8 (1994)	JRA (1996 & 2002)	Transit NZ (1995)
a_g=0.8g	a_g=0.8g at stiff site	k_I=1.3	k_I=1.0	k_I=1.3
Soil profile II, III & IV	a_g=0.4g at soft site	a_g=0.7g		k_I=1.2
	M=7.25+/-0.25 M=8.0+/-0.25			
	Soil profile C, D. & E			

✓ Determine the largest elastic design response accelerations with damping ratio of 0.05 assuming the largest factor prescribed in each code

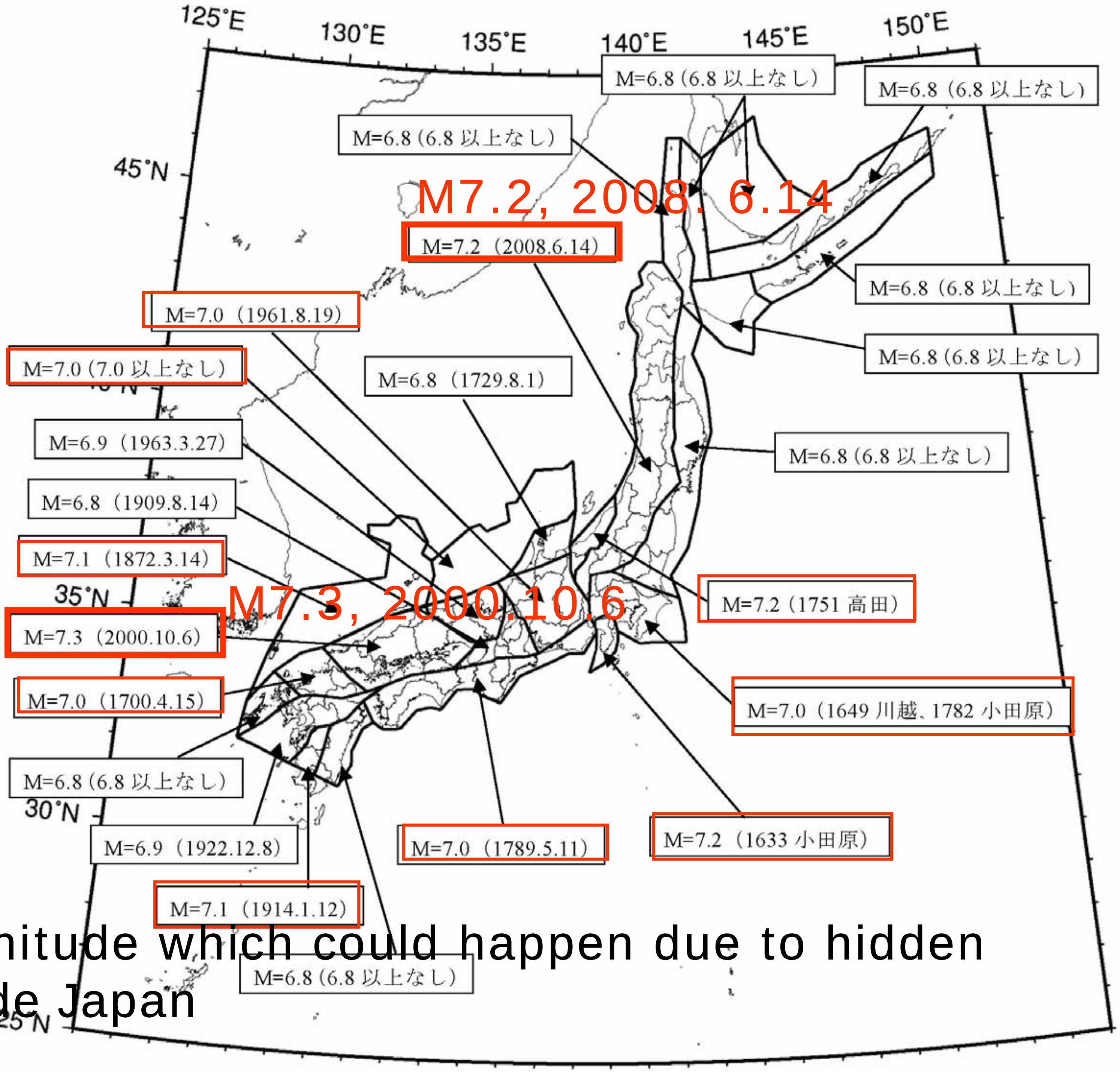
Comparison of Design Response Accelerations (continued)



Deterministic or Probabilistic?

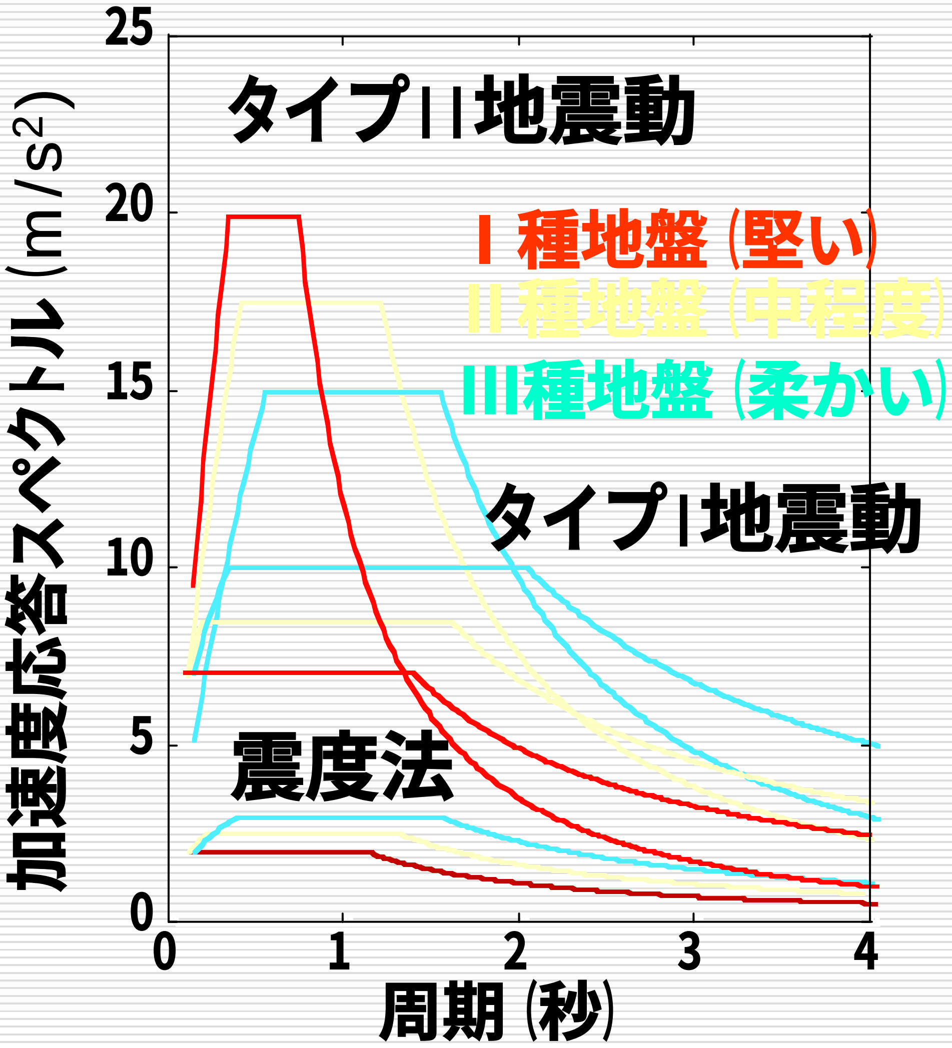
- Deterministic ground motions which occurred in Tokyo during the 1923 Kanto EQ and in Kobe during the 1995 Kobe EQ is used with modification of regional

陸域の震源断層を予め特定しにくい地震の最大マグニチュード 防災科学技術研究所 (2009)



Max. magnitude which could happen due to hidden faults inside Japan

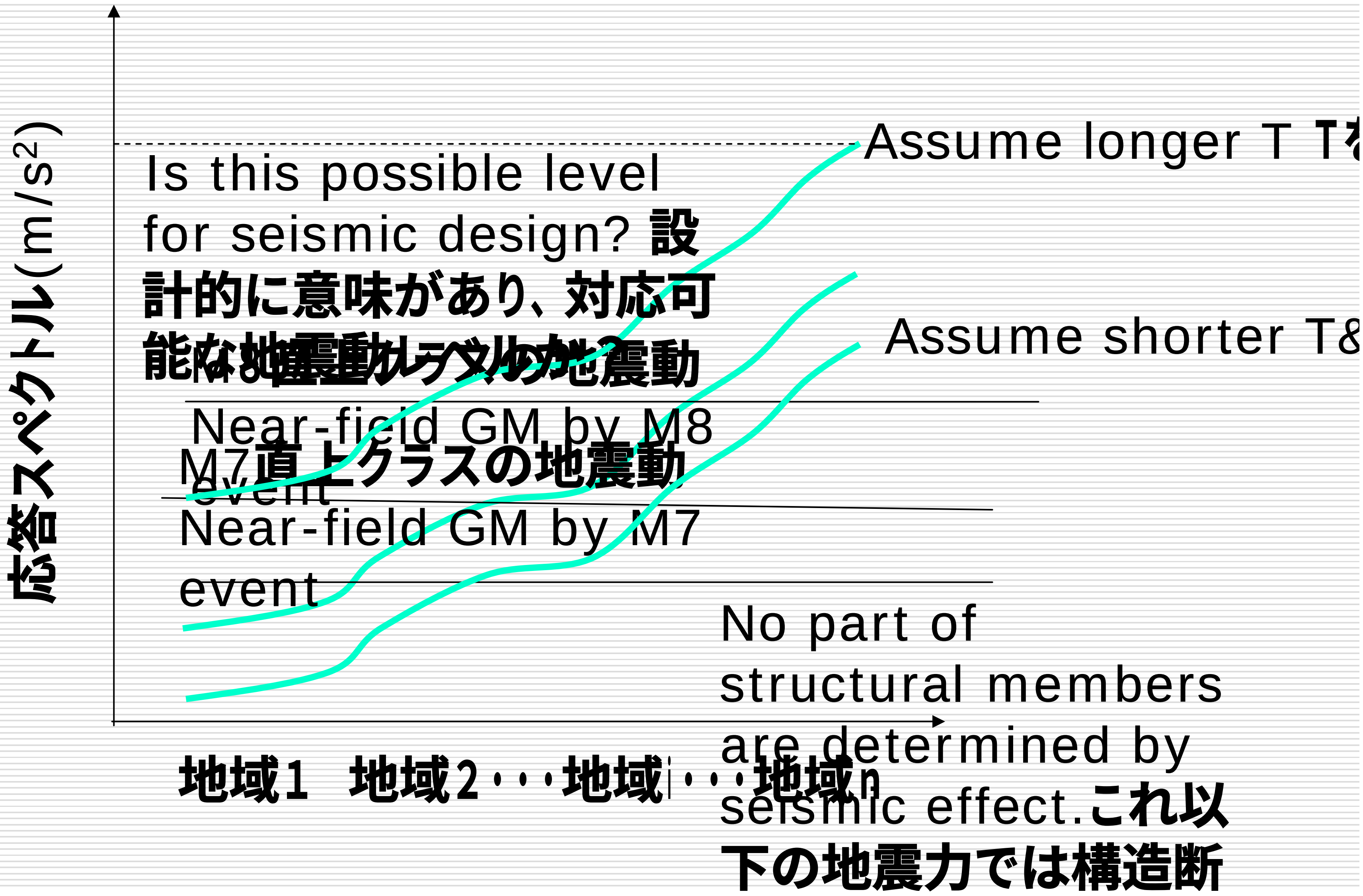
M_J7.3という直下型地震による地震動は、道路橋示方書のタイプII地震動と同レベル



Ground motions generated by a MJ7.3 is the same level with the ground motions during the 1995 Kobe earthquake.

再現期間T年に相当する応答スペクトル概念図

Probabilistic response acceleration levels



海外では、確率論的地震動評価に基づいてそのまま地震動を定める事例が多いが、・・・・

●米国やヨーロッパのように、地震活動が活発な地域からほとんどない地域まで、広範囲に地震動強度を定めるためには、確率論的地震動は便利

●米国でも、カリフォルニア州等、西海岸の一部でしか耐震設計は支配的ではない。将来、中西部に被害地震が生じ、これによる地震動が確率論的地震動を上回った場合には、当然、いろいろな議論が起こるだろう。ヨーロッパも同じ。

●地震が起こらない地域はないと国民が理解している我が国では、安易に米国やヨーロッパの考え方を踏襲できない。

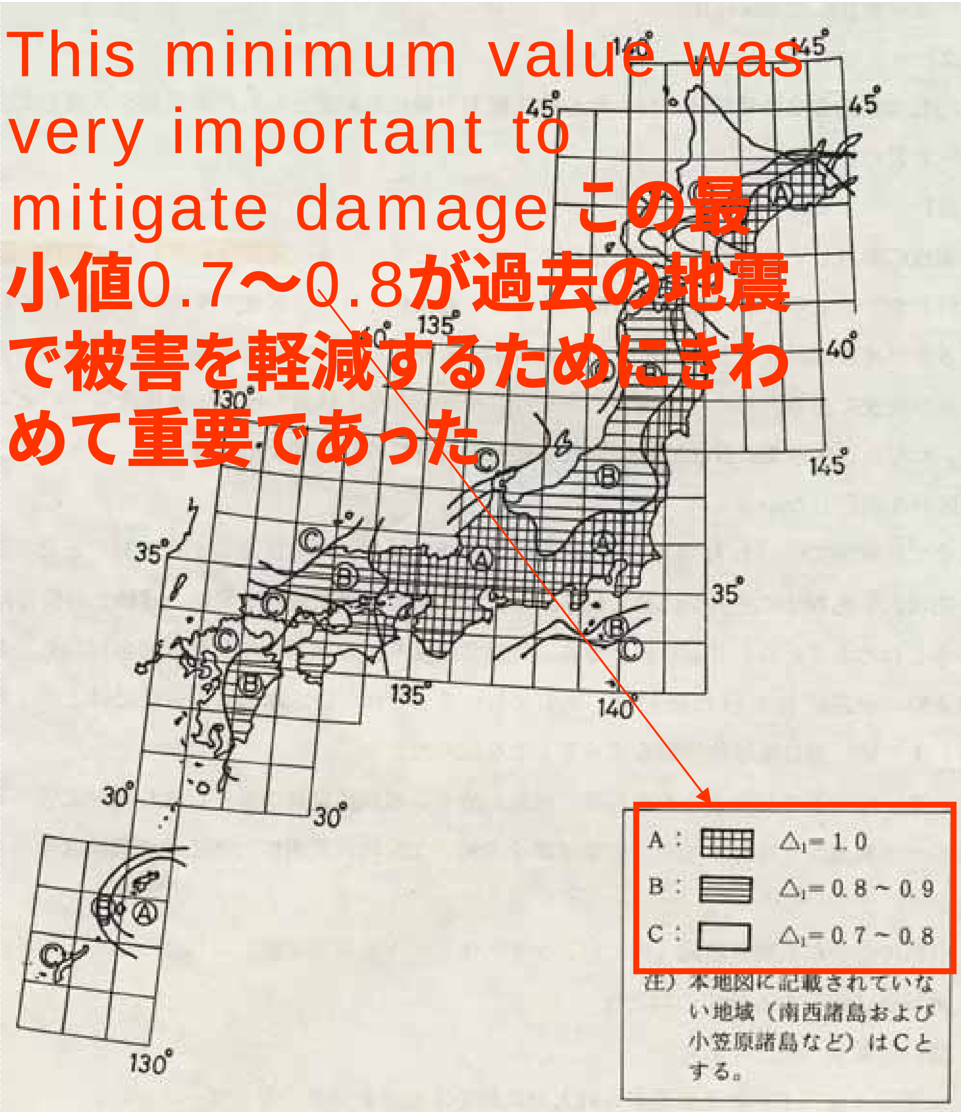
一般構造物（特殊な構造物を除く）では、確率論的地震動マップをどのように利用可能か？

- 確率論的地震動マップにしたがって、地震動強度を求める（米国やECではこの考え方に近い）
- 確率論的地震動マップから相対的な地域区分を定め、これに工学的判断を加えて、地震動強度を評価（現在の多くの技術基準の考え方）
- 確率的地震動マップによる地震動に、最小値の足切りを加える
  - ✓M_J7.2直上の地震動を考慮すれば、事実上、これで設計地震動が決定される。
  - ✓再現期間を少し長くすると、M_J8クラスの地震が発生する地域があり、これが設計地震動とならざるを得ない。

Seismic Zoning Map widely accepted in Japan since 1983

昭和53年3月

This minimum value was very important to mitigate damage この最小値0.7〜0.8が過去の地震で被害を軽減するためにきわめて重要であった

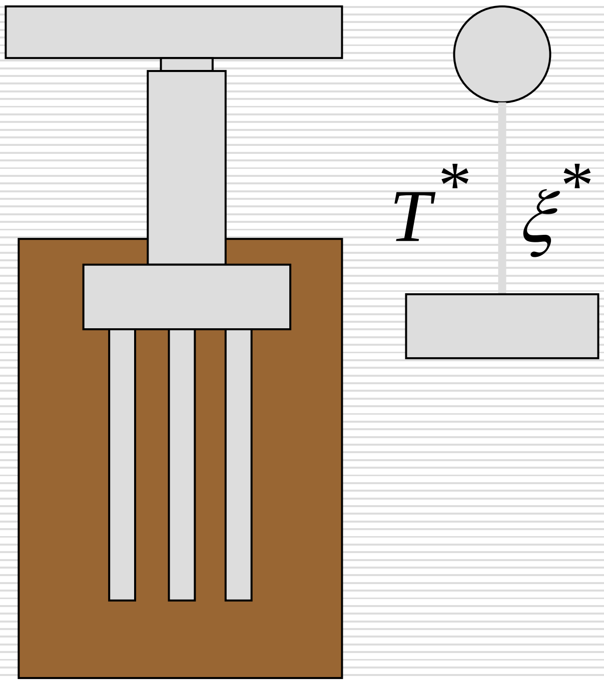


- ●当時までに公表されていた地震危険度に関する12例の研究に加重平均を加えて求められた再現期間100年の地震危険度区分
- ●現在に至るまで、土木、建築分野で、広く採用されてきている

Evaluation of Demand

Evaluation of Demand

Force based seismic design



- Nonlinear dynamic response analysis
- Static analysis

$$D_{ave} = \frac{m \cdot S_A(T^*, \xi^*)}{R}$$

Response modification factor

$$R = \begin{cases} \sqrt{2\mu_r - 1} \\ \mu_r \\ \text{.........Empirical values} \end{cases}$$

Japanese practice to consider damping ratio in the evaluation of elastic load

Since μ_r is not known at the first stage of the design, the response modification factor is evaluated as

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

Design displacement ductility factor

$$\because \ \mu_r \approx (<) \mu_a$$

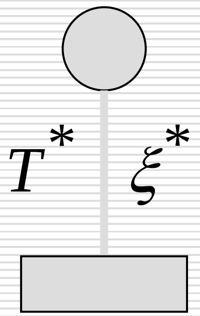
$$\gamma \cdot D_{ave} \leq \phi \cdot C_{ave}$$

Where do we consider the damping characteristics of the bridge in the static design?

How should we incorporate the damping ratio of the bridge in the static seismic design?

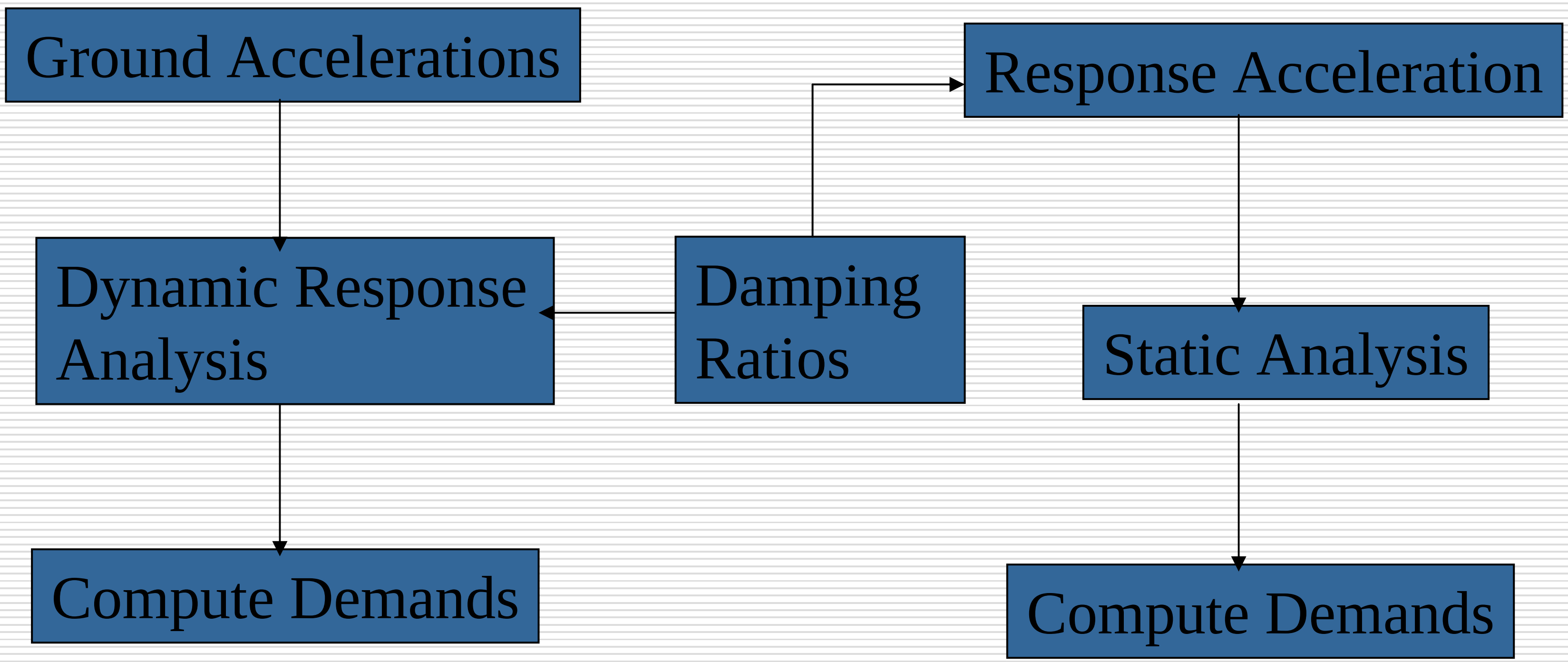
Dynamic Analysis

Static Analysis

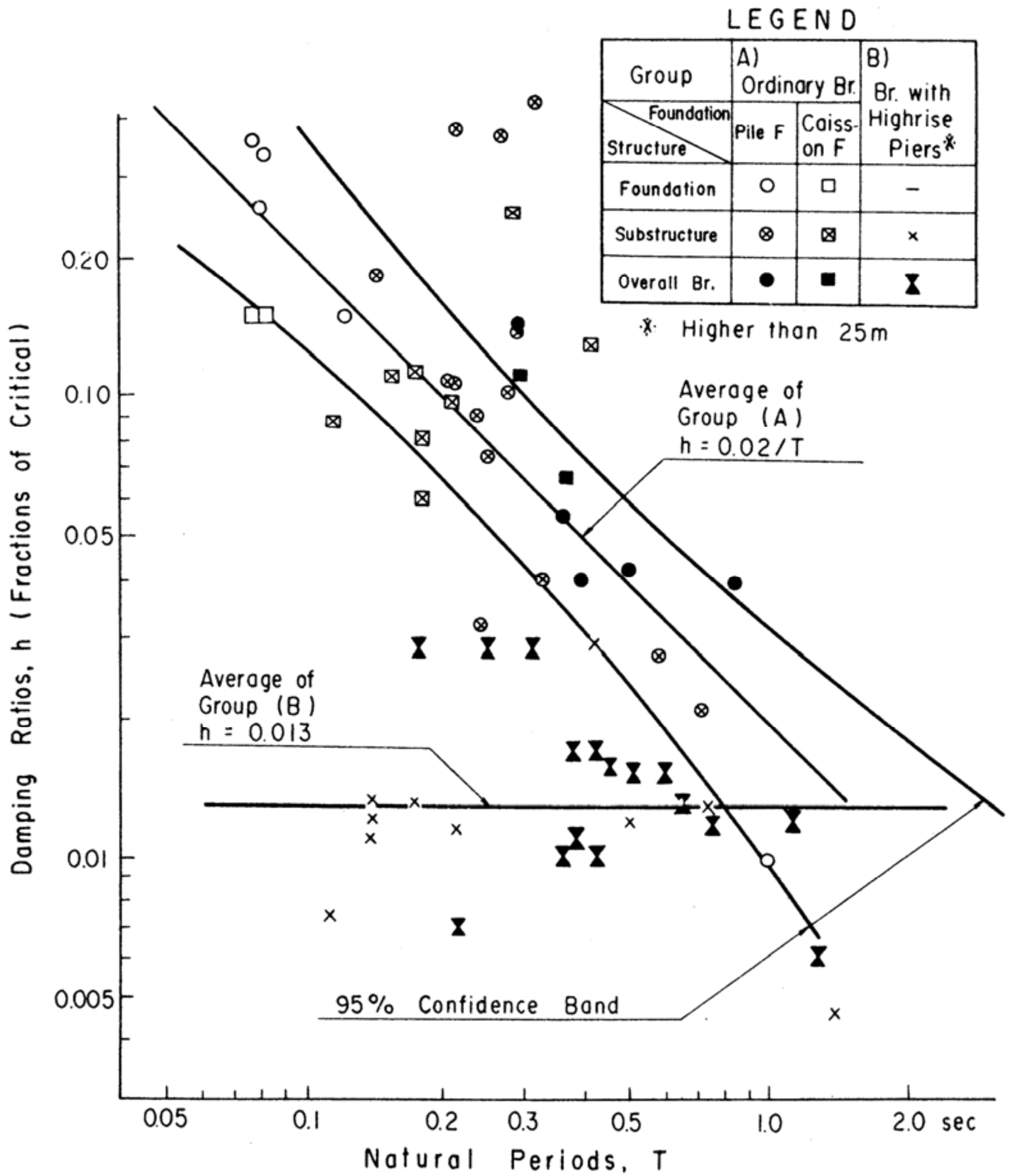


$$D = \frac{m \cdot S_A(T^*, 0.05)}{R}$$

$$D = \frac{m \cdot S_A(T^*, \xi^*)}{R}$$



How can we estimate the damping ratio of bridges?

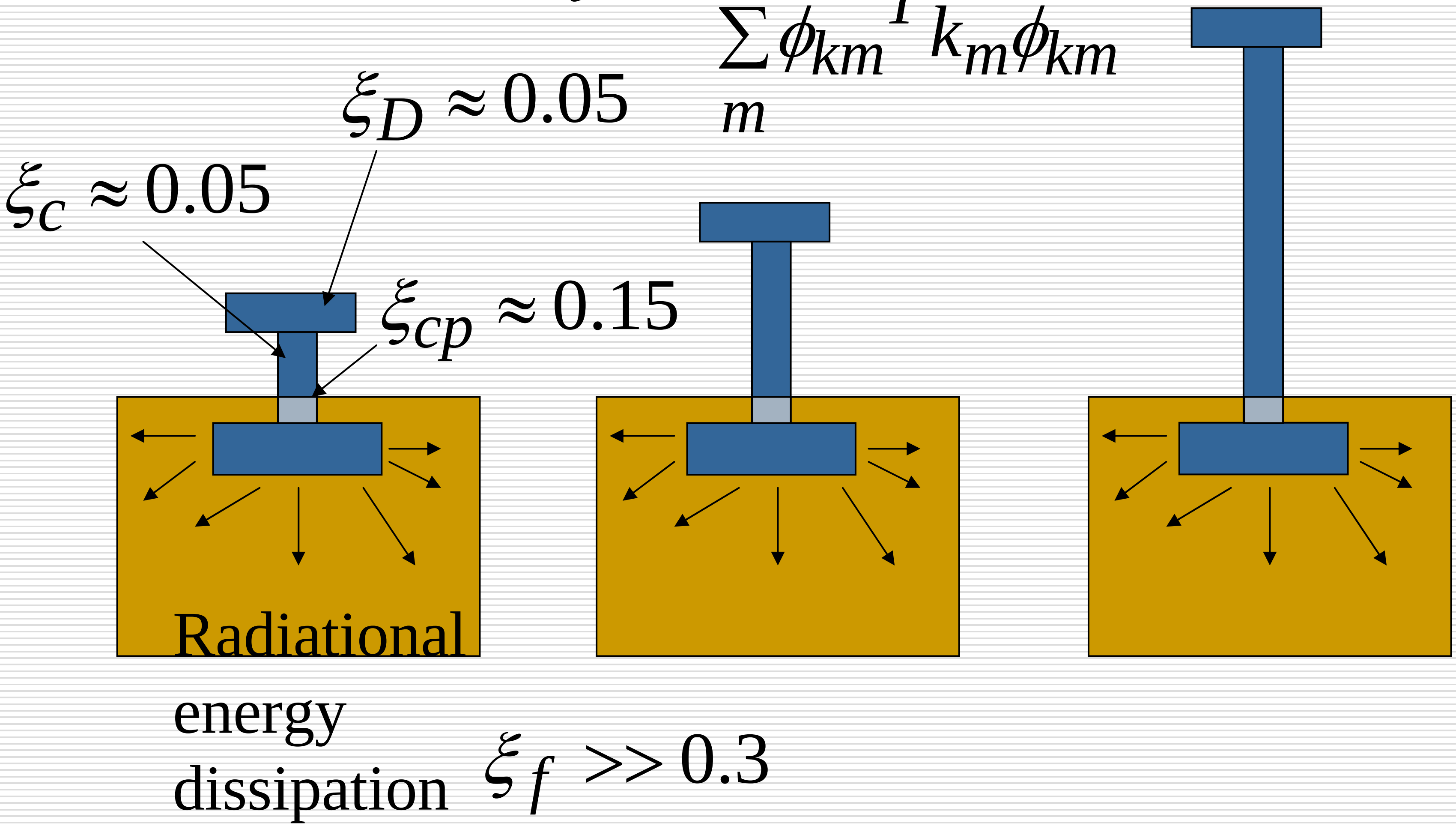


- Empirical relation on the damping ratio vs. fundamental natural period of bridges
- This is based on force excitation tests on bridges supported by various types of foundations

$$\xi = \frac{0.02}{T}$$

Why is the damping ratio inversely proportional to the fundamental natural period?

$$\xi = \frac{\sum_m \xi_m \phi_{km}^T k_m \phi_{km}}{\sum_m \phi_{km}^T k_m \phi_{km}}$$



How should we incorporate the damping ratio of the bridge in the static seismic design?

Damping ratio of the bridge should be incorporated in the evaluation of response accelerations used in the static analysis

Not $S_A(T^*, 0.05)$ but $S_A(T^*, \xi^*)$

$$S_A(T^*, \xi^*) = S_A(T^*, 0.05) \times c_D(\xi^*)$$

where

$$c_D(\xi^*) = \frac{1.5}{40\xi^* + 1} + 0.5$$

$$\xi^* = \frac{0.02}{T^*}$$

How should we incorporate the damping ratio of the bridge in the static seismic design?

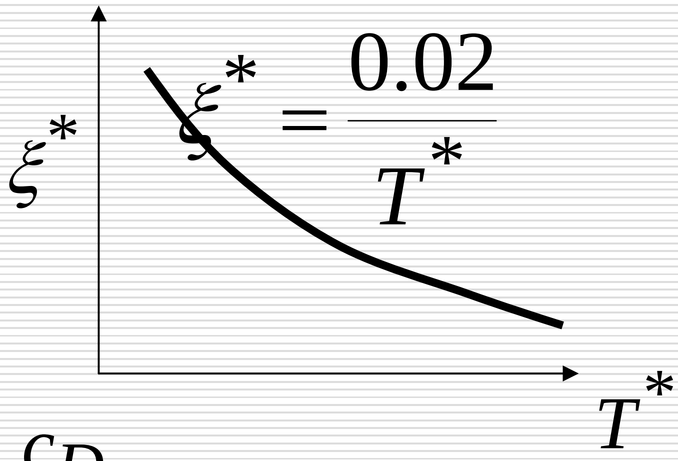
$$S_A(T^*,\xi^*) = S_A(T^*,0.05) \times c_D(\xi^*)$$

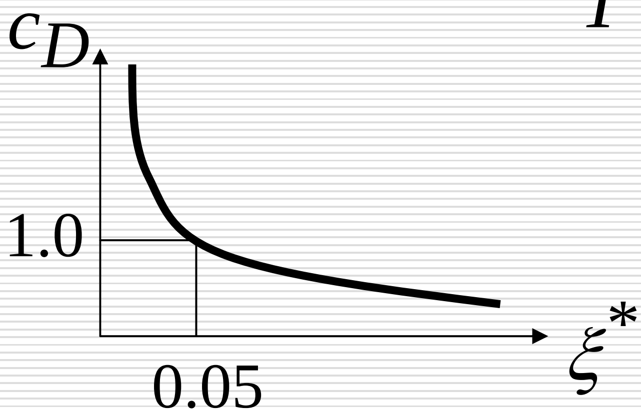
$$= S_A(T^*,0.05) \times \left\{ \frac{1.5}{\frac{0.8}{T^*}+1} + 0.5 \right\}$$

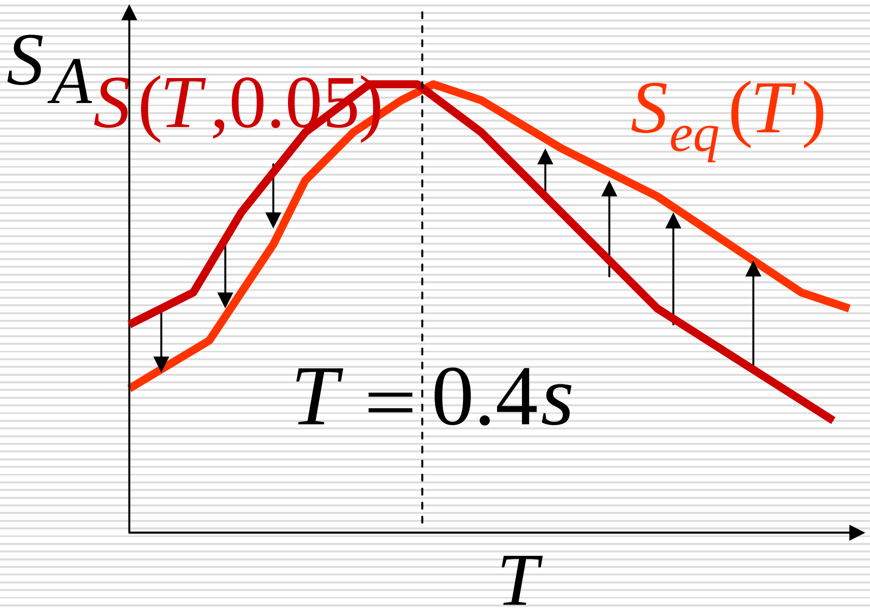
How should we incorporate the damping ratio of the bridge in the static seismic design?

$$c_D\left(\xi = \frac{0.02}{T}\right) = \frac{1.5}{\frac{0.8}{T} + 1} + 0.5$$

$$\xi^* = \frac{0.02}{T^*}$$







Japanese practice to take the damping ratio of bridge into account in the static design force

Safety Evaluation Ground Motion

Type II GM

Dynamic response analysis $S_A(T, 0.05)$

$$S_{SII} = c_Z \cdot c_D \cdot \begin{cases} S_7 \cdot T^{2/3} & (0 < T \leq T_5) \\ S_8 & (T_5 < T \leq T_6) \\ S_9 / T^{5/3} & (T_6 < T) \end{cases}$$

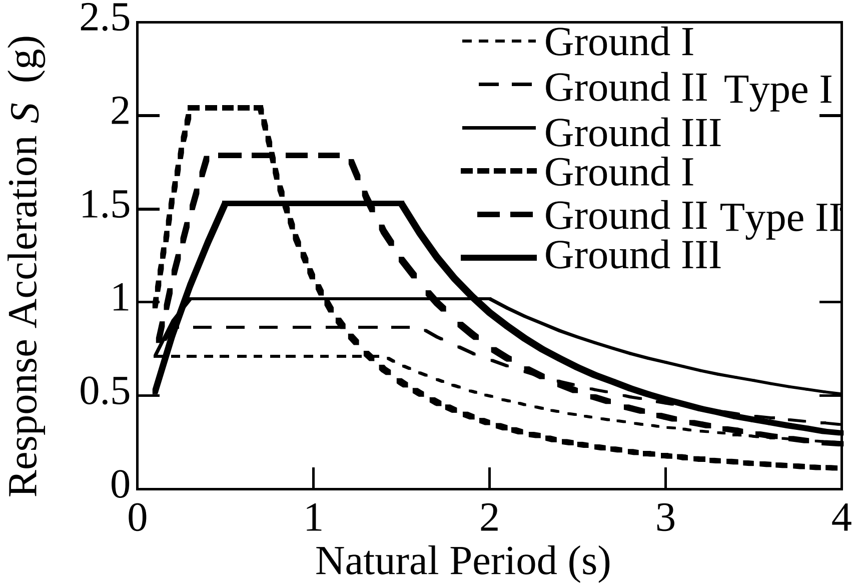
Static analysis $S_A(T, 0.02/T)$

$$S_{esSII} = c_Z \cdot \begin{cases} S_{s7} T^{2/3} & 0 < T \leq T_5 \\ S_{s8} & T_5 \leq T \leq T_6 \\ S_{s9} / T^{4/3} & T_6 \leq T \end{cases}$$

Japanese practice to take the damping ratio of bridge into account in the static design force

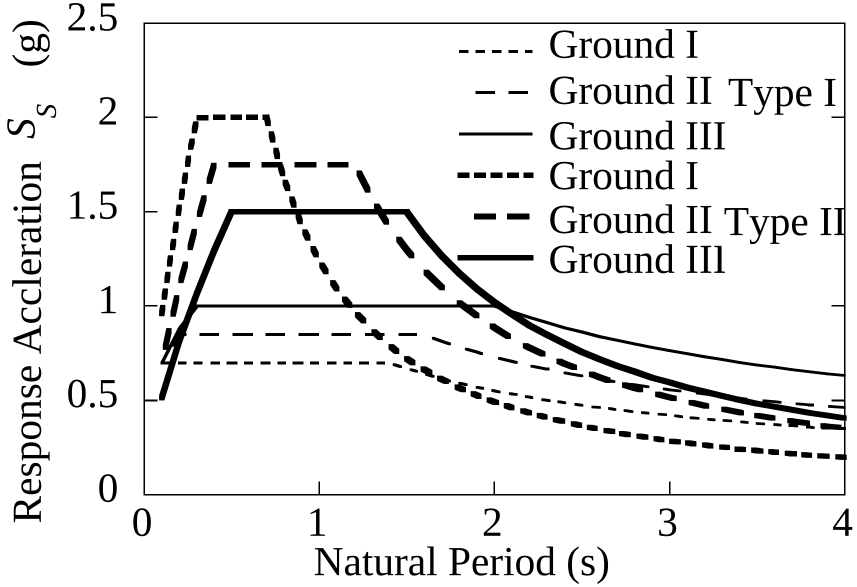
Dynamic response analysis

$$S_A(T, 0.05)$$



Static analysis

$$S_A(T, 0.02/T)$$

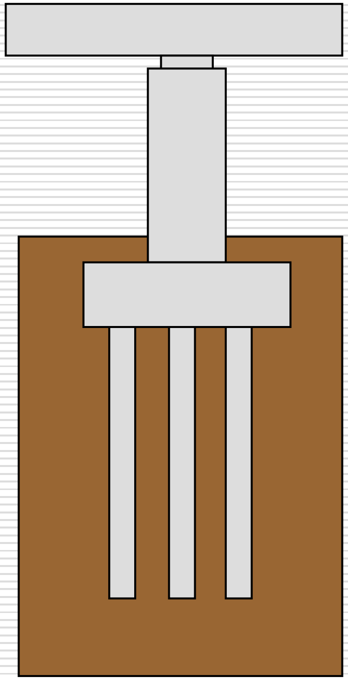


Features of the Japanese Practice in the Evaluation of Design Seismic Forces

- ✓ Explicit Two Level Design Forces are used
- ✓ Near field GMs and Middle field GMs resulted from M 8 EQs are used for the safety evaluation GMs
- ✓Importance is accounted for not in the evaluation of design ground motions but in the evaluation of design ductility factors
- ✓ A damping force vs. natural period relation is included in the design seismic forces for static analysis

Evaluation of Capacity

Evaluation of Capacities

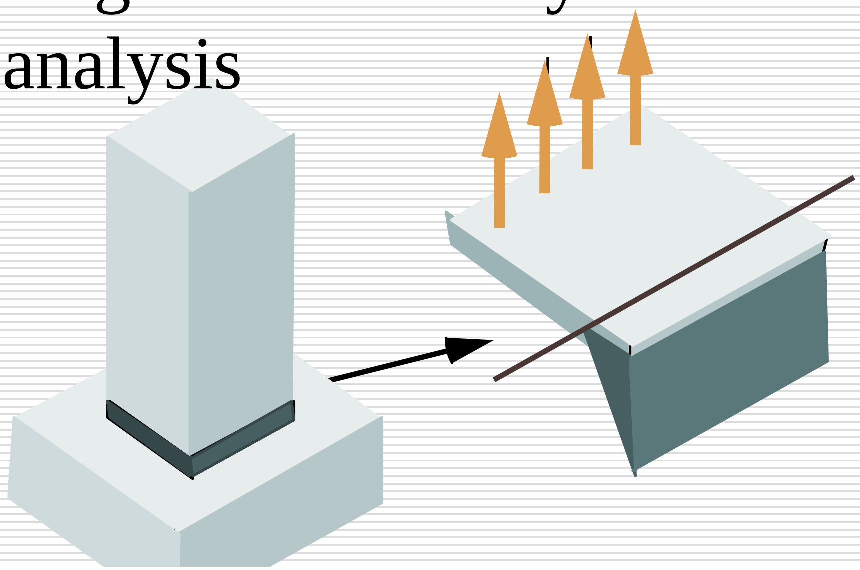


- ✓ Deck
- ✓ Bearings
- ✓ Columns/Piers
- ✓ Foundations
- ✓ Stability of foundations
- ✓ Surrounding ground

Evaluation of Capacities (continued)

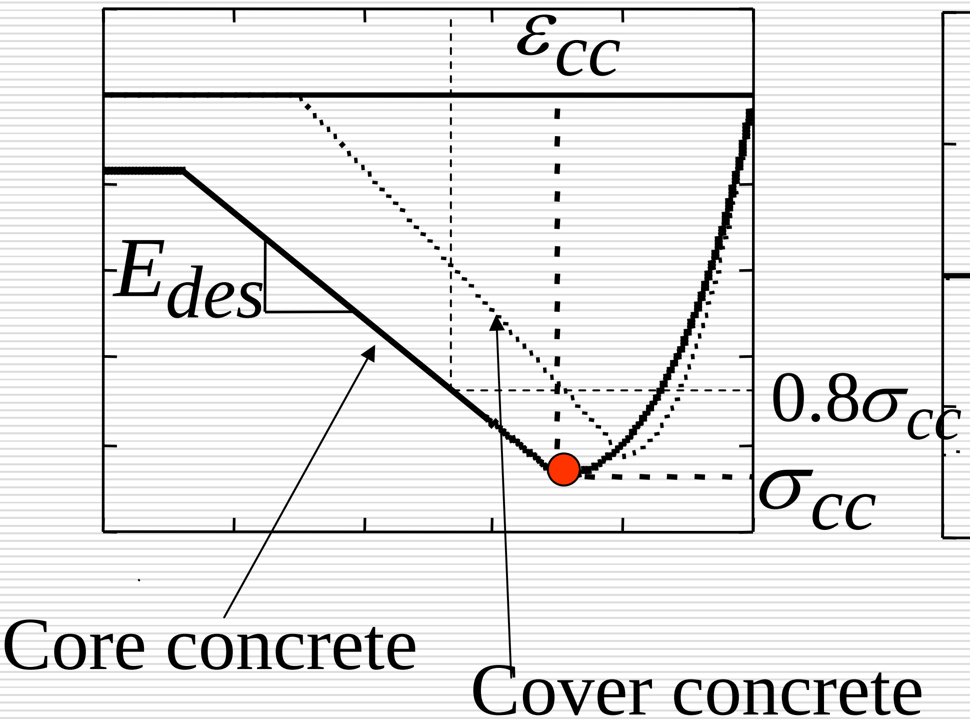
Reinforced concrete columns/piers

1) Assume a section based on the requirement for static loads
2) Evaluate tie reinforcement ratio ρ_s
3) Evaluate stress vs. strain relation of confined concrete and reinforcing bars
4) Evaluate strength and ductility of columns using the fiber element analysis

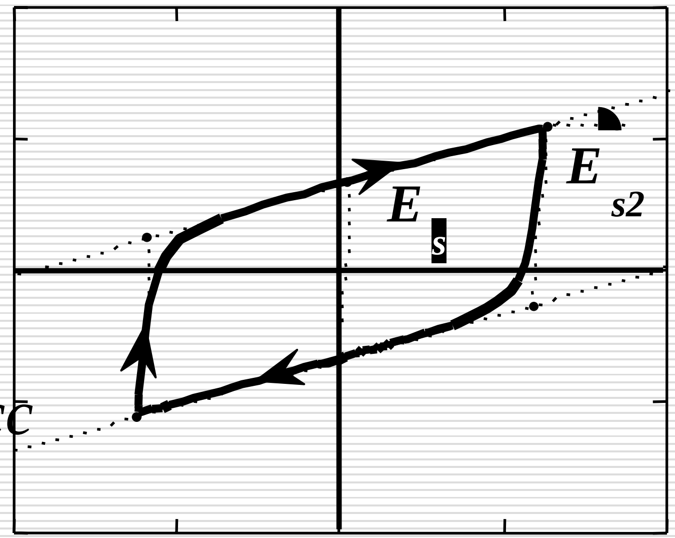


Appropriate Idealization of Hysteresis of Confined Concrete and Reinforcing Bars

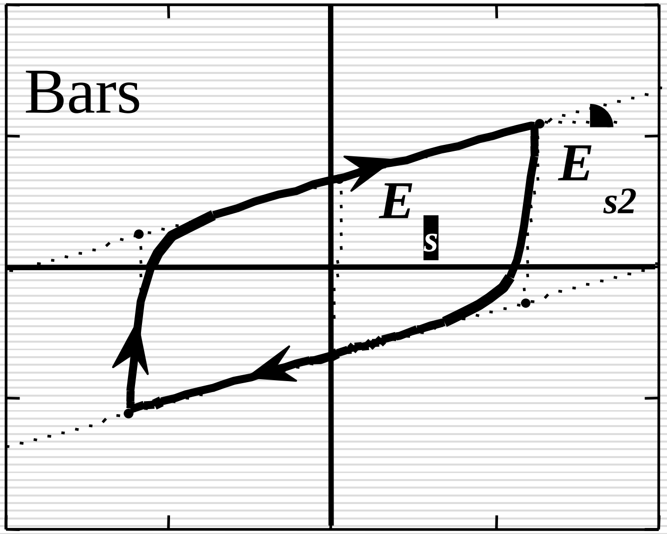
Confined Concrete

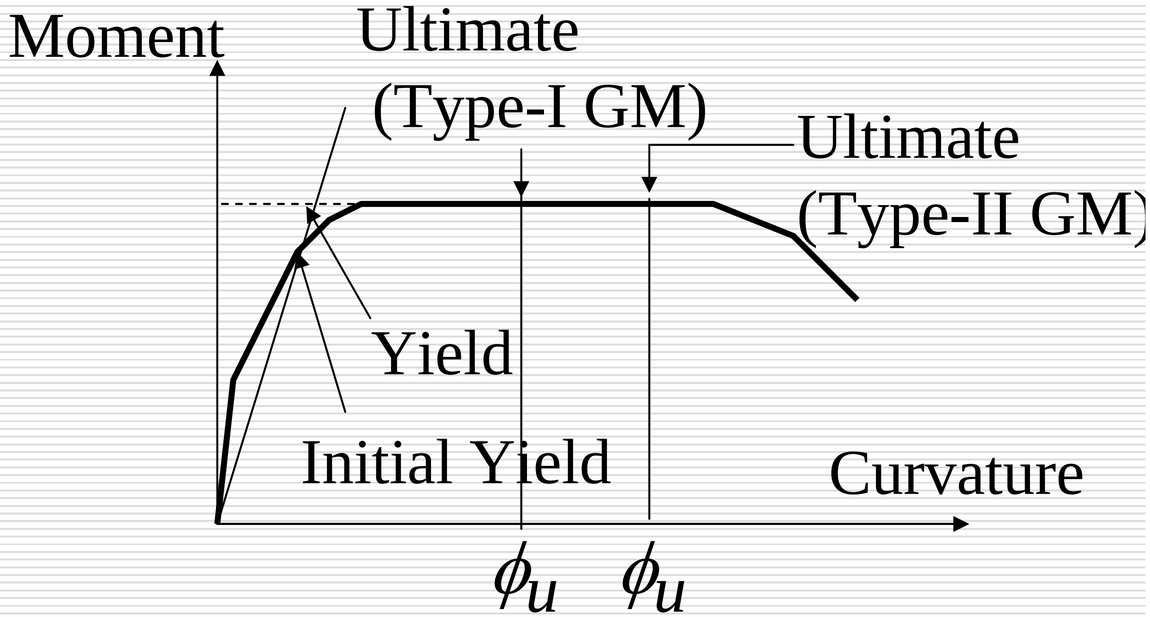


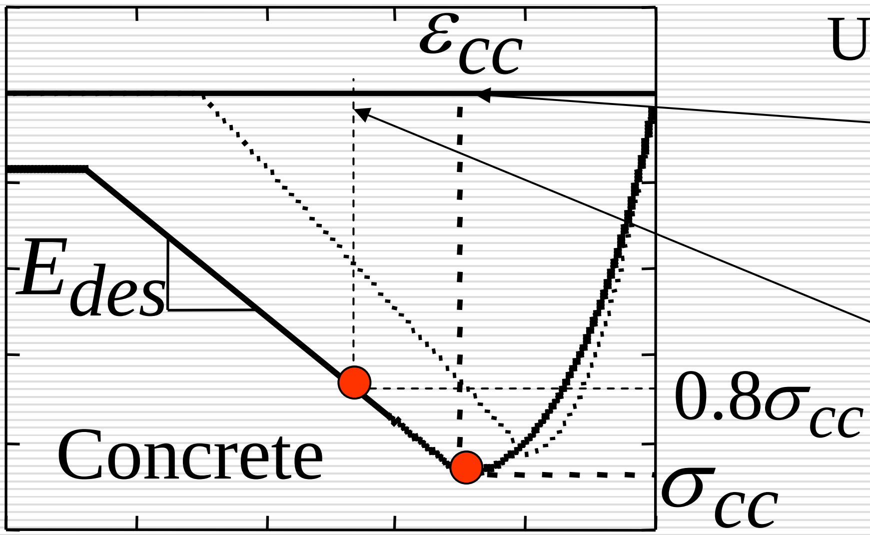
Rebars



Evaluate the Yield & Ultimate Curvature







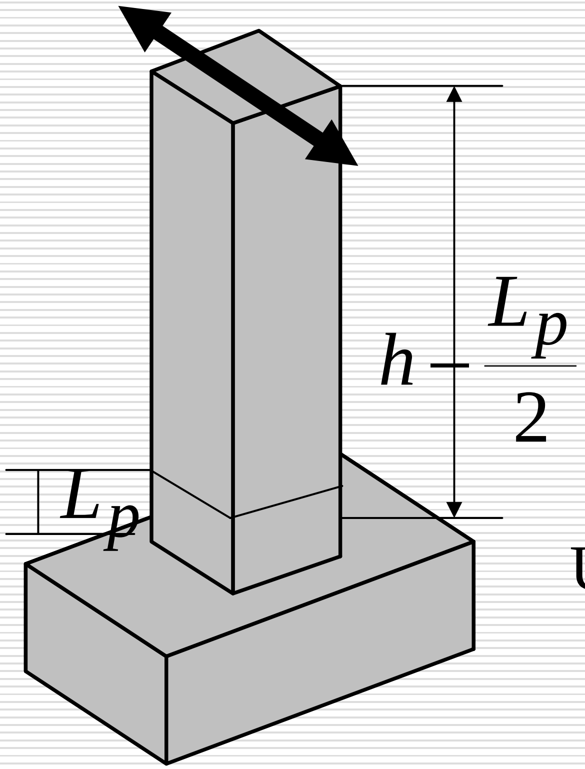
Ultimate Strain of Confined Concrete

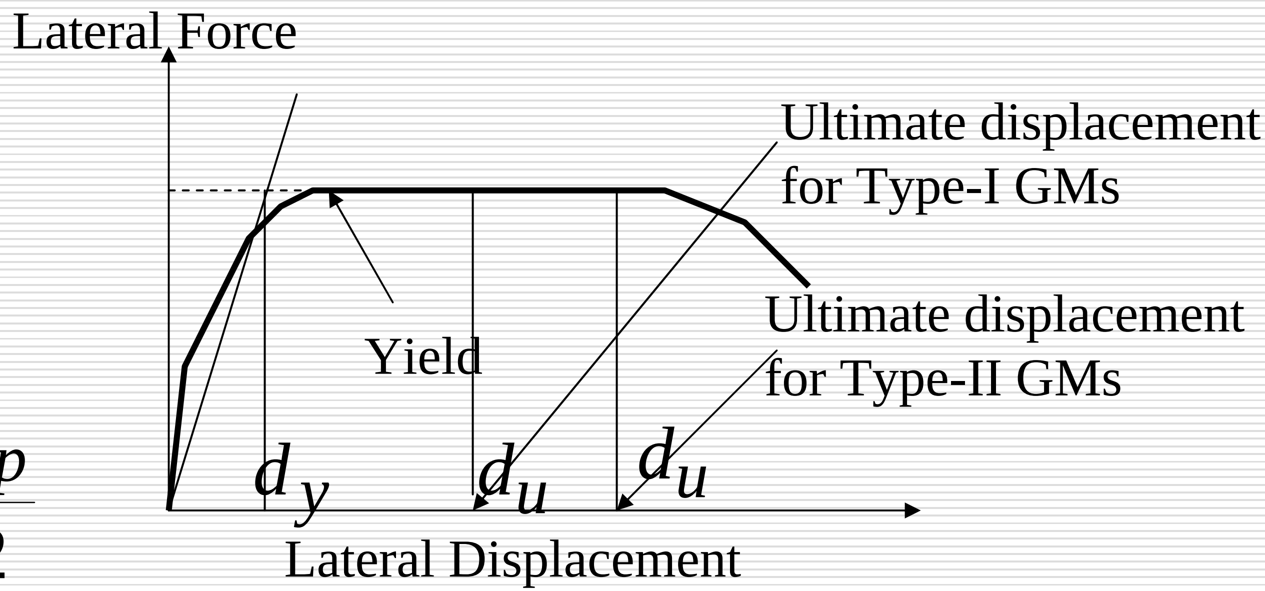
Type-I GMs

$$\varepsilon_{cu} = \begin{cases} \varepsilon_{cc} \\ \varepsilon_{cc} + 0.2 f_{cc} / E_{des} \end{cases}$$

Type-II GMs

Evaluate the Yield & Ultimate Lateral Displacement





Ultimate displacement

$$d_u = d_y + (\phi_u - \phi_y)(h - \frac{L_p}{2})L_p$$

Design displacement

$$d_a = d_y + \frac{d_u - d_y}{\alpha}$$

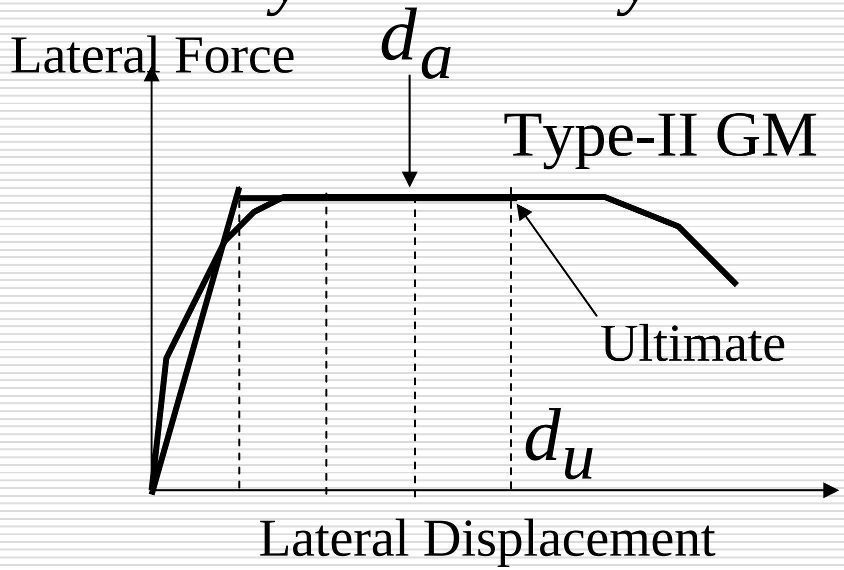
Design displacement

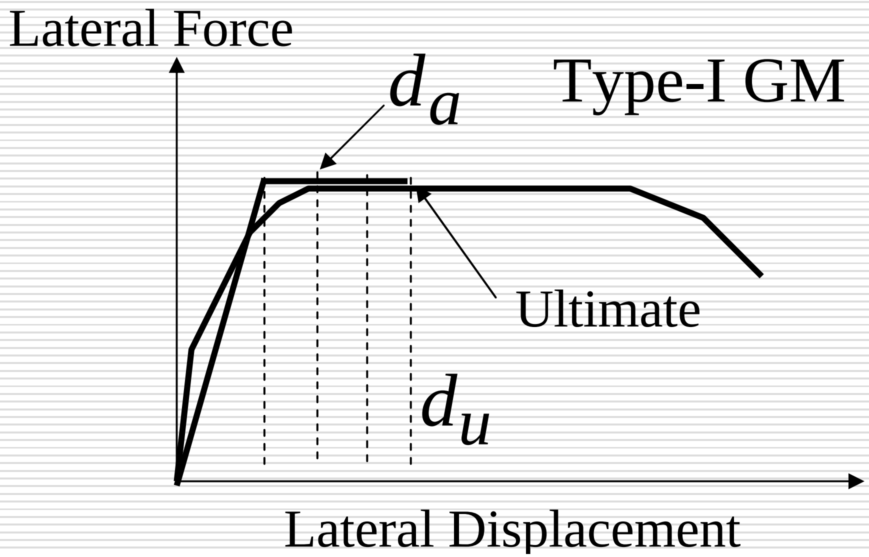
$$d_a = d_y + \frac{d_u - d_y}{\alpha}$$

Design displacement ductility factor

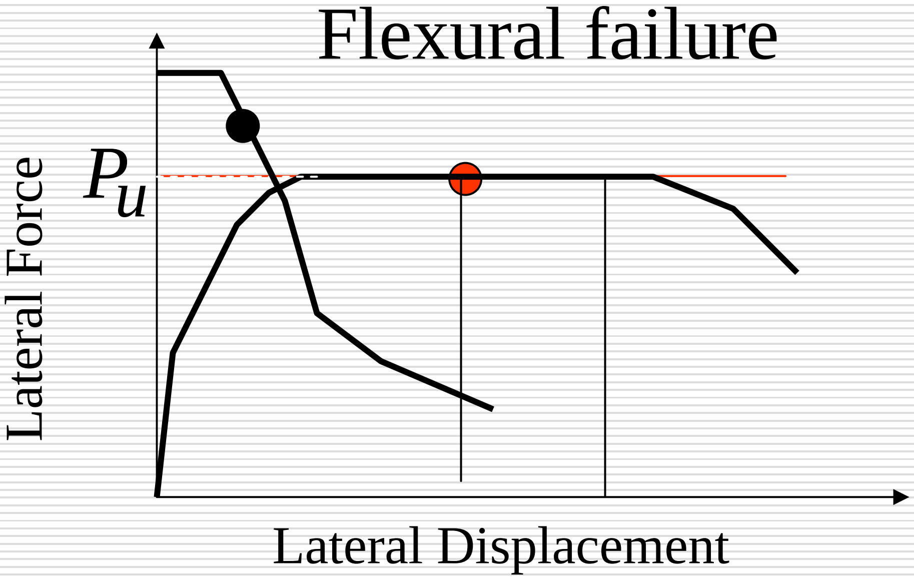
$$\mu_a = \frac{d_a}{d_y} = 1 + \frac{d_u - d_y}{\alpha d_y}$$

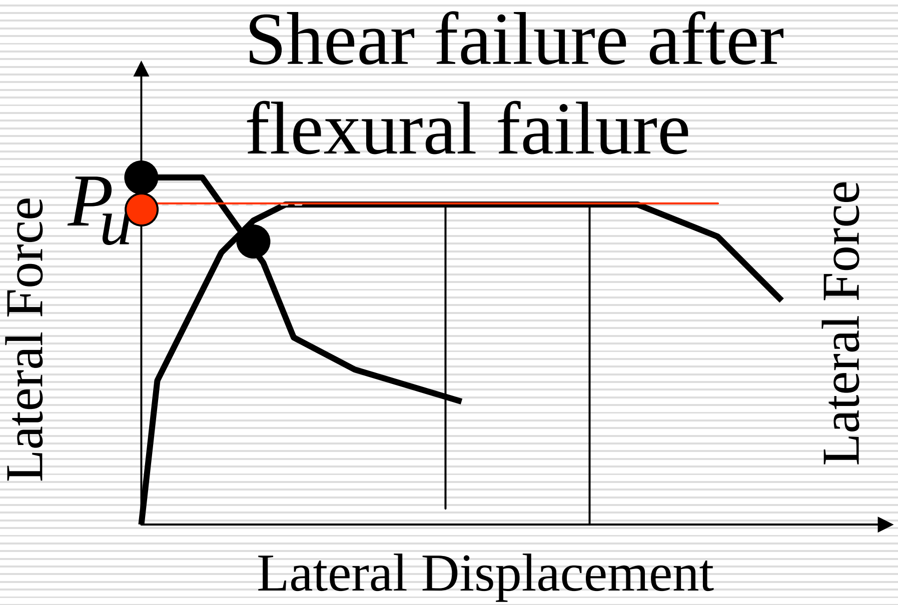
	Type-I GM	Type-II GM
Important bridge	3.0	1.5
Standard Bridges	2.4	1.2

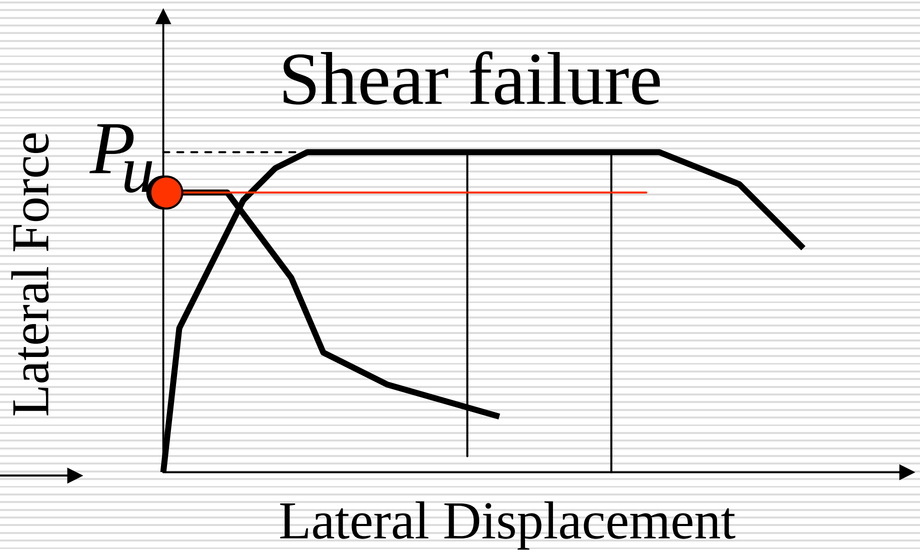




Failure Mode of RC Columns (continued)







Design Strength & Ductility Capacities

$$P_a = \begin{cases} P_u & \cdots\cdots\cdots \text{ flexural failure + shear failure after flexural damage} \\ P_{s0} & \cdots\cdots\cdots \text{shear failure} \end{cases}$$

$$\mu_a = \begin{cases} 1 + \dfrac{d_u - d_y}{\alpha \cdot d_y} \cdots\cdots\cdots \text{flexural failure} \\ 1 \cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots \text{shear failure after flexural damage + shear failure} \end{cases}$$

(6.26)

Sizing of Structures

$$\gamma \cdot D_{ave} \leq \phi \cdot C_{ave}$$

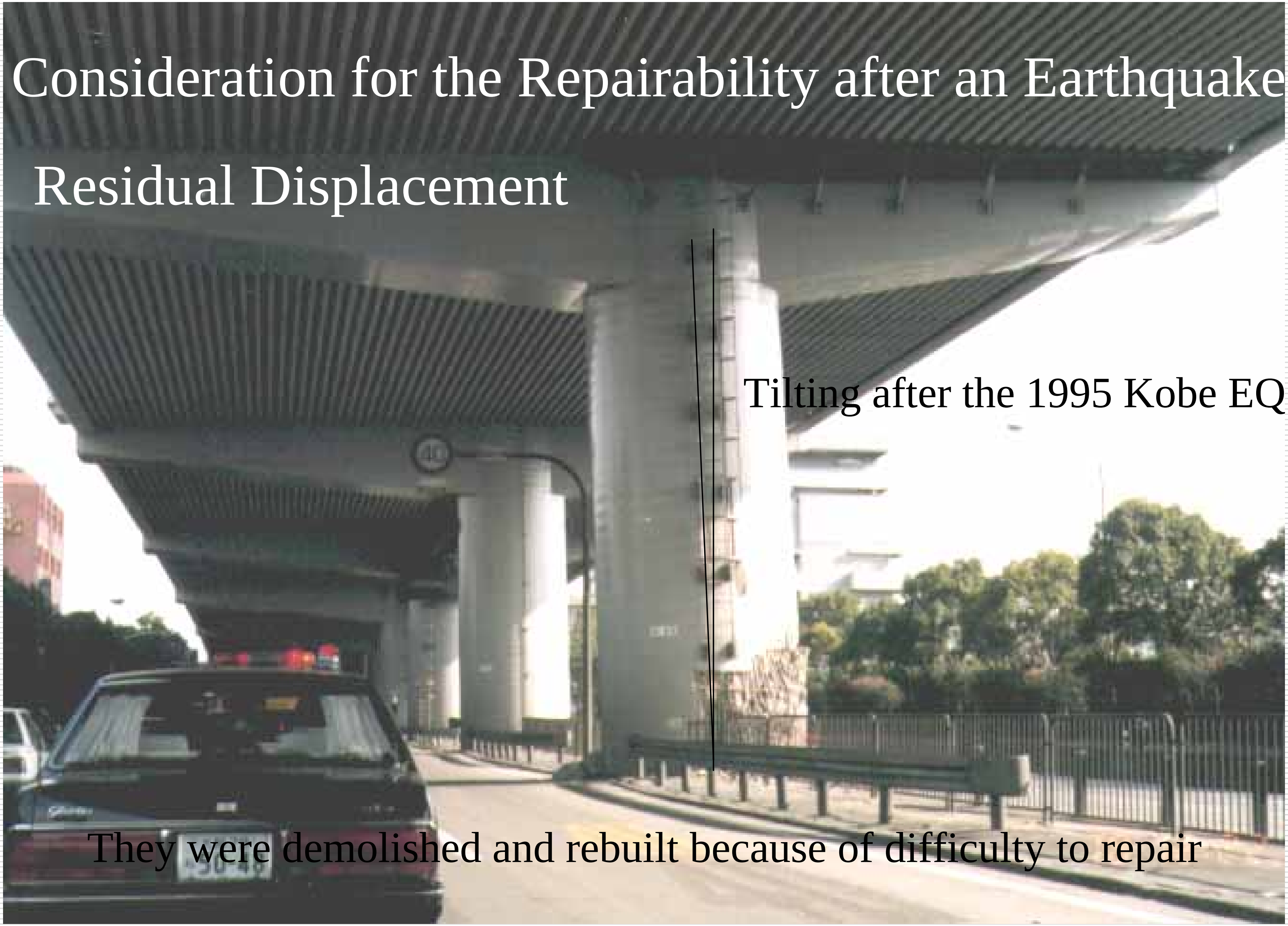
Where,

$$D_{ave} = \frac{m \cdot S_A(T^*, \xi^*)}{R}$$

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

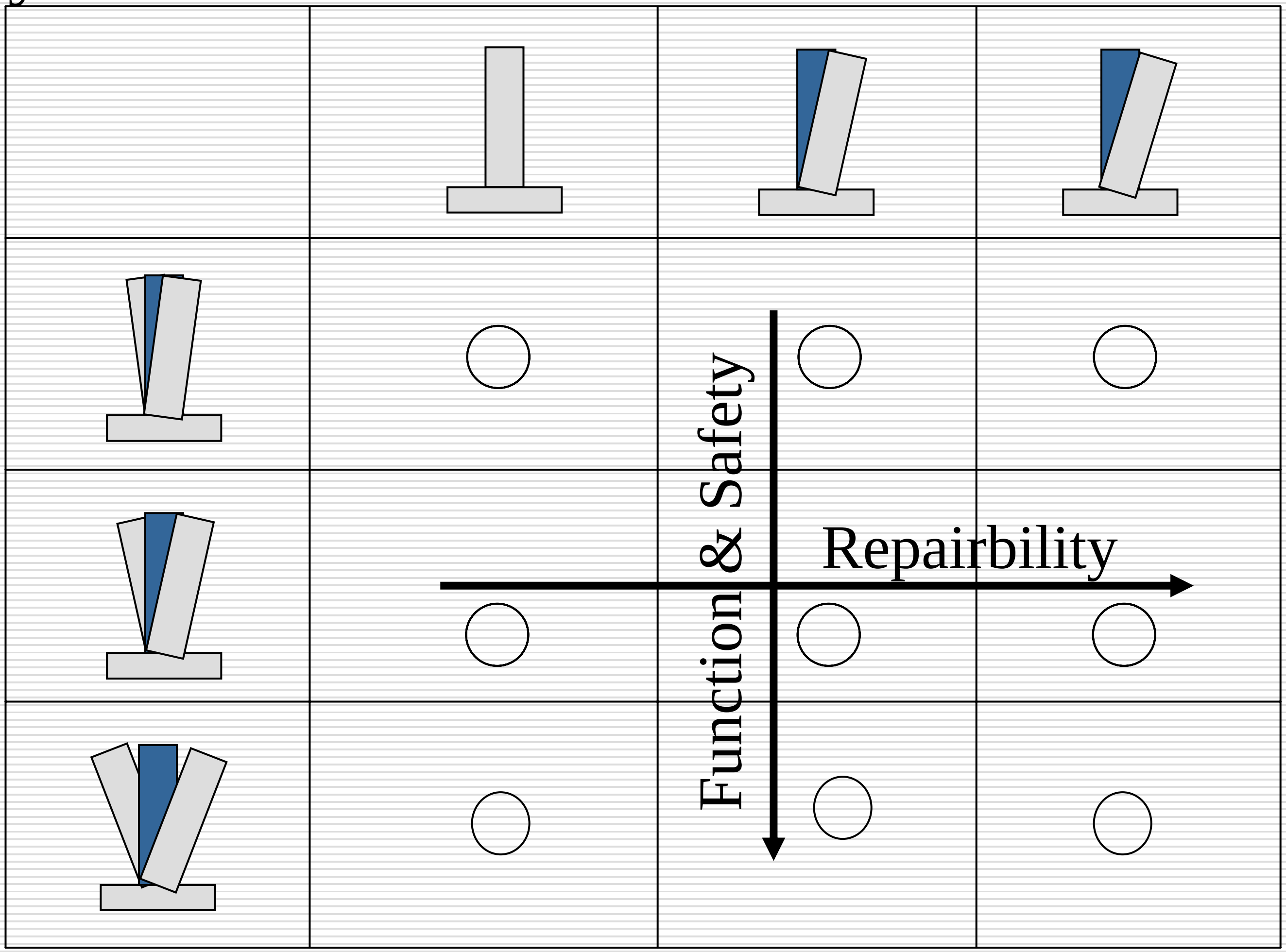
Consideration for the Repairability after an Earthquake

Residual Displacement

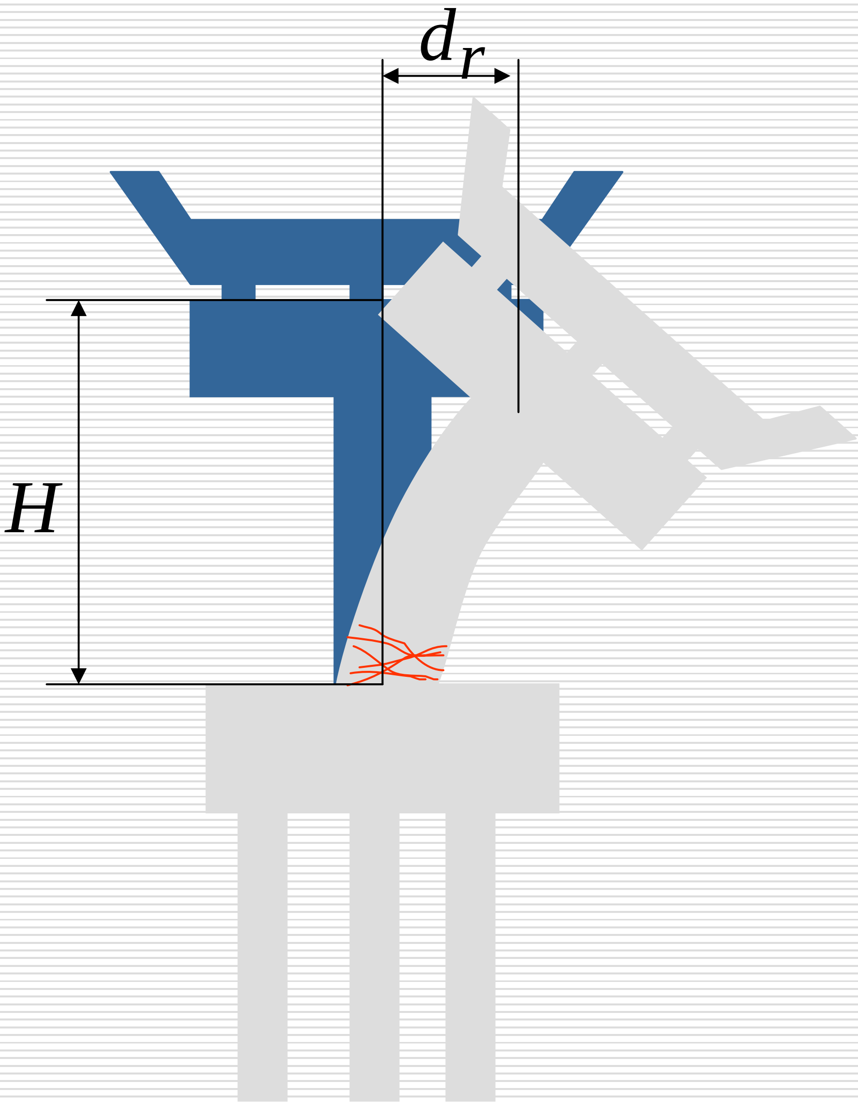


Requirement for the Check of Residual Displacement of Columns

A wide range of residual displacement occurs for the same ductility demand



Requirement for the Check of Residual Displacement of Columns



$$d_R \leq d_{Ra}$$

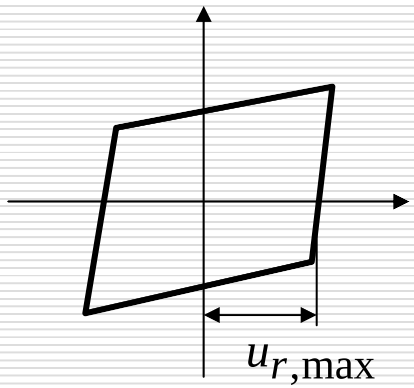
Design (allowable) residual displacement

$$d_{Ra} = \frac{1}{100} H$$

Determination of residual displacement

Residual displacement ratio response spectra

$$S_{RDR} = \left| \frac{u_r}{u_{r,\max}} \right| \qquad r = \frac{k_2}{k_1}$$



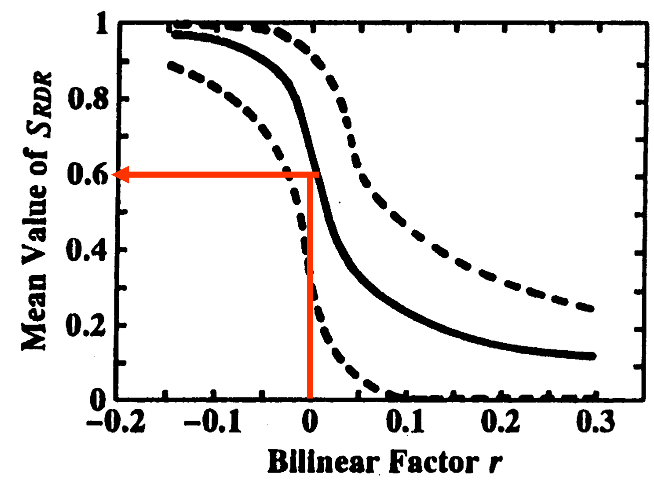
Maximum possible residual displacement

$$u_{r,\max} = (\mu - 1)(1 - r)u_y$$

$$\therefore \quad u_r = S_{RDR}(\mu - 1)(1 - r)u_y$$

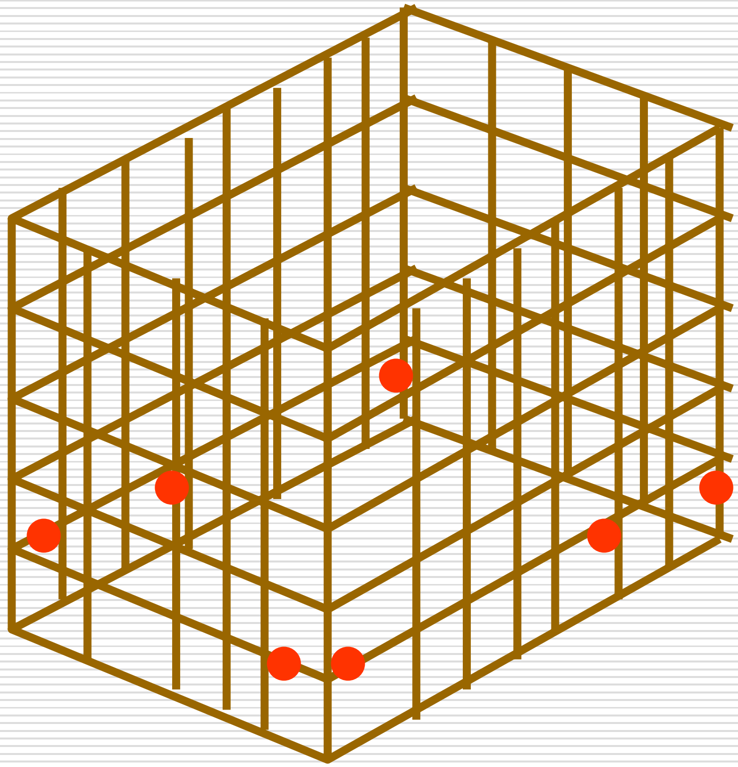
For reinforced concrete columns

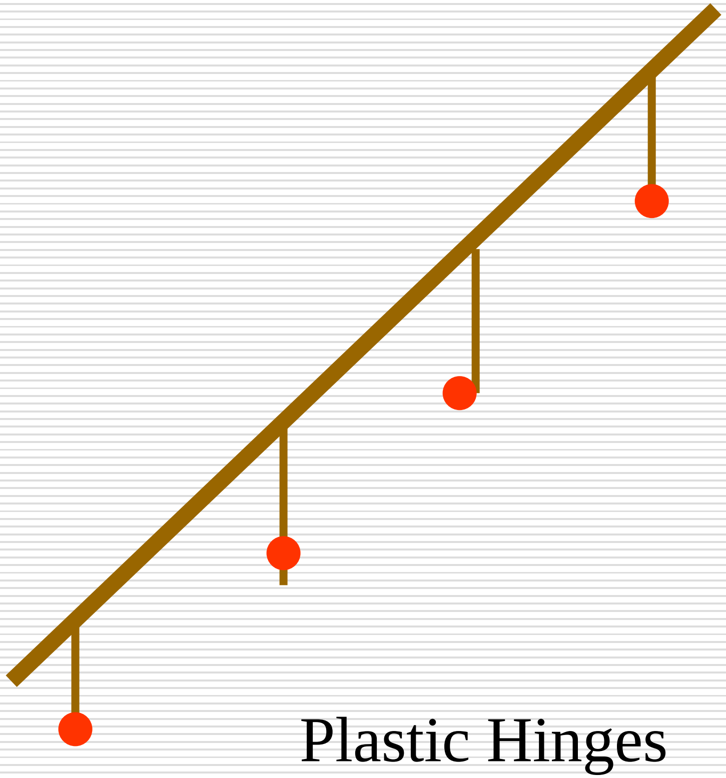
$$S_{RDR} = 0.6 \qquad r = 0$$



Residual Displacement

Significance of Bridges with Fewer Static Indeterminacy than Buildings





Features of Japanese Practice in the Evaluation of Design Ductility Capacity

- Compute design ductility factor for every columns using the fiber element analysis. In the fiber element analysis, stress vs. strain relations of confined concrete and bars are explicitly considered.

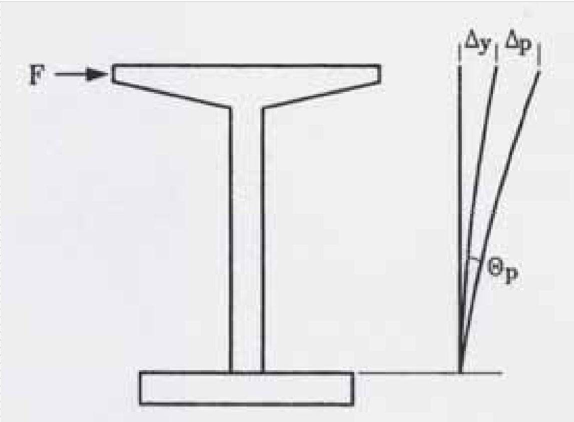
- Characteristics of ground motions (loading hysteresis & number of repeated loadings) is accounted in the evaluation of the ultimate displacement, therefore the ultimate displacement depends on the ground motions (Type I & II ground motions).

- Importance of bridges is accounted for not in the evaluation of design ground motions but in the evaluation of design ductility factor
- To account for repairability, residual displacement that would occur after an earthquake is checked

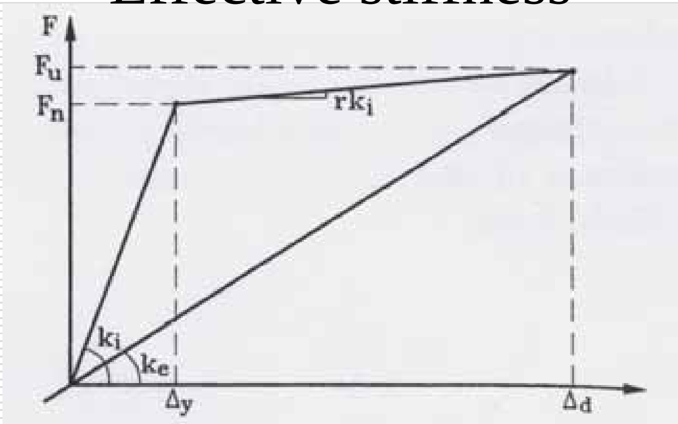
Displacement based Seismic Design

Displacement-based Seismic Design

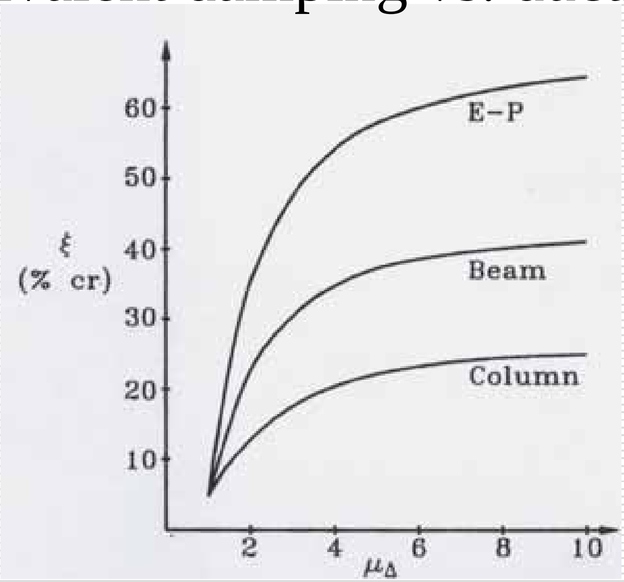
Column Displacement



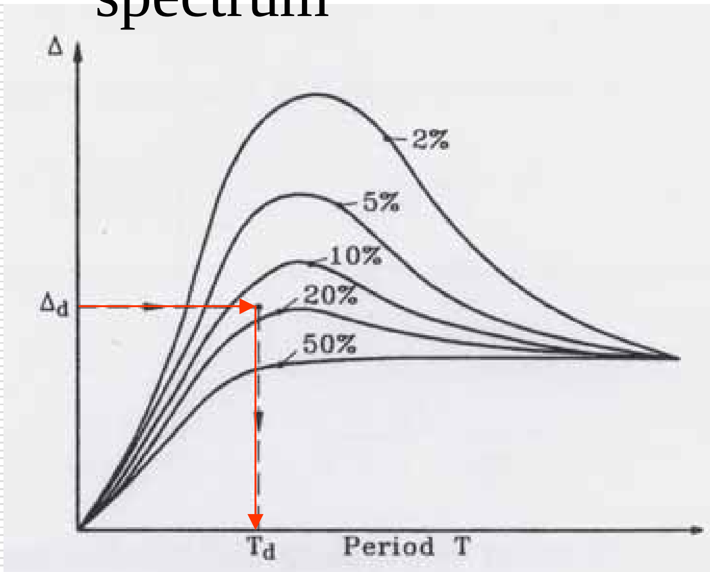
Effective stiffness



Equivalent damping vs. ductility



Design displacement response spectrum



Priestley et al, Seismic Design & Retrofit of Bridges, John Wiley

Displacement-based Seismic Design (continued)

It is interesting because

- Displacement which is important in design is always accounted in the displacement design

However, the following points need further clarification

- Damping ratio which has a large scattering is directly included in the process of main design calculation
- Determination of the stiffness of a bridge from a design displacement involves a large error because

$$k \propto T^2$$