Seismic Design of Urban Infrastructures
都市施設の耐震設計

(4-1) Engineering Characteristics of Ground Motion and Design Ground Motions
(4-1) 地震動の工学的性質と設計地震動

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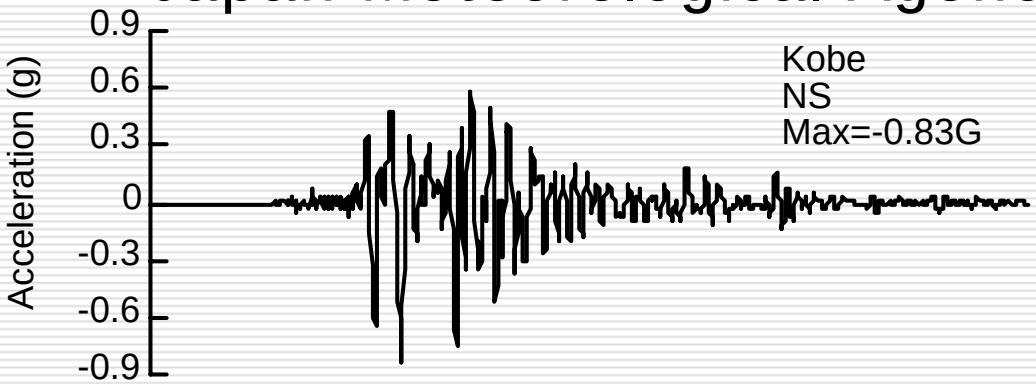
4.1 Ground Motion　地震動の特性

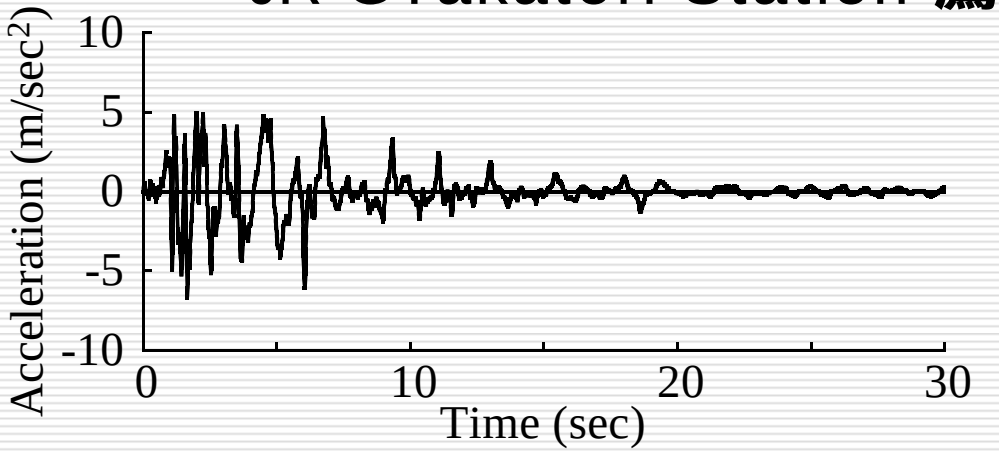
1) Ground Motion Characteristics depend on Locations 同じ地震でも、地震動は場所によって異なる

1995 Kobe, Japan earthquake 1995年兵庫県南部地震

Japan Meteorological Agency 神戸海洋気象



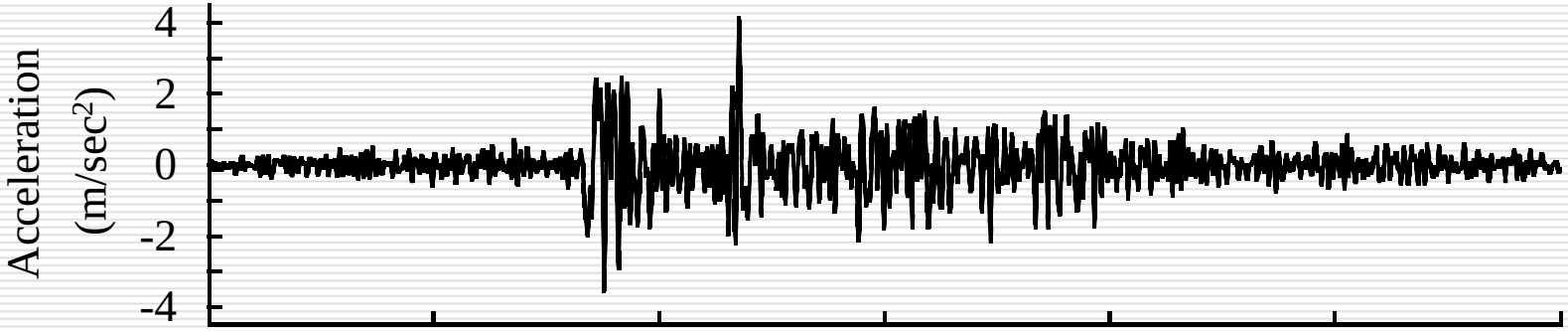
JR GTakatori Station 鷹取駅



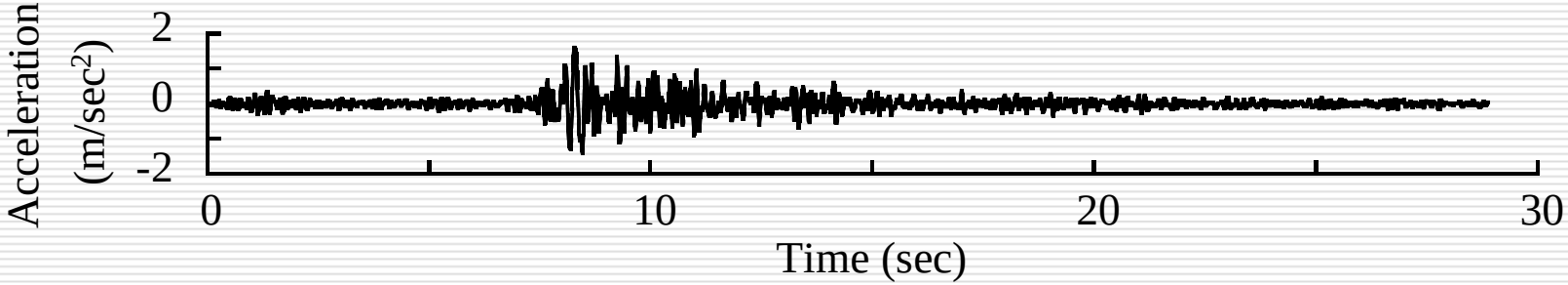
2) Ground motions at the same location depends on earthquakes 同じ点で観測される地震動は、地震が異なれば同じではない

Kaihoku bridge, Ishinomaki, 開北橋周辺地盤

1978 Miyagi-ken-oki earthquake 宮城県沖地震 (M7.4)



An earthquake in February 1978 (M6.7)宮城県沖の地震



3) Characteristics of ground motions 地震動の特性

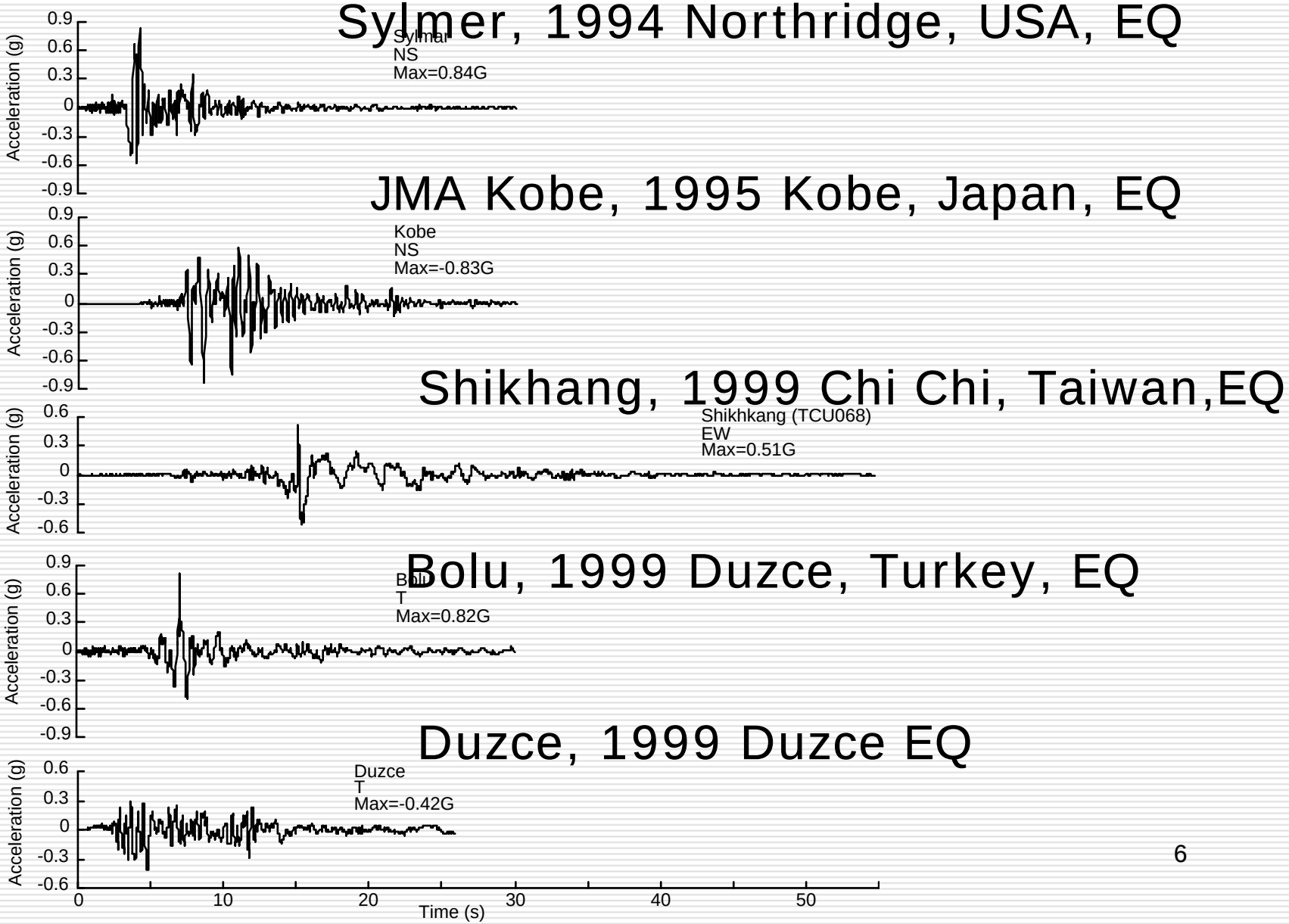
- Intensity, frequency components and duration of ground motions depends the fault mechanism, size of the fault and distance from the fault zone. Recently estimate of ground motions using fault rupture model becomes available, however accuracy of the estimate is still insufficient for seismic design.

地震動は地震の規模や断層からの距離によって複雑に変化する。近年、断層モデルを用いた解析が行われるようになってきたが、精度にはまだ大きな問題があり、これを解析的に再現することは大変困難である。

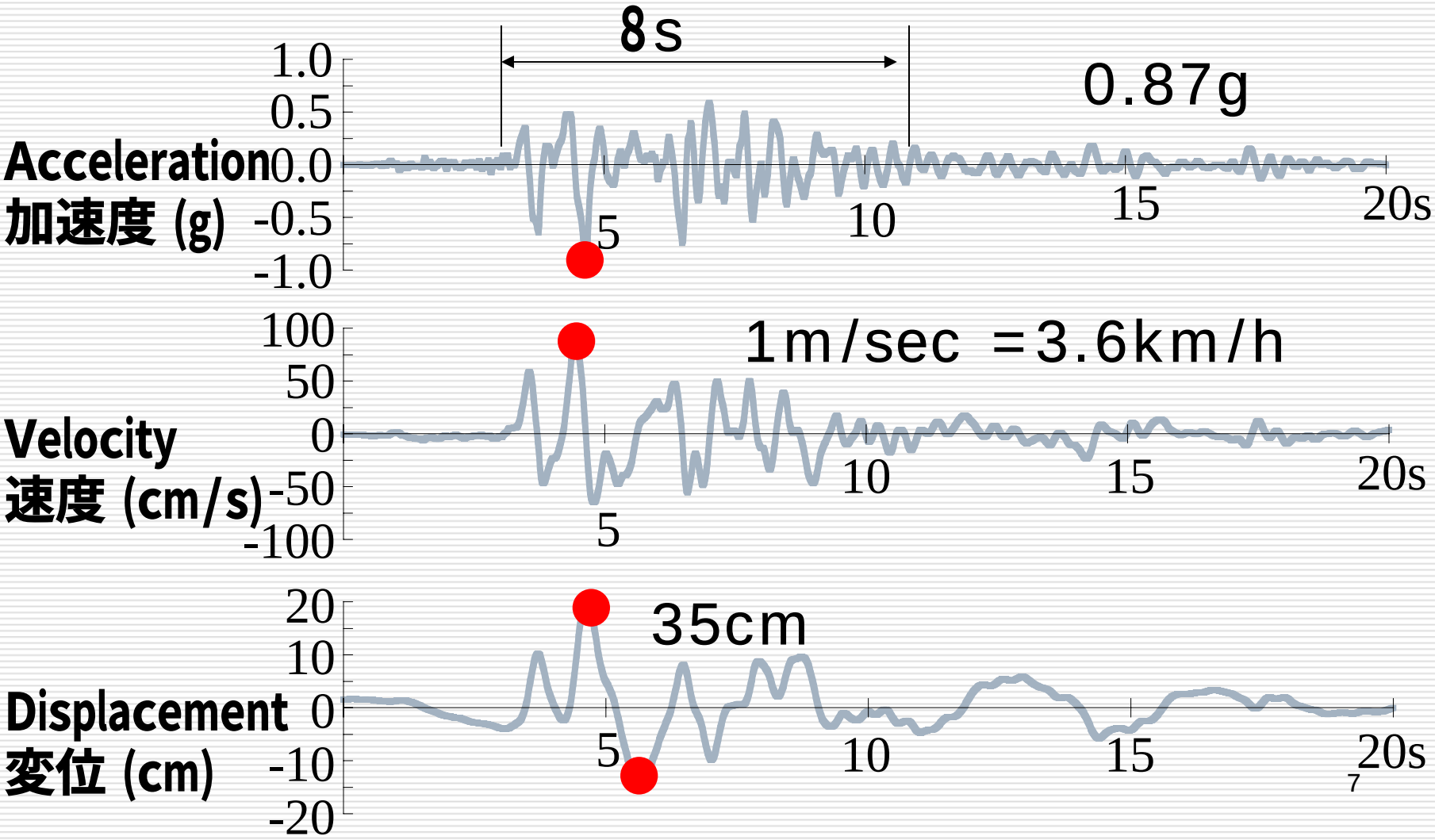
- It is recommended to use recorded ground motions for evaluating seismic response analysis of structures. **現在の耐震設計では、実測された強震記録に基づいて設計地震力が定められている**

● Since 1933 when the first strong ground motion was recorded, it is only ¾ century. Thus, our knowledge on the characteristics of ground motions is still insufficient.
強震記録が得られるようになってからまだ3／4世紀しか経っておらず、強震動の特性がまだ十分解明されていない。世界最初の強震記録は1933年 Long Beach earthquake**によって得られた。**
● Recently number of strong motion records which are important for seismic design is sharply increasing.
最近になり、強震計の数が飛躍的に増加し、徐々に、断層近傍での地震動が得られるようになってきた

4) Typical near-field ground motions 断層近傍地震動の例



4) JMA Kobe ground acceleration: A typical near-field ground motion during 1995 Kobe, Japan earthquake



5) Important Characteristics of Near-Field Ground Accelerations 断層近傍地震動の特徴

- Long pulse acceleration (often called “killer pulse”) **断層近傍地震動は周期の長いパルス地震動を含んでいる。**

  ✓Pulse accelerations contribute to reduce the force reduction factor **パルス地震動は荷重低減係数を小さくする可能性が高い**

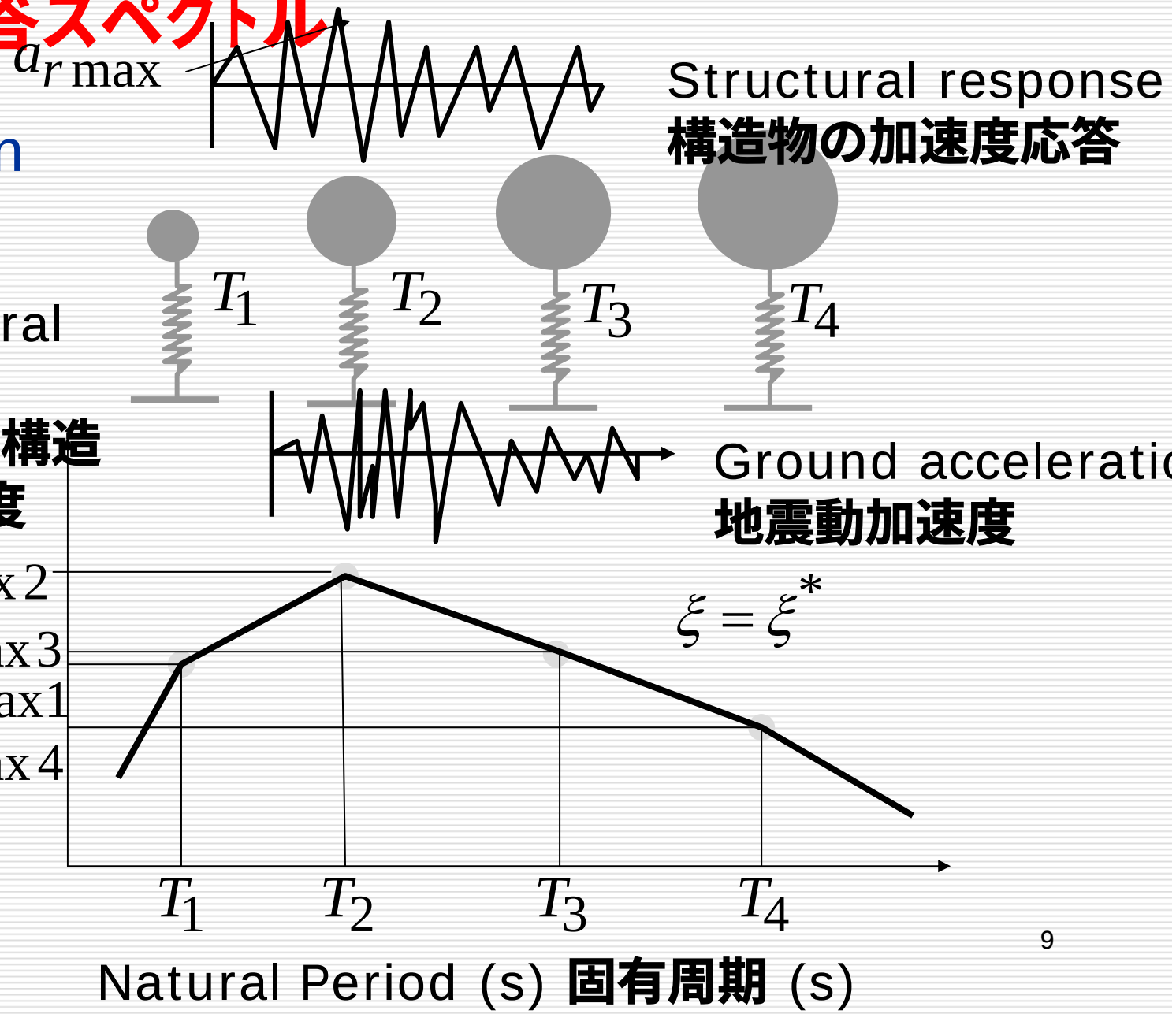
  ✓Pulse accelerations can increase residual displacement (permanent displacement) after an earthquake **パルス地震動は残留変位を大きくする可能性が高い**

- Intensity of ground motions can be amplified due to directivity **断層近傍地震動はディレクティビティー（方向によって、地震動強度が異なること）がある**

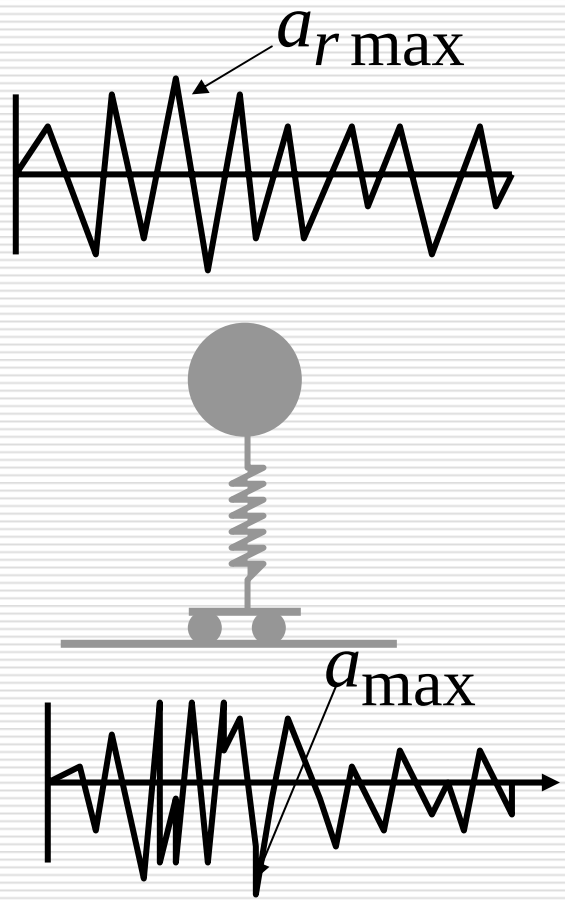
4.2 Acceleration Response Spectrum
加速度応答スペクトル

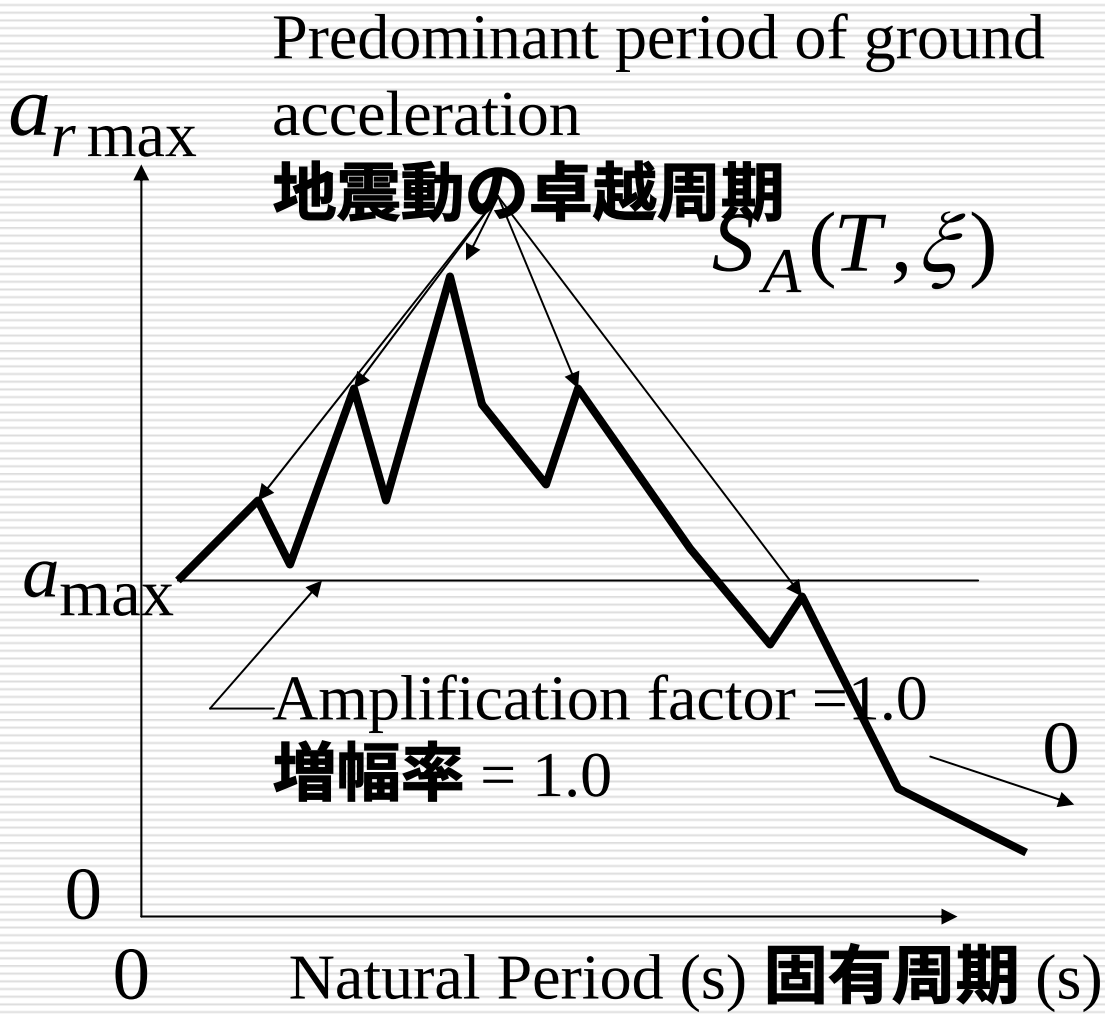
1) Definition
定義

Peak Structural Response Acceleration **構造物の最大加速度**

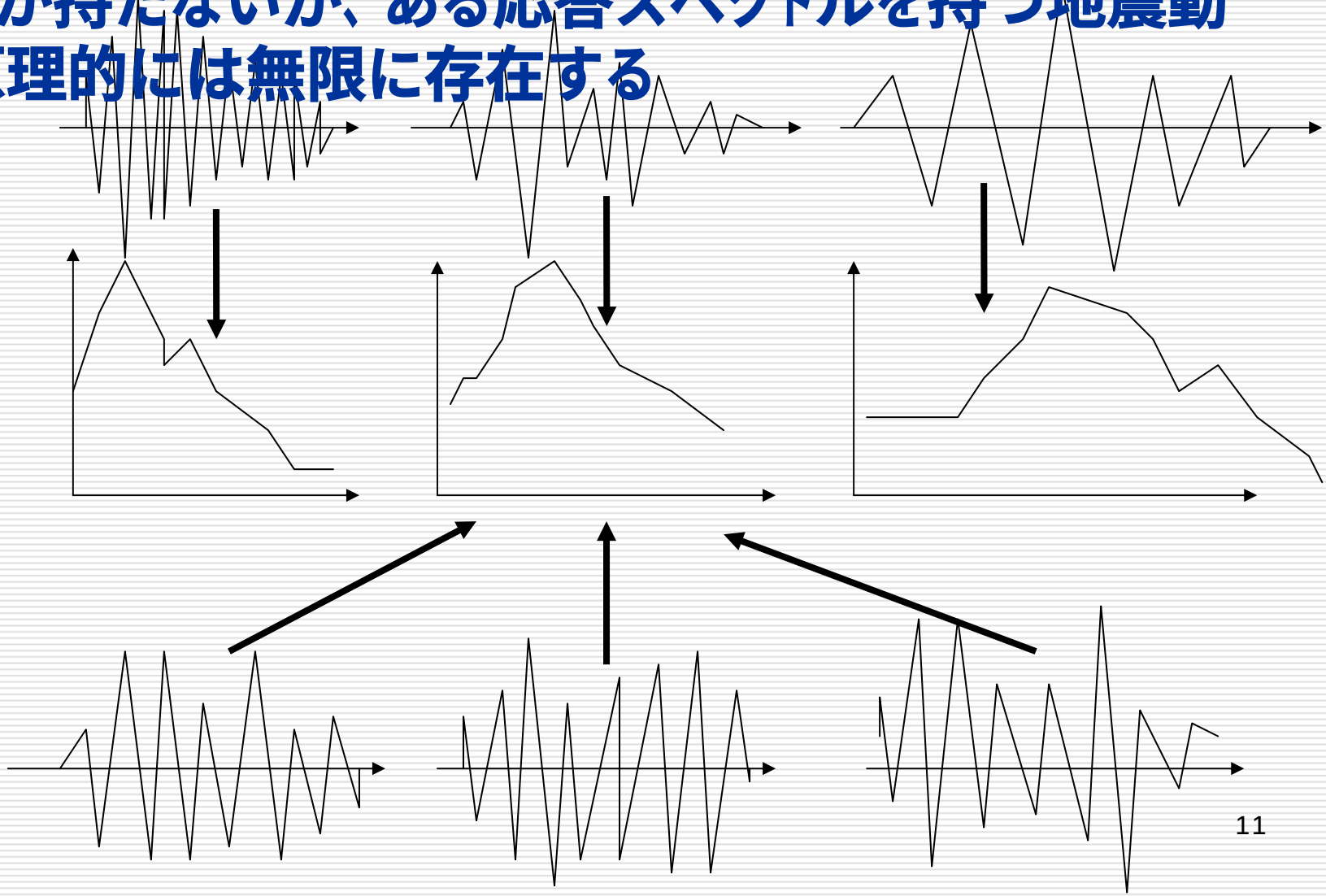


2) Nature of Acceleration Response spectrum
加速度応答スペクトルの性質

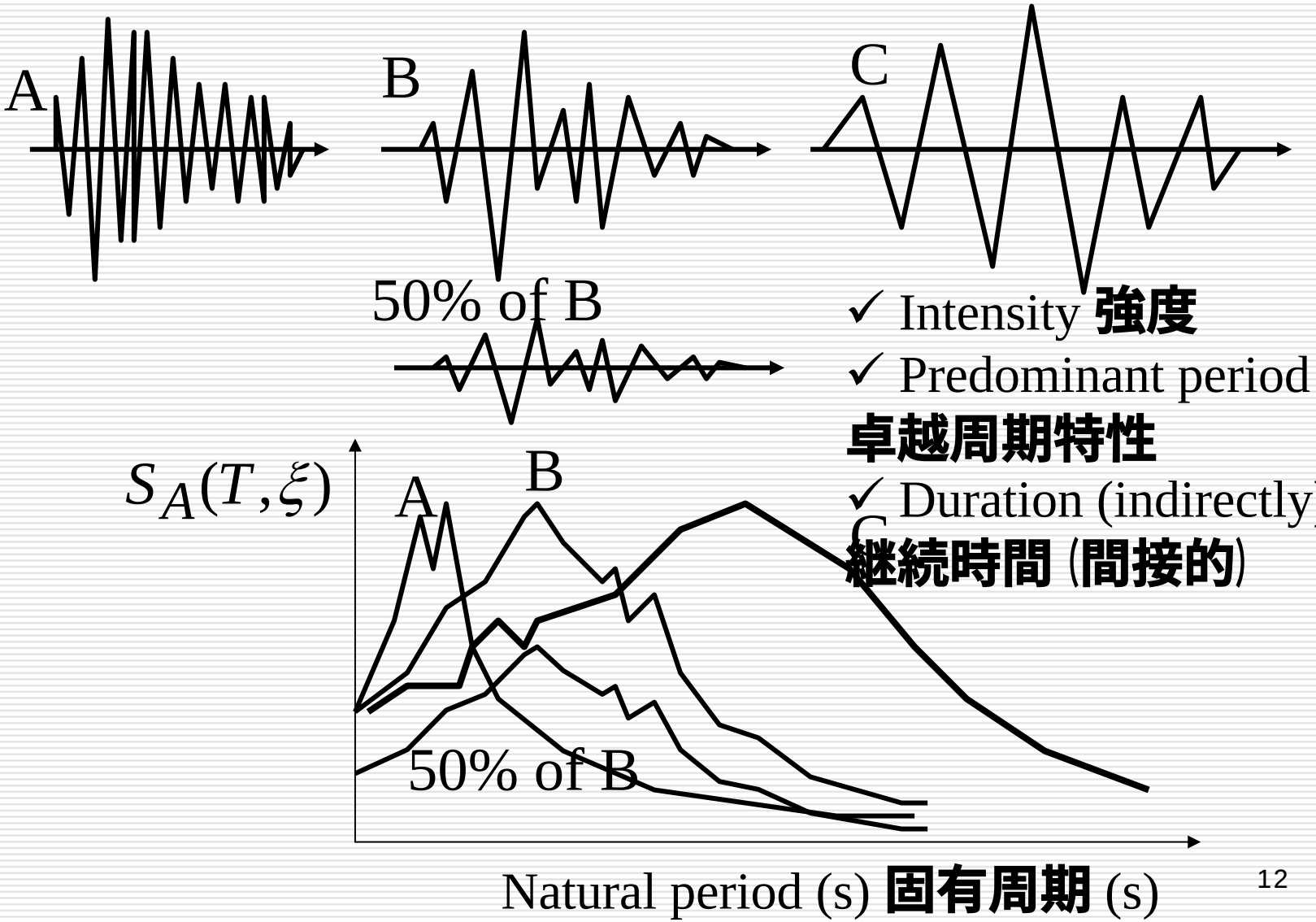




3) One GM has only one response spectrum, however theoretically there are number of GMs which have this response spectrum. **1つの地震動は1つの応答スペクトルしか持たないが、ある応答スペクトルを持つ地震動は原理的には無限に存在する**

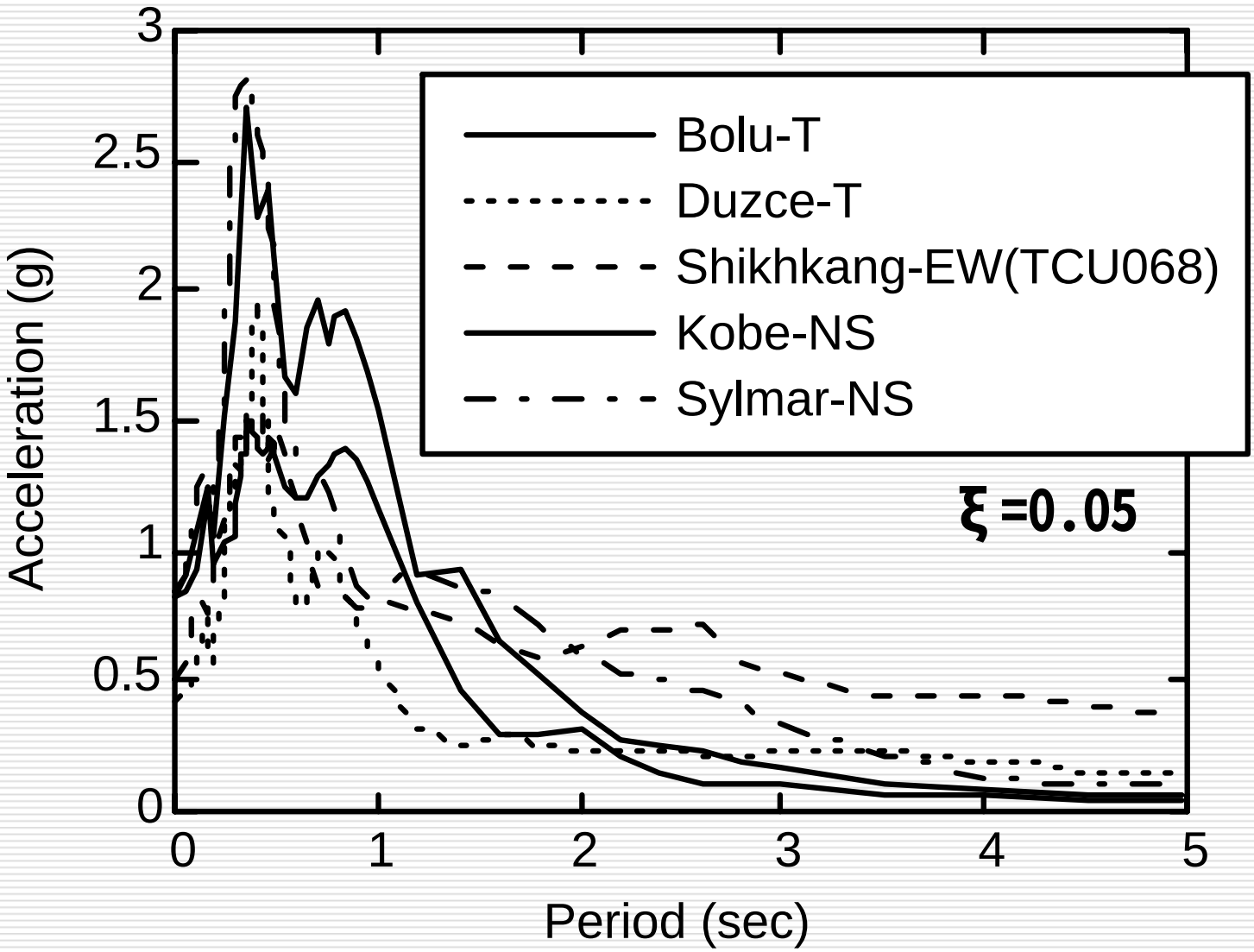


4) What can we know from a response spectrum?
地震応答スペクトルから地震動の何がわかるか？

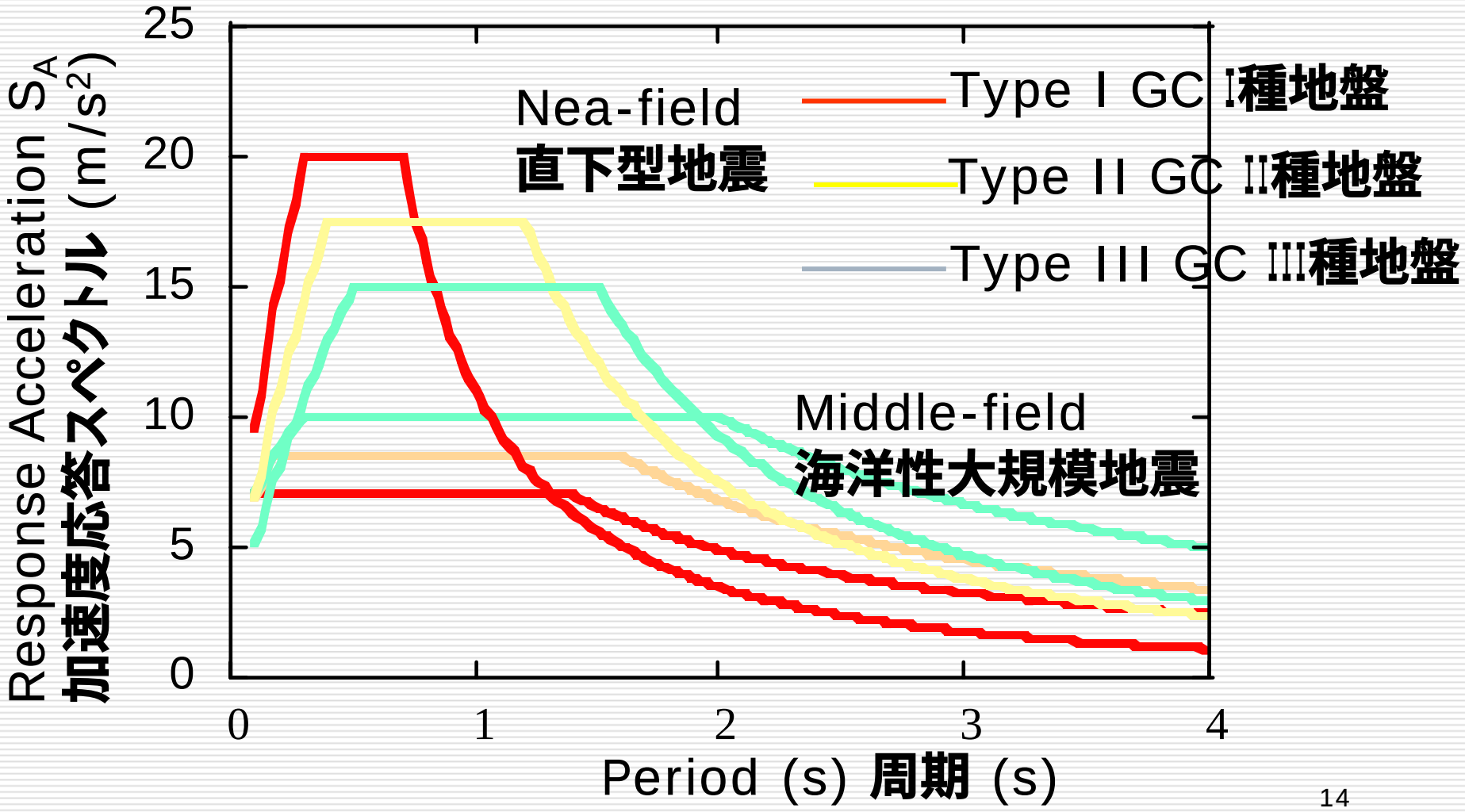


- ✓ Intensity 強度
- ✓ Predominant period 卓越周期特性
- ✓ Duration (indirectly) 継続時間 (間接的)

5) Response spectra of typical near-field GMS
典型的な強震記録の加速度応答スペクトル



6) Design GM represented in terms of response spectra
設計地震力に用いられる応答スペクトル

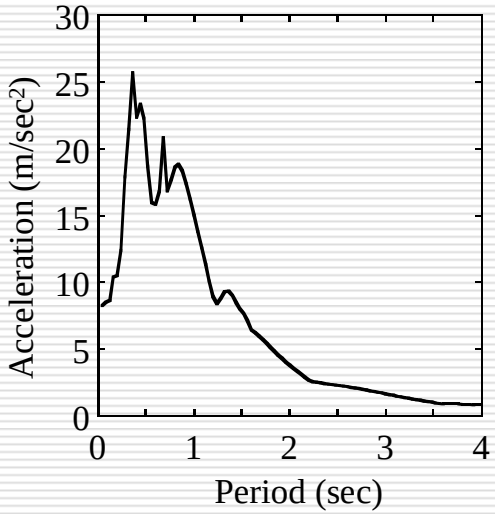


7) Velocity & displacement Response Spectrum
速度および変位応答スペクトル

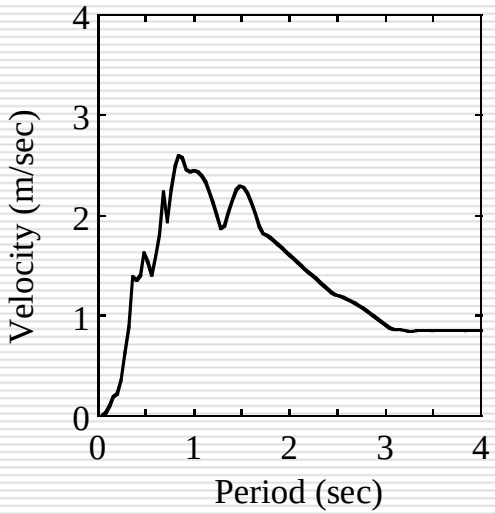
1995 JMA Kobe Observatory

$$\xi = 0.05$$

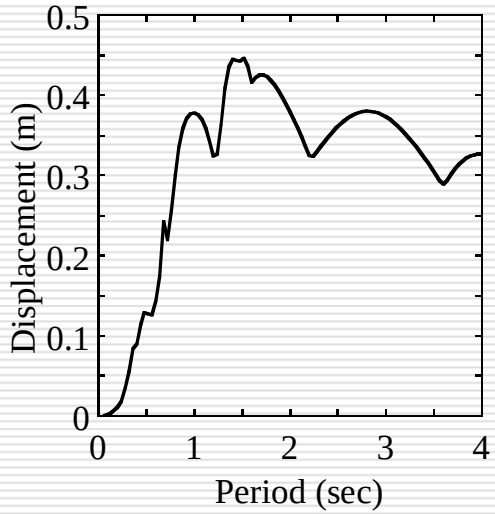
Acceleration



Velocity



Displacement

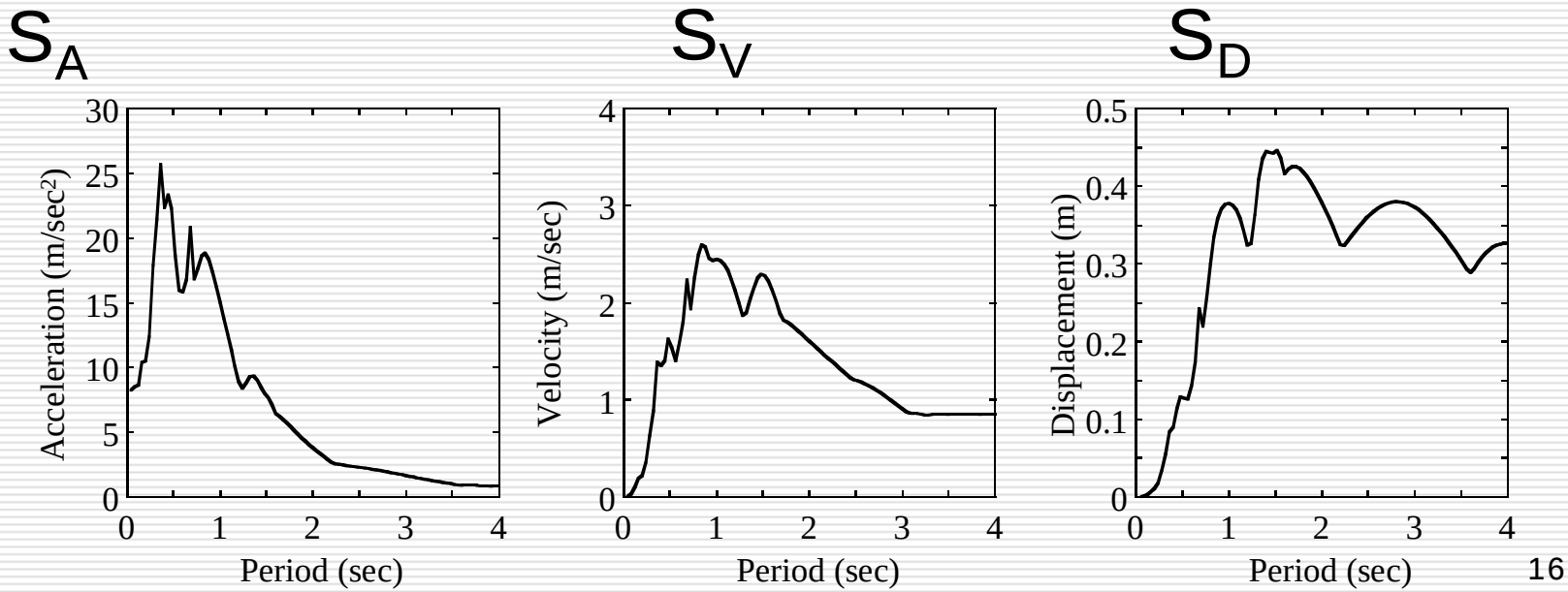


Velocity & displacement response spectrum (2)
速度および変位応答スペクトル (2)

$$S_A(T,\xi) \approx \left(\frac{2\pi}{T}\right) \cdot S_V(T,\xi)$$

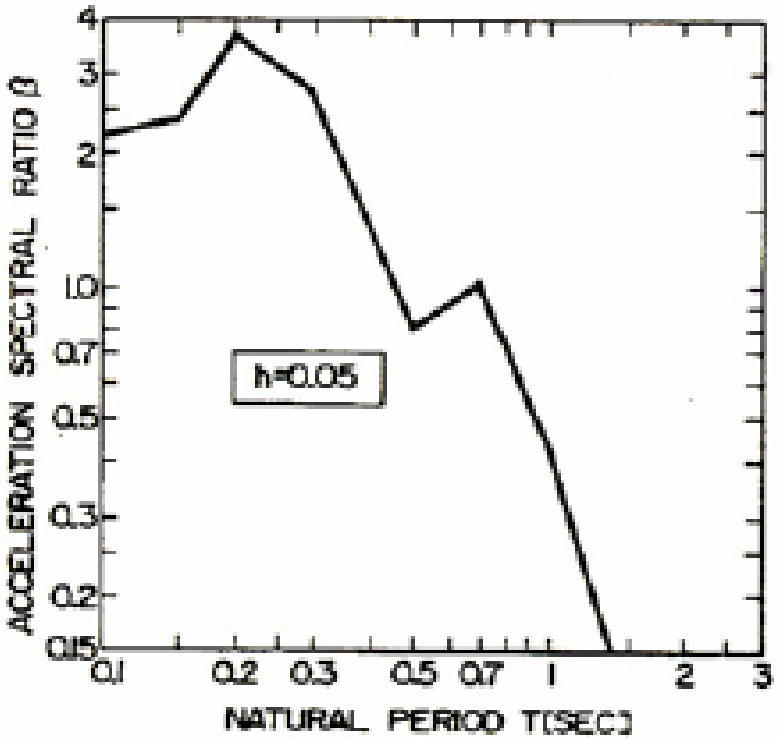
$$S_D(T,\xi) \approx \left(\frac{T}{2\pi}\right) \cdot S_V(T,\xi)$$

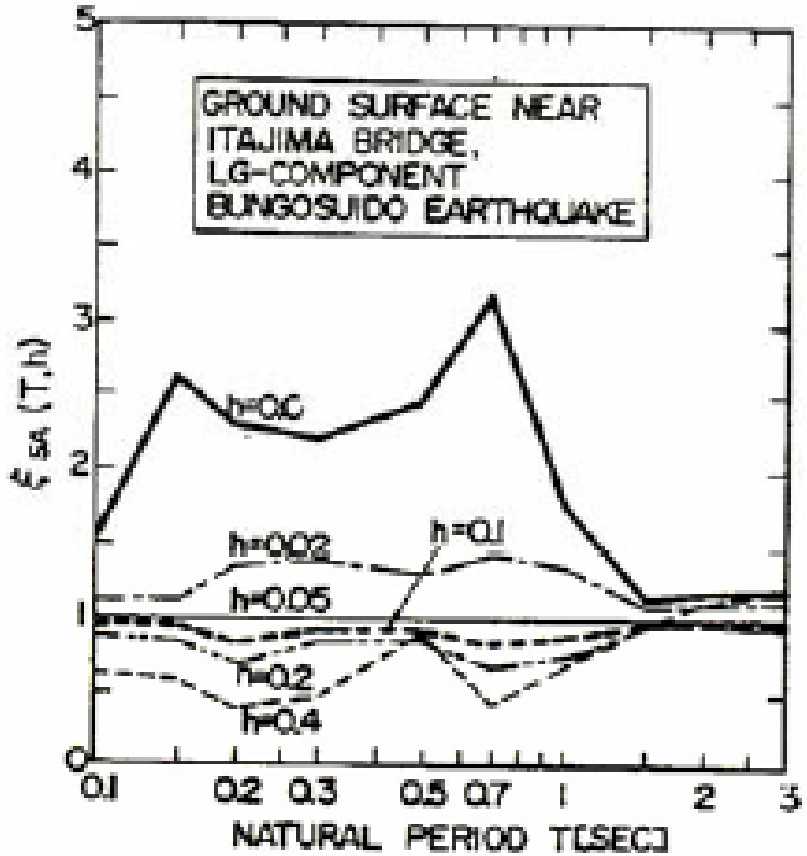
$$\xi = 0.05$$



(8) How does S_A change in accordance with damping ratio? 減衰定数が異なると、加速度応答スペクトルはどのように変化するか？

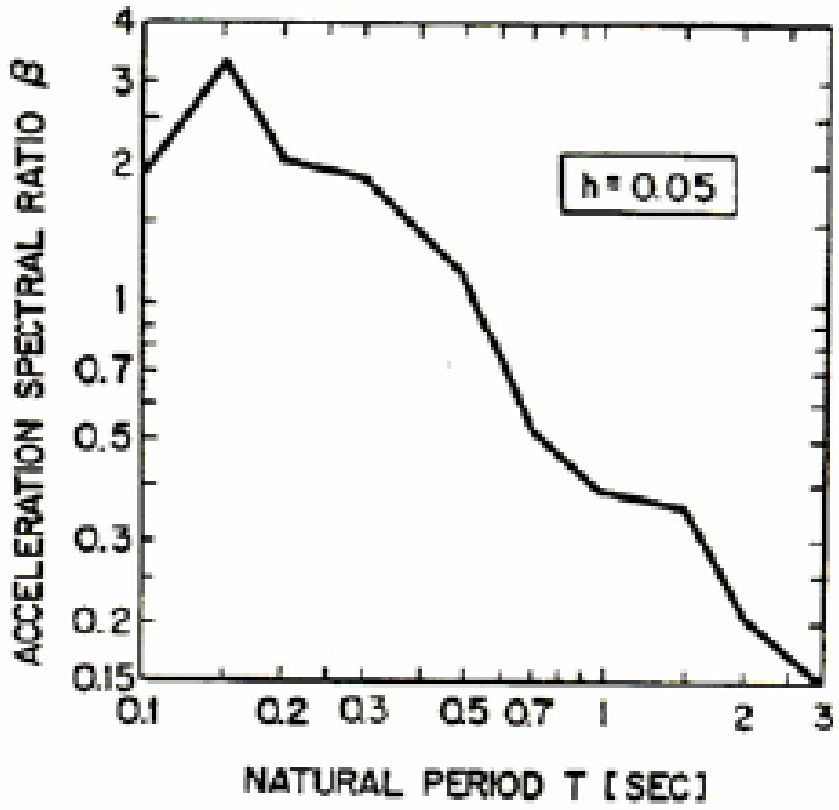
A ground motion recorded at Itajima Bridge site

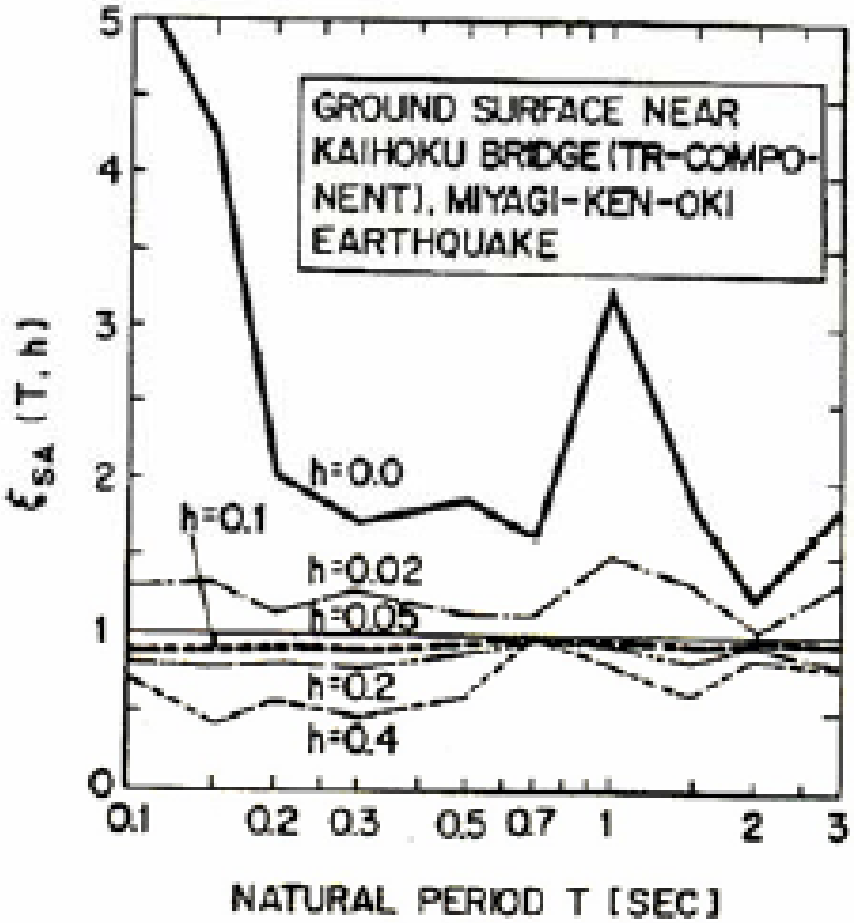




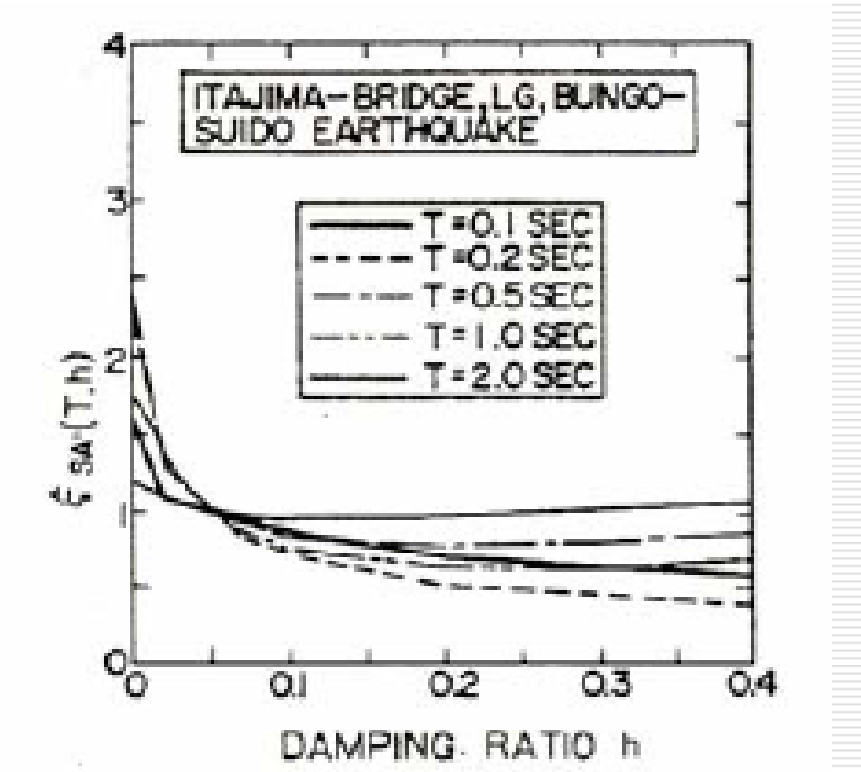
How does response acceleration change depending on damping ratio?

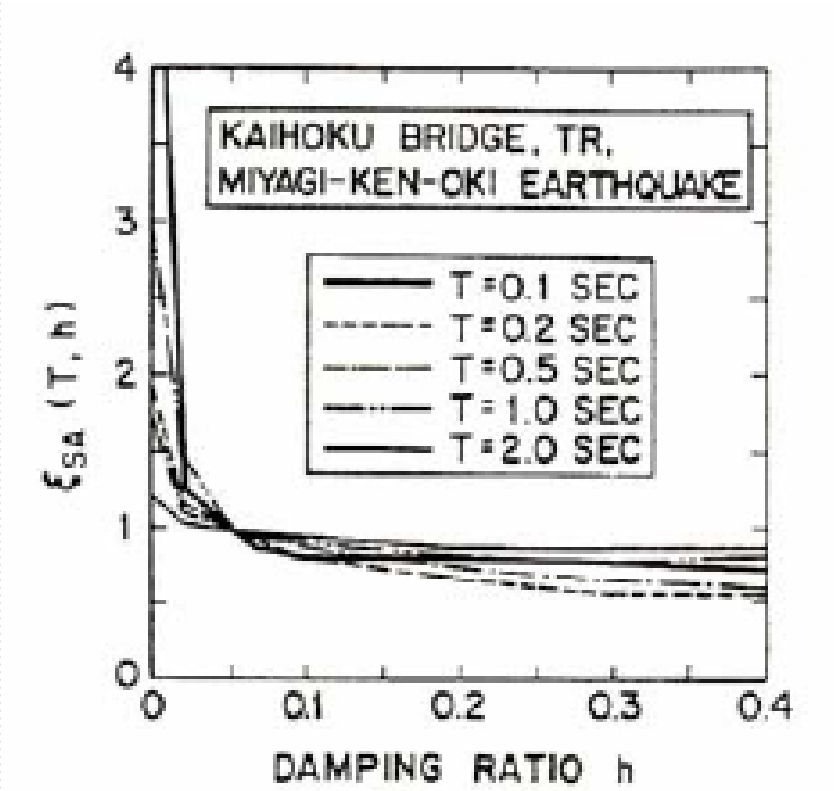
A ground motion recorded at Kaihoku Bridge site



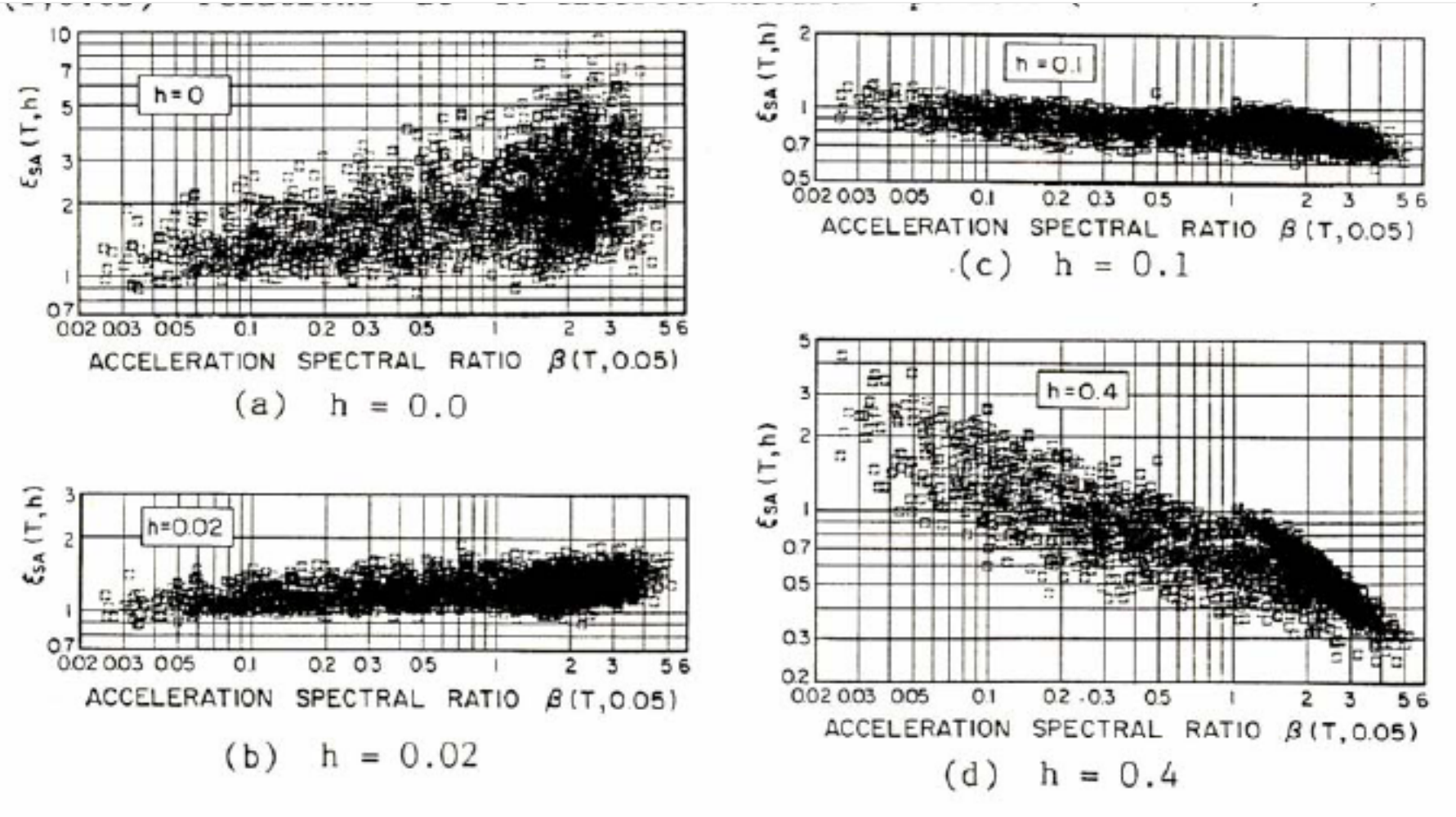


How does $\xi_{SA}(T,h)$ vary in accordance with damping ratio?



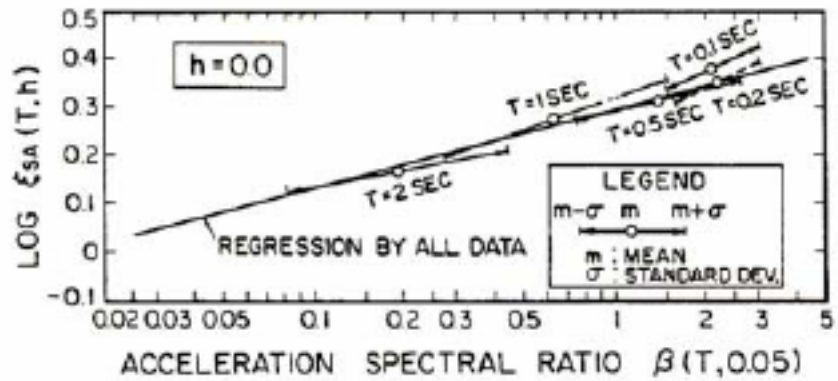


How does $\xi_{SA}(T,h)$ vary in accordance with response spectrum shape?

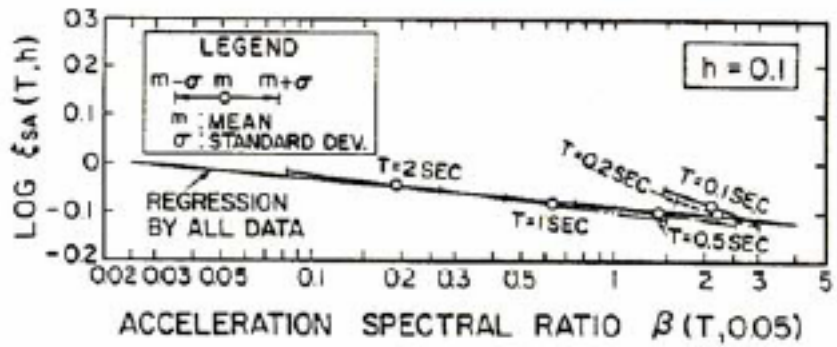


How does $\xi_{SA}(T,h)$ vary in accordance with response spectrum shape?

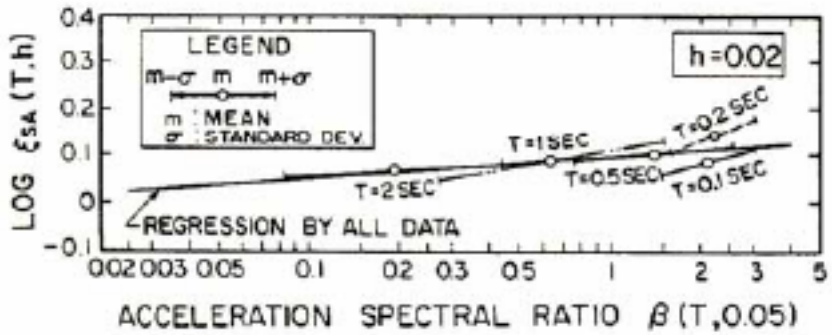
$$\log \xi_{SA}(T,h) = \log a(h) + b(h) \log \beta(h, 0.05)$$



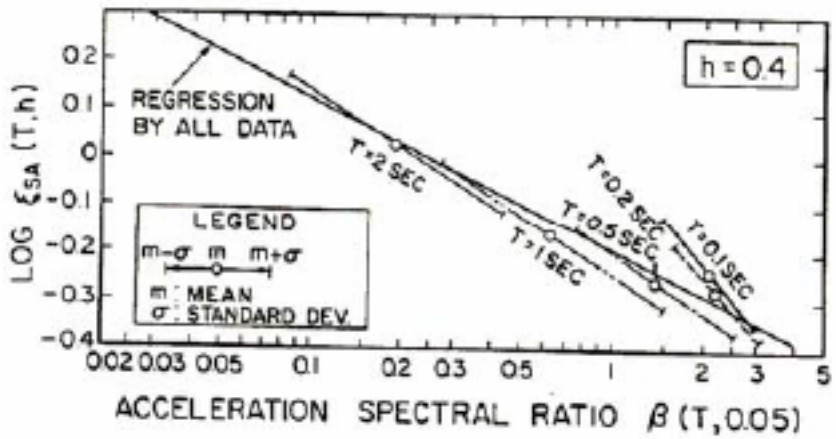
(a) h = 0.0



(c) h = 0.1

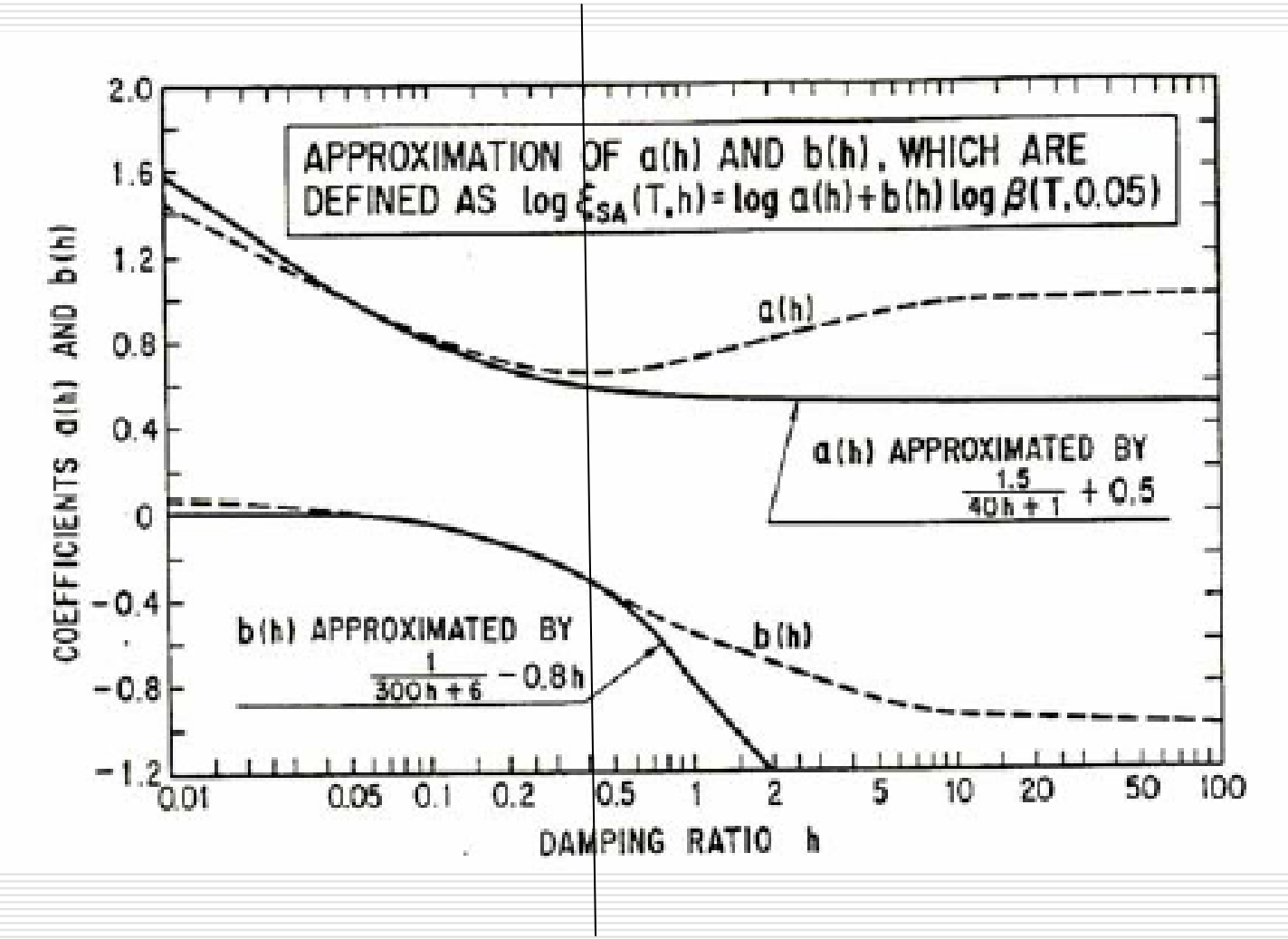


(b) h = 0.02



(d) h = 0.4

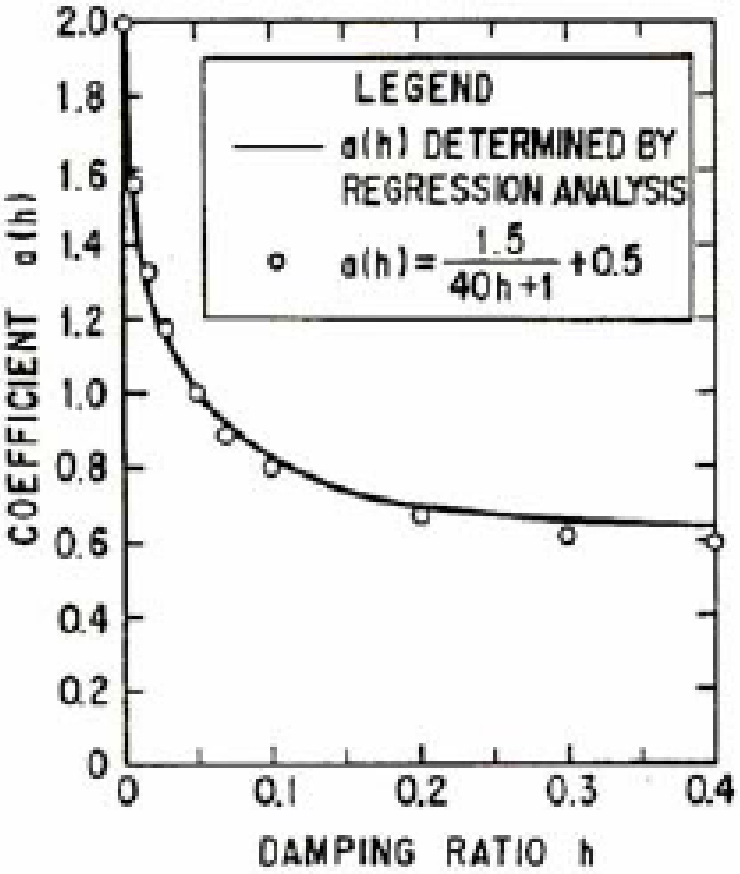
Damping Ratio Dependence of a(h) and b(h)



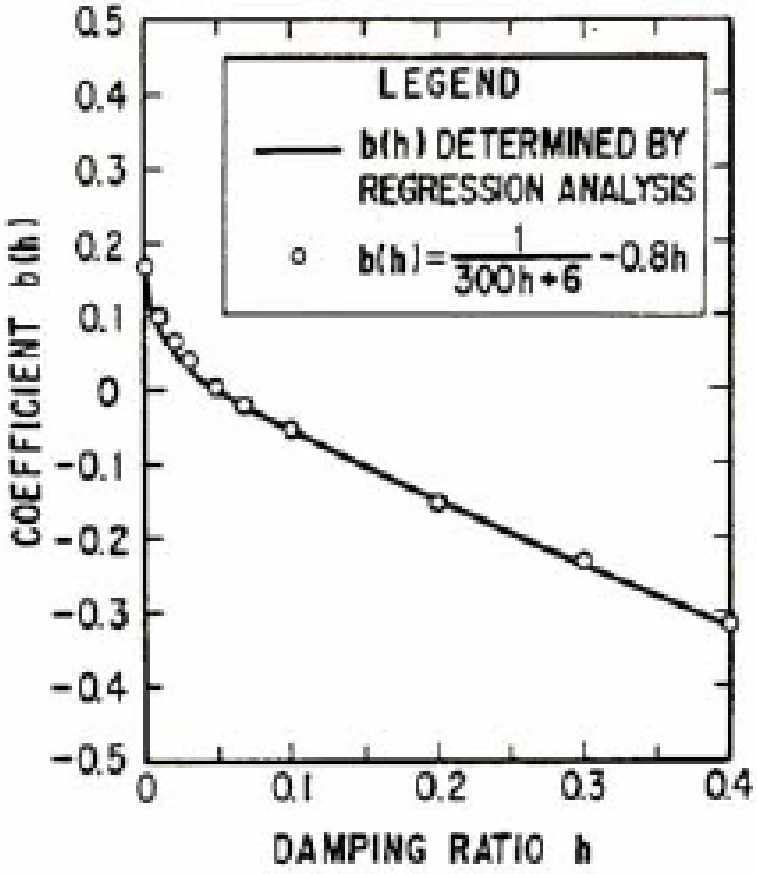
If we are interested in structural response under damping ratio less than 0.4, a(h) and b(h) may be approximated as

$$\log \xi_{SA}(T,h) = \log a(h) + b(h) \log \beta(h, 0.05)$$

$a(h)$



$b(h)$



Least square fitting for a(h) and b(h)

$$a(h) = \frac{1.5}{40h+1} + 0.5$$

$$b(h) = \frac{1}{300h+6} - 0.8h$$

Therefore, $\xi_{SA}(T,h)$may be evaluated as,

$$\xi_{SA}(T,h) = a(h) \times \beta(T,0.05)^{\left(\frac{1}{300h+6} - 0.8h\right)}$$

川島、相沢：地震応答スペクトルに及ぼす減衰定数の影響、土木学会論文集、３３５、25-29、1983

Practical Evaluation of Damping Ratio Dependence of Acceleration Response Spectra $S_A(T,h)$

$$\xi_{SA}(T,h) = \frac{S_A(T,h)}{S_A(T,0.05)}$$

$$= a(h) \times \beta(T,0.05)^{\left(\frac{1}{300h+6} - 0.8h\right)}$$

$$\approx \frac{1.5}{40h+1} + 0.5$$

This equation is widely used to estimate the response acceleration with an arbitrary damping ratio based on the response acceleration with 5% damping ratio

$$S_A(T,h) \approx \left(\frac{1.5}{40h+1} + 0.5\right) \times S_A(T,0.05)$$

S_A with arbitral damping ration can be approximately estimated by S_A with 0.05 damping ratio as

減衰定数5%の加速度応答スペクトルを基準にすると、その他の減衰定数を有する加速度応答スペクトルは、

$$S_A(T,\xi) = c_D(\xi) \cdot S_A(T,0.05)$$

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

Various other equations are proposed

このほかにもいろいろな式が提案されている。

4．3　Design Seismic Force 設計地震力

1) Basic Issues 基本事項

●Design seismic force depends on seismicity at the construction site 設計地震力は地震活動度（地震発生頻度、最大地震規模、断層の大きさ等）に応じて定められる。

●Probabilistic hazard map in terms of peak ground acceleration is well used for this purpose 最大加速度（ＰＧＡ）の期待値によって表される地震ハザードマップが設計地震力の設定によく用いられる。

●For seismic design, hazard map in terms of response acceleration is more important. 最近は、ＰＧＡよりも加速度応答スペクトルの方が設計地震力を与えるために使用される。これは耐震設計に必要なパラメータはＰＧＡではなく、構造物の応答であるためである。

●Difference of seismicity is generally included in terms of zoning factor **地域的特性を取り入れるため、標準設計スペクトルを地域別補正係数で修正する**

$$S_A(T,\xi)_{design} = c_z \cdot S_A(T,\xi)_{standard}$$

●Importance of structures is taken into account by importance factor. There are two ways:**構造物の重要性を重要度別補正係数で考慮する。この際、2つの考え方がある。**

- ✓Increase seismic design force for important structures **重要な構造物には重要度別補正係数によって設計地震力を大きくする。**
- ✓Consider the importance in the evaluation of capacity of structural members **構造部材のキャパシティーを評価する際に重要性を考慮する**

2) Seismic performance goals in road bridges in Japan
耐震性能目標の例 (道路橋、1996-)

<table>
<tr><th colspan="2">設計地震動</th><th>A種</th><th>B種</th></tr>
<tr><td colspan="2">Level 1 GMs (Function evaluation GMs) レベル1地震動</td><td colspan="2">Functional (SPG 1) 橋としての健全性を損なわない性能 (耐震性能1)</td></tr>
<tr><td rowspan="2">Level 2 GMs (Safety evaluation GMs) レベル2地震動</td><td>Type I GMs タイプ I 地震動 (プレート境界型の大規模地震)</td><td rowspan="2">Prevent critical damage (SPG 2) 損傷が橋として致命的にならない性能 (耐震性能2)</td><td rowspan="2">Retain limited damage (SPG 3) 損傷が限定的なものにとどまり、橋としての機能回復が速やかに行い得る性能 (耐震性能3)</td></tr>
<tr><td>Type II GMs タイプ II 地震動 (兵庫県南部地震のような内陸直下型地震)</td></tr>
</table>

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)		

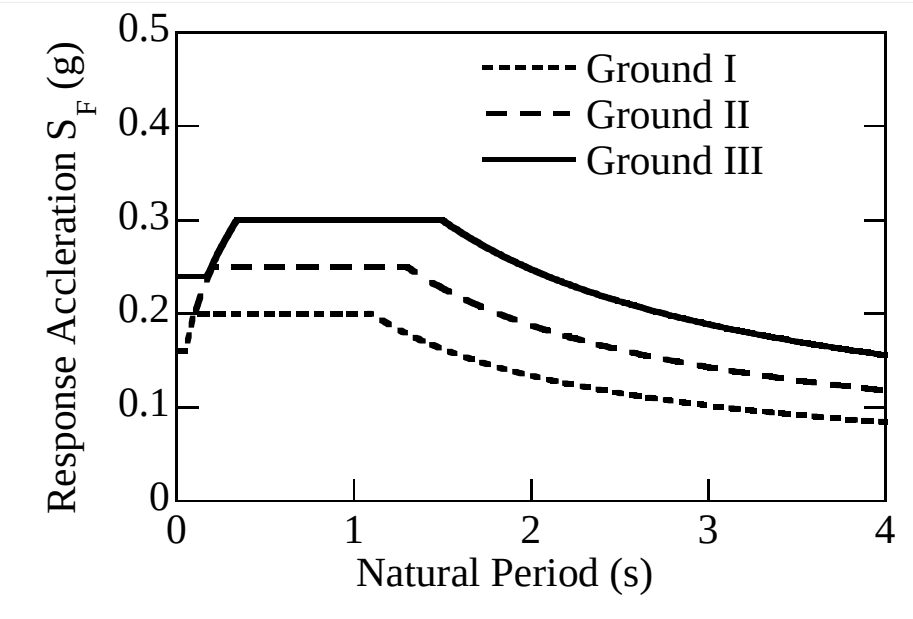
ATC and Caltrans (1999)

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

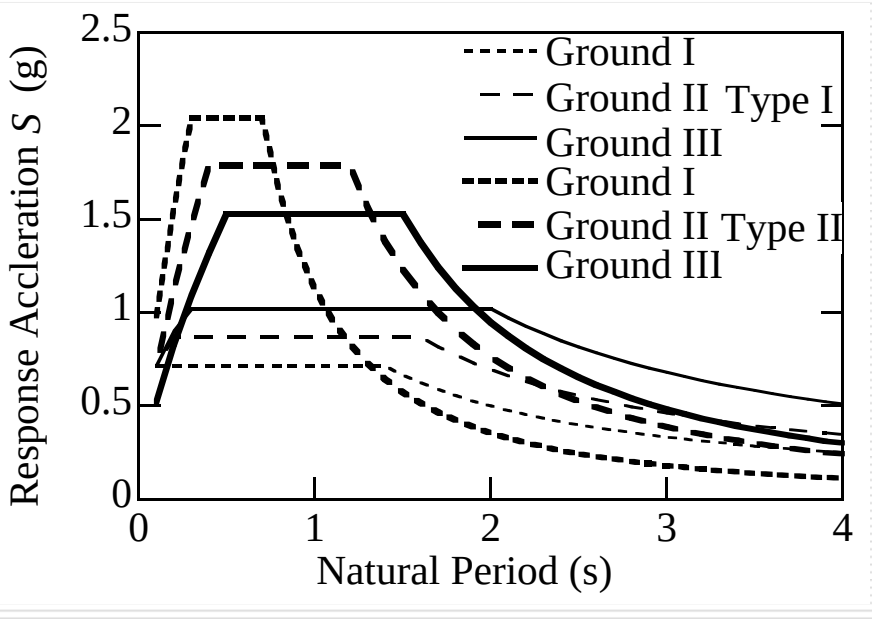
3) Seismic design force for road bridges 道路橋の設計地震力

$\xi = 0.05$

L1 (Function Evaluation)



L2 (Safety Evaluation)



Japan Road Association (2002)

L1 GMs

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

L2 GMs

タイプ I GMs

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

タイプ II GMs

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

4) Seismic Design Force in Other Regions

(1) EC code

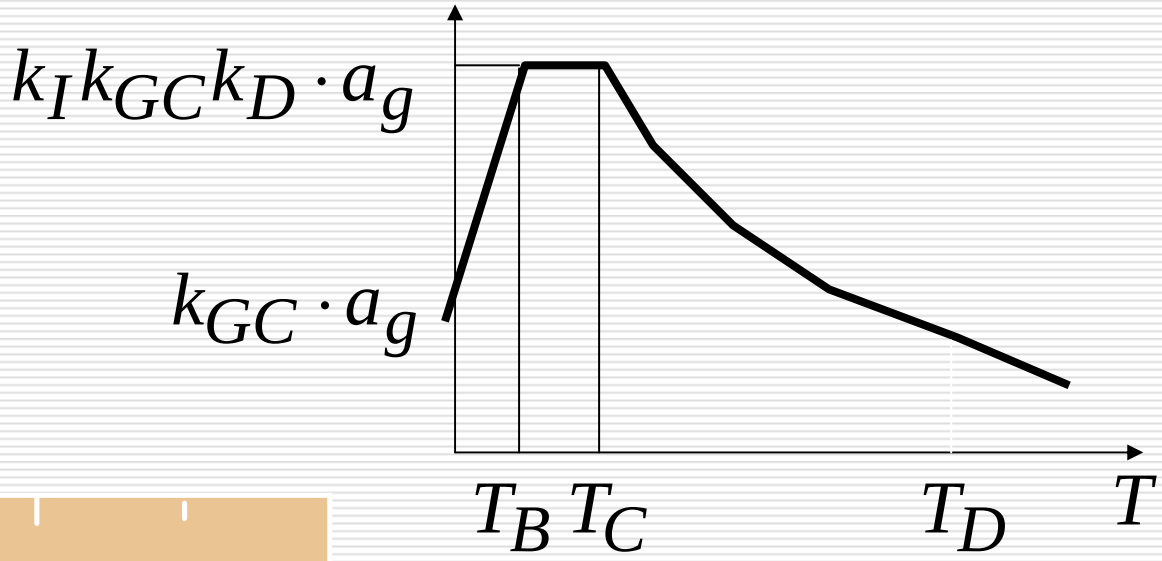
Elastic response accelerations for the referenced return period depending on soil condition

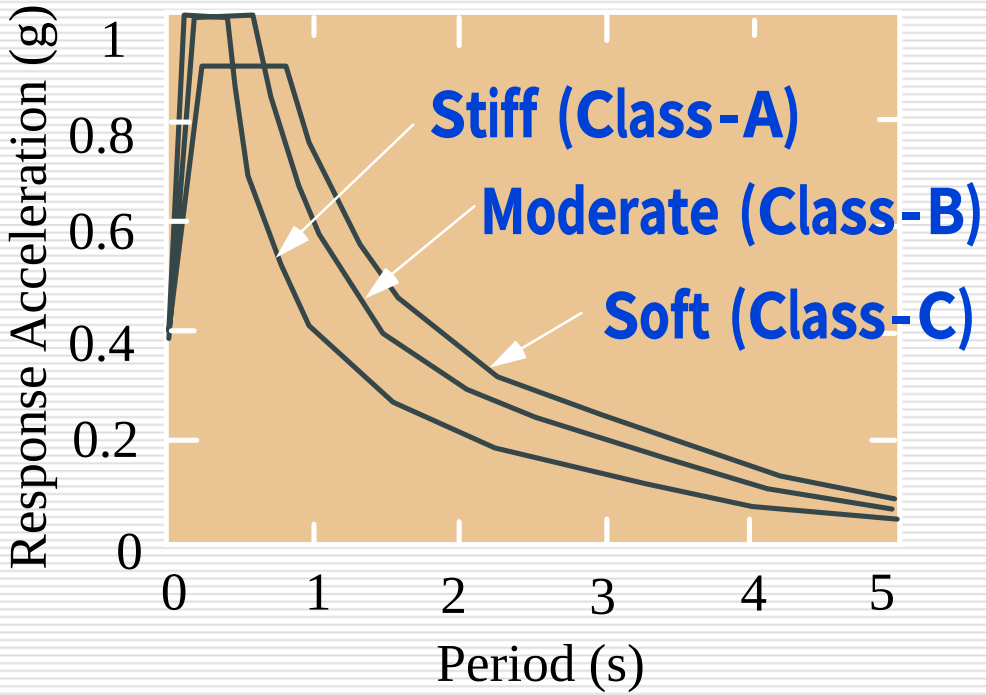
$$S = k_I \cdot k_{GC} \cdot a_g \cdot \begin{cases} 1 + \dfrac{T}{T_B}(k_D \beta_0 - 1) & 0 \le T \le T_B \\ k_D \cdot \beta_0 & T_B \le T \le T_C \\ k_D \cdot \beta_0 \cdot \left(\dfrac{T_C}{T}\right) & T_C \le T \le 3s \\ k_D \cdot \beta_0 \cdot \left(\dfrac{T_C}{3}\right) \cdot \left(\dfrac{3}{T}\right)^2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

0.9 & 1.0

1.3, 1.0, 0.7

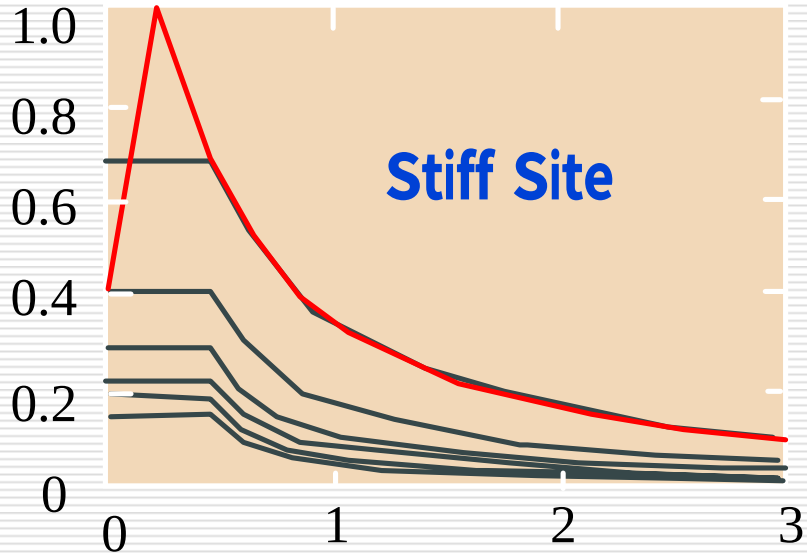
Eurocode 8
Part " Bridges"
1994

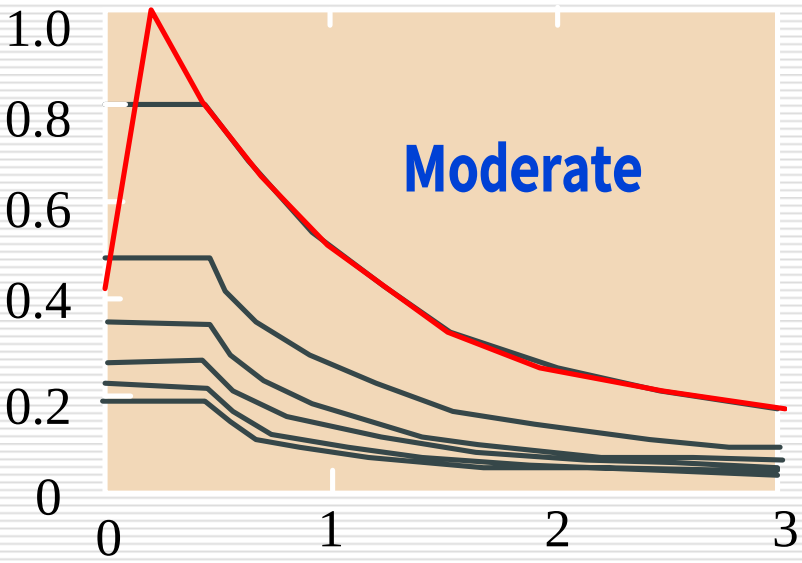


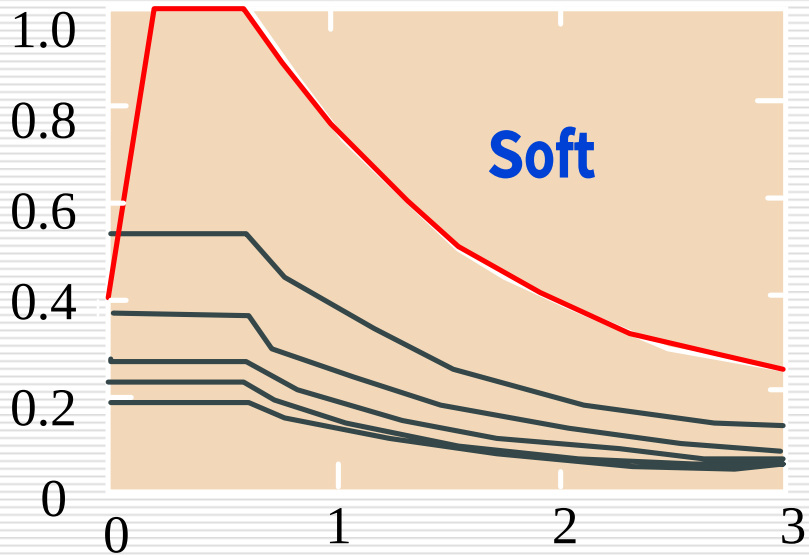


(2) Transit New Zealand Bridge Manual (1995)

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







(3) AASHTO (American Association of State Highway and Transportation Officials)

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

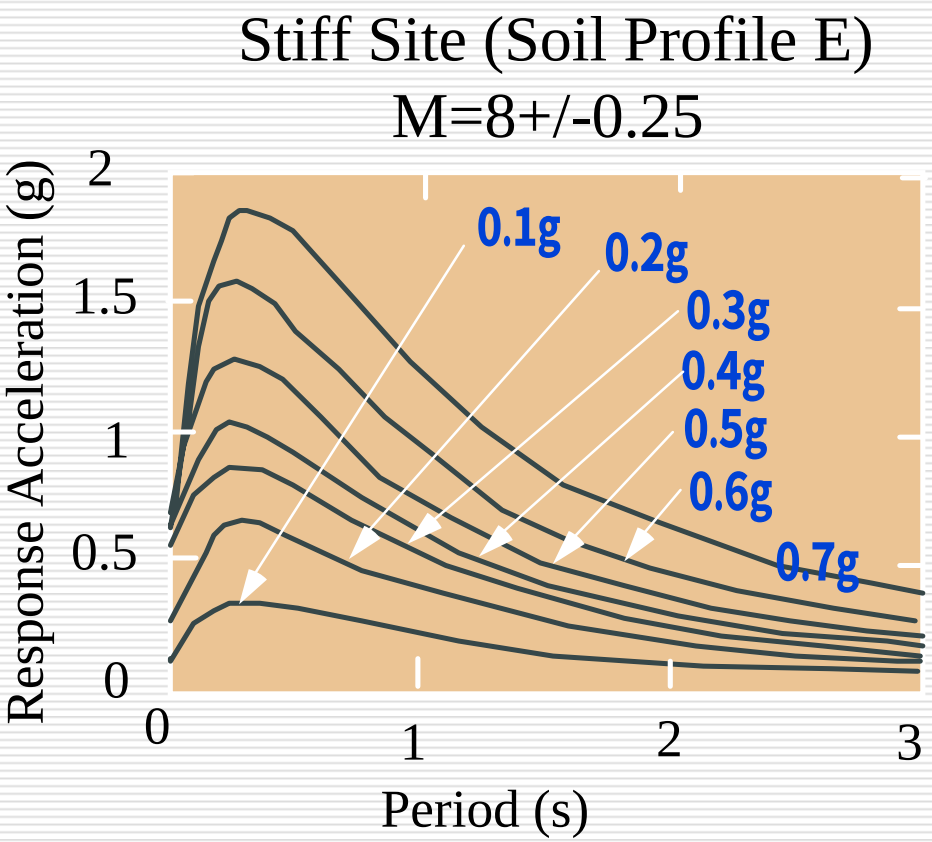
1.0, 1.2, 1.5, 2.0

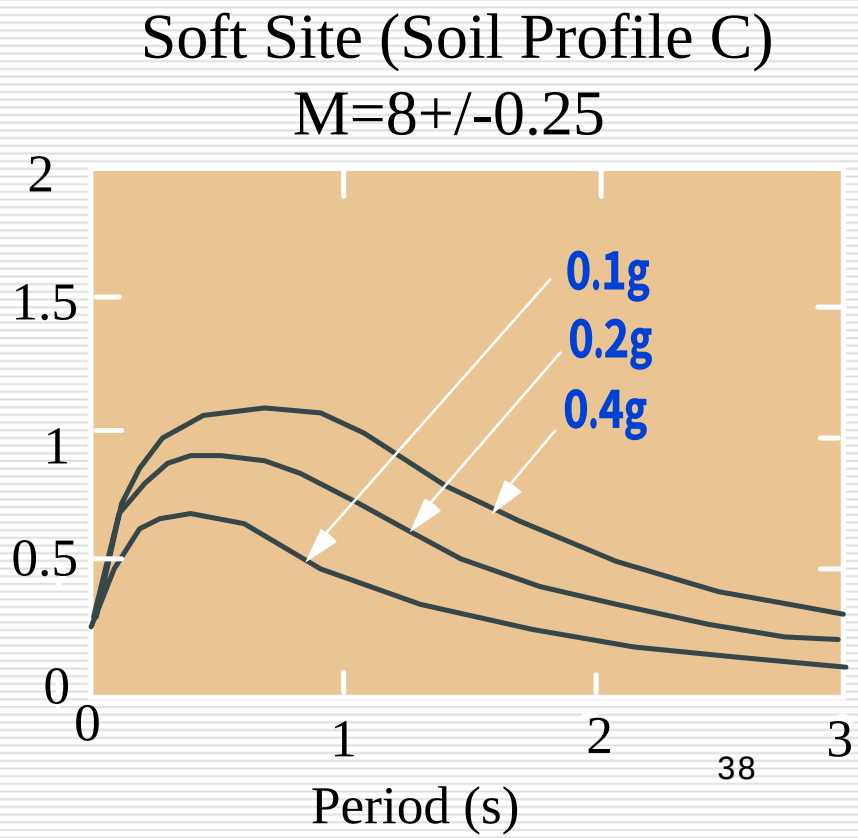
Given as a contour for T_R=475 years

(4) Applied Technical Council (ATC-32) & California Department of Transportation (Caltrans,1999)

Deterministic & probabilistic evaluation

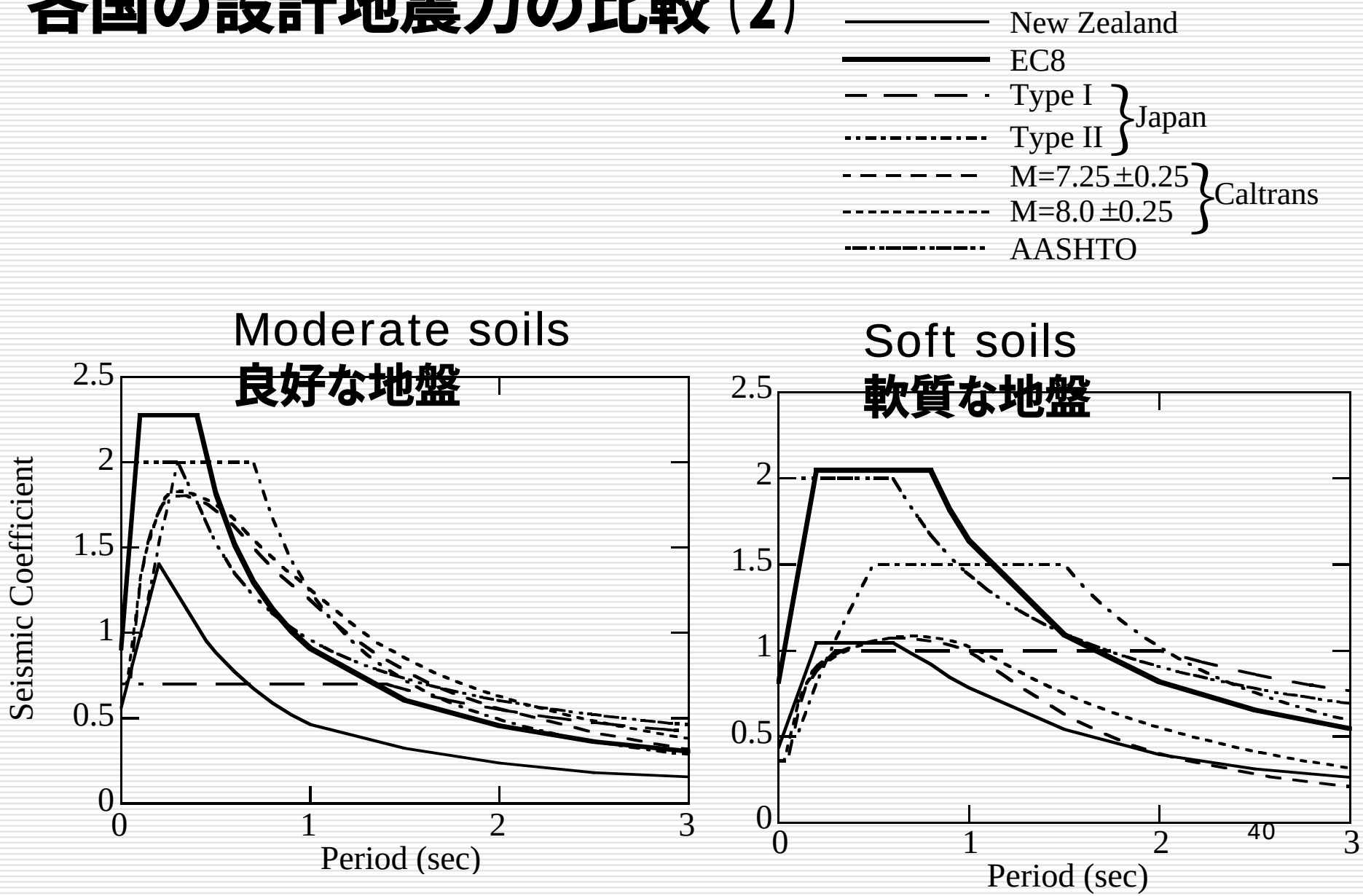
T_R=1,000-2,000 years





(5) Comparison with design forces
各国の設計地震力の比較

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
		a_g=0.7g		k_I=1.2
Soil profile II, III & IV	a_g=0.4g at soft site			
	M=7.25+/-0.25 M=8.0+/-0.25			
	Soil profile C, D. & E			

✓ 各基準で想定されている最大の応答加速度を求める。減衰定数は0.05とする。

Comparison of design forces
各国の設計地震力の比較（2）