Response Modification of Urban Infrastructure
都市施設の免震設計

(2) Chapter 2 Seismic Damage of Bridges due to Ground Vibration (Part 1)

(2) 2**章　地震動による橋梁の地震被害** (Part 1)

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2.1 What Types of Damage Does It Take Long Time for Repair?
どういう施設の被害が簡単に直らないか?

Bridges

1995 Kobe
Earthquake, Japan

Large Slope Failure

2008 Iwate-Miyagi
Earthquake, Japan

2.2 Why Is Mitigation of Seismic Damage Difficult?
-Japanese Experience of Mitigating of Seismic Damage of Bridges
どうして地震被害は簡単に減らないのか?
ー日本における橋梁の耐震設計の歴史

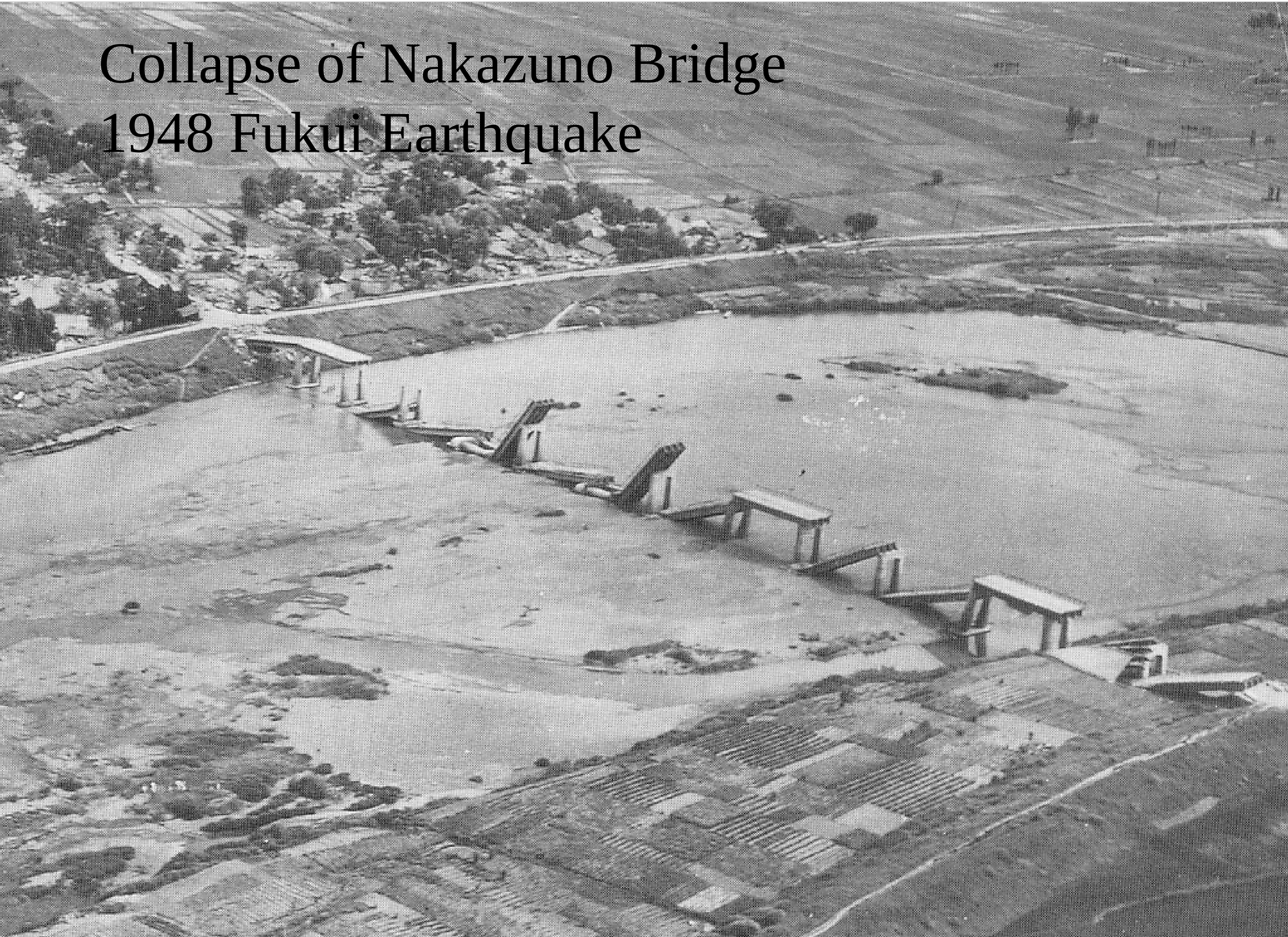
2.2.1 Stage I: Damage which occurred at the days when the seismic effect was not considered or poorly considered in design

- Modern technology was imported from UK, US, German and French after 1868 Meiji Revolution
- Of course, from the early days, the seismic effect must be considered in construction of structures. However technical was transferred by “from father to children” style, and often the technology did not stand on the sound engineering background.
- Because earthquake was not known in those countries, the seismic effect was disregarded.

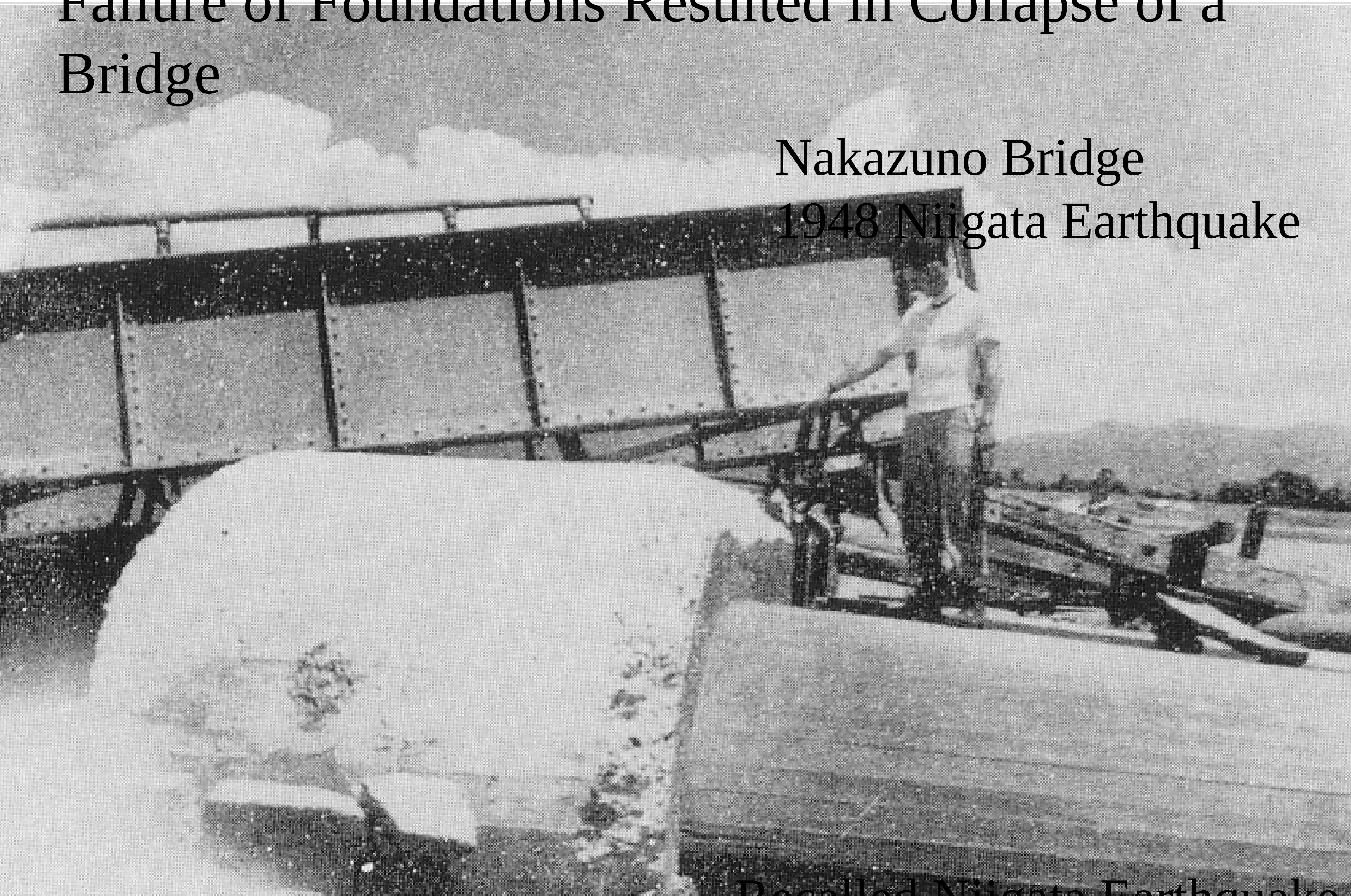
The importance of considering the seismic effects became soon apparent

1923 Kanto Earthquake

Collapse of Nakazuno Bridge
1948 Fukui Earthquake



Failure of Foundations Resulted in Collapse of a Bridge



Nakazuno Bridge
1948 Niigata Earthquake

Recalled Niigata Earthquake

Unique Topological & Geological Conditions in Asian Countries

- Being located in the monsoon area, the high-rate erosion of mountain regions developed thick sedimentation of soft soils at the mouth of large rivers.
- Most mega cities are resting on the thick sedimentation
- Foundation suffered damage resulting from the instability of clayey soil and the liquefaction and the lateral spreading of sandy soils.

Stage I (-1950s)

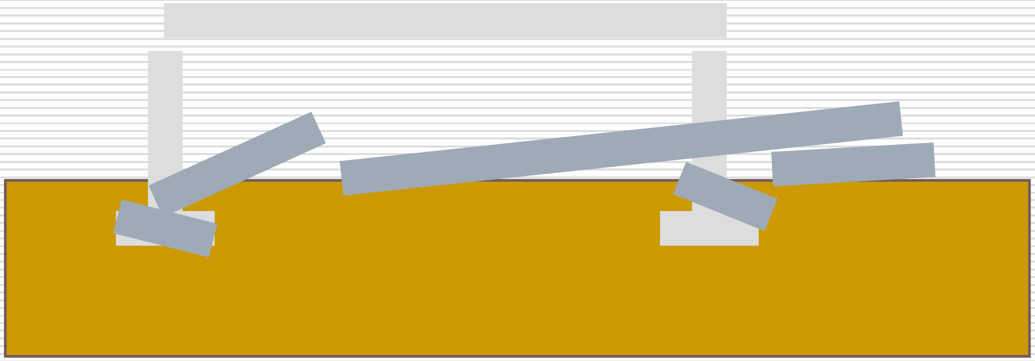
Seismic effects were not considered or poorly considered in design

1923 Kanto Earthquake
1946 Nankai Earthquake
1948 Fukui Earthquake

Tilting, Overturning and Settlement of Foundations

↓

Collapse



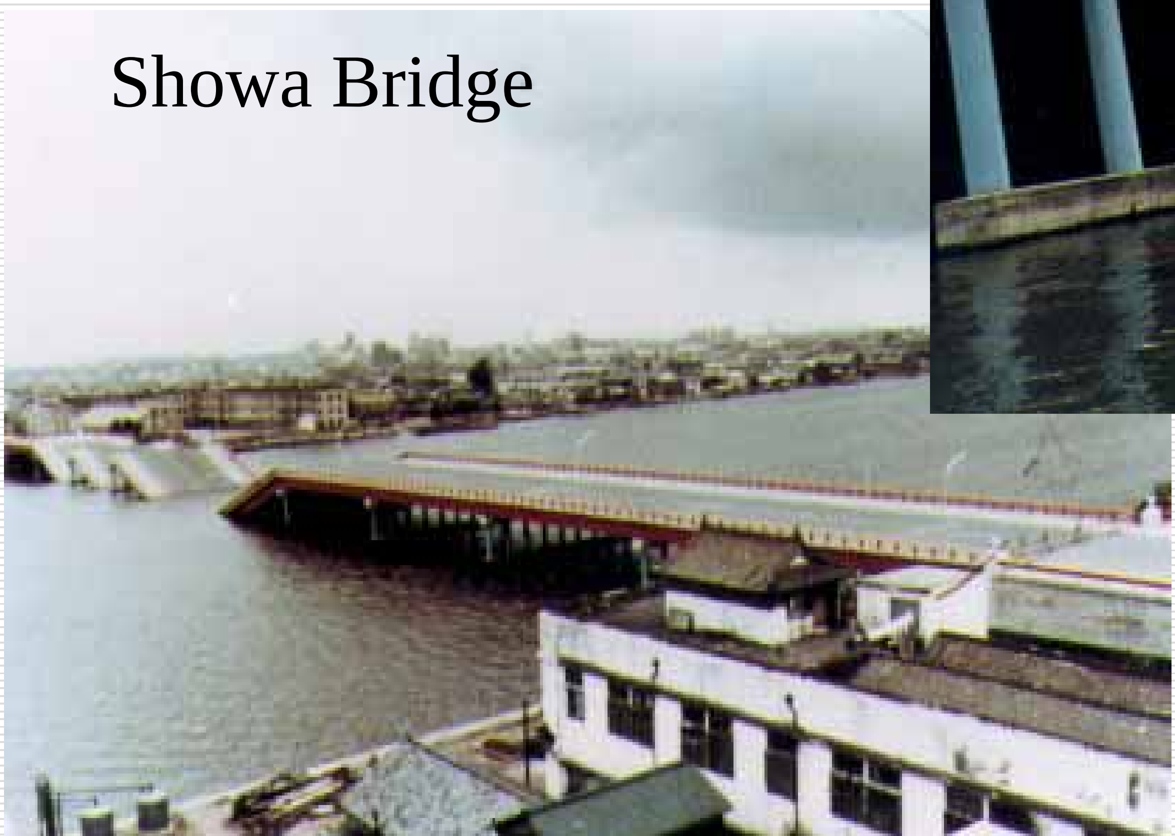
As a consequence of the extensive damage in the 1923 Kanto earthquake, seismic design was initiated in 1925

- Elastic static seismic design using 0.2-0.3 seismic coefficients based on the allowable stress design approach (**震度**0.2〜0.3**を用いた弾性静的横力法と許容応力度法を組み合わせた震度法**)
- Construction of massive & rigid piers with large sections started

2.2.2 Stage II (1960-1970s): Importance of considering soil liquefaction and unseating prevention devices was first recognized

1964 Niigata Earthquake

Showa Bridge



Soil Liquefaction

- In old documents there are many descriptions that water and soil spilled out from cracks of the ground during an earthquake.

- After the 1964 Niigata earthquake, this phenomena was first named as “liquefaction” (**液状化**) by a Japanese professor (Professor Mogami), and scientific research was initiated worldwide for the mechanism of liquefaction.

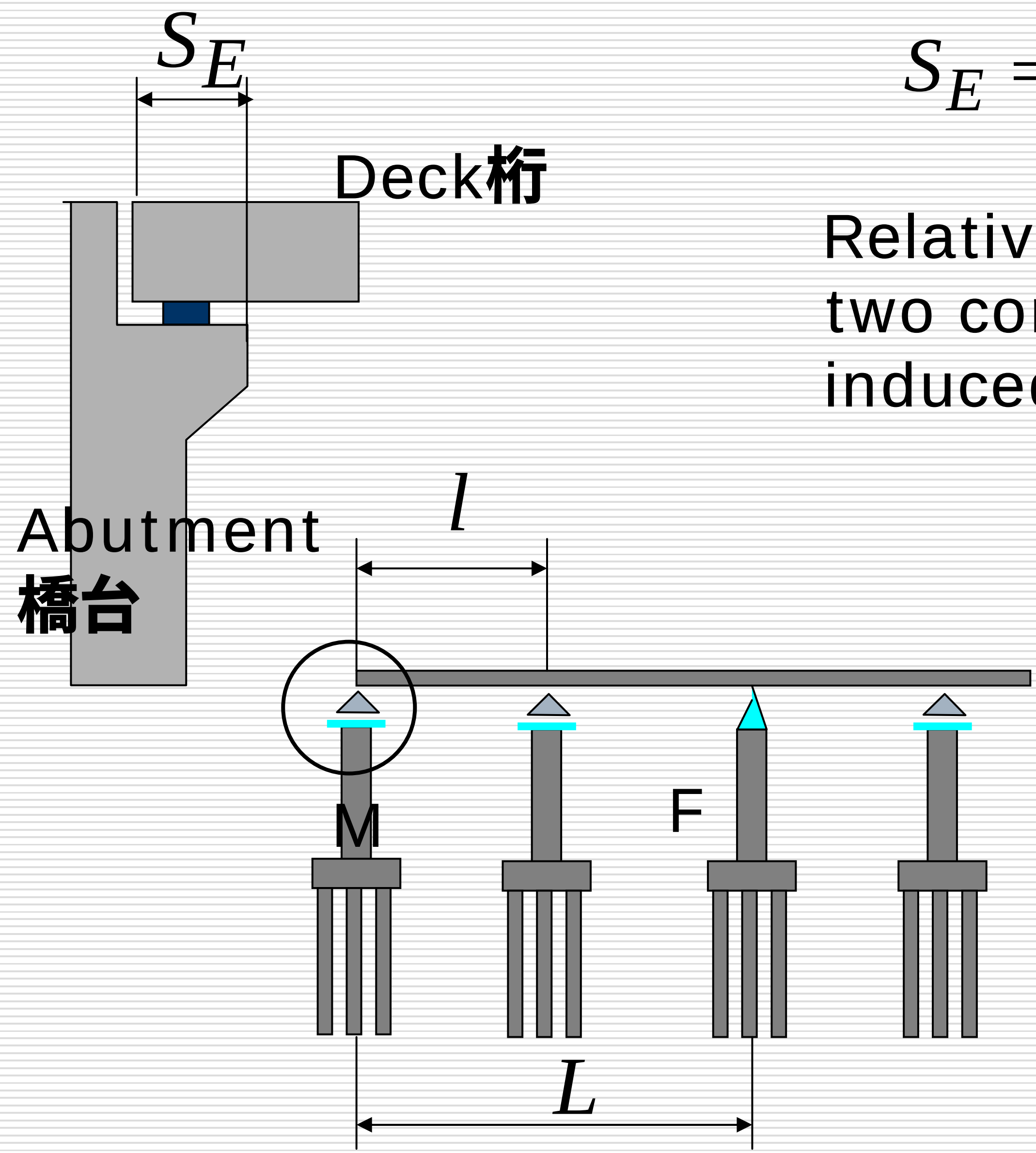
- Fact that the ground moved during or after an earthquake (lateral spreading) (**流動化**) was described in many reports about Niigata earthquake, however research initiated after Niigata earthquake was directed to only liquefaction. It was the late 1980s when the importance of lateral spreading was identifiedt.

The experience of Niigata earthquake resulted in the first development of unseating prevention devices (落橋防止構造)

Unseating Prevention Devices (落橋防止構造)

- Effectiveness of unseating prevention devices was first identified by a Japanese engineer who studied the damage of bridges after 1964 Niigata earthquake.
- He proposed to
  - ✓extend the seat length
  - ✓provide connection between adjacent decks
  - ✓connect the deck to the substructures
- This practice was incorporated in the seismic retrofit of existing bridges first, and then incorporated in the 1971 JRA Guide Specifications for Seismic Design (**道路橋耐震設計指針**) . The practice was then spread worldwide.

Seat Length (桁かかり長の確保)



Deck displacement relative to substructure

$$S_E = u_R + u_G \quad > S_{EM}$$

Relative displacement between two controlling substructures induced by seismic ground strain

$$u_G = \varepsilon_G L$$

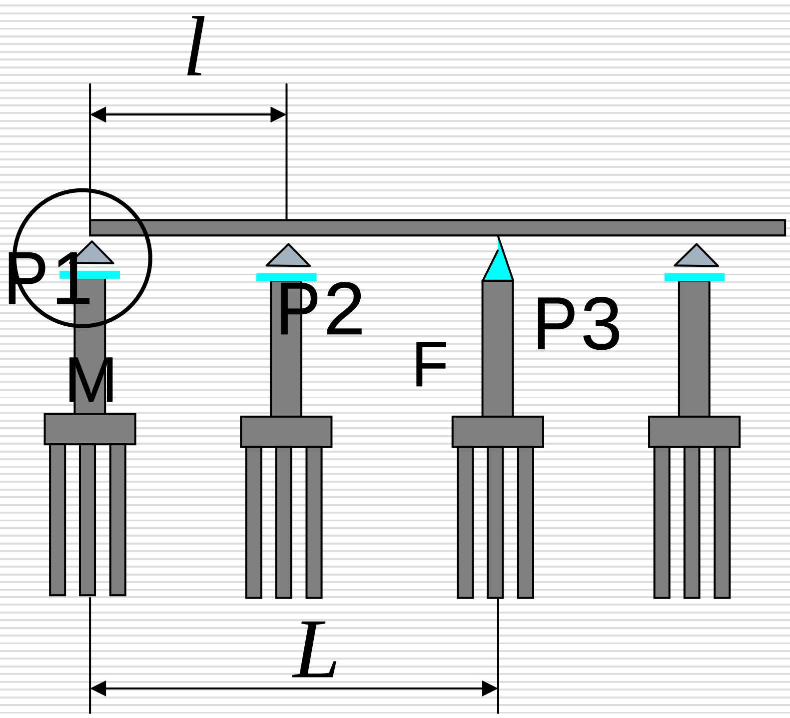
ε_G :Design ground strains (0.0025、0.00375、0.005)設計地盤ひずみ

L :Distance between two controlling substructures

$$S_{EM} = 0.7 + 0.005l$$

Example of Seat Length S_E

- Assume that the deck is supported by elastomeric bearings, and that deck response displacement relative to the top of a pier is 0.5m
- Assume that $l = 30m$, $L = 60m$ and Group III soil condition.



- Because $\varepsilon_G = 0.005$,

$$u_G = \varepsilon_G L$$
$$= 0.005 \times 60m = 0.3m$$

- Thus,

$$S_E = u_R + u_G$$
$$= 0.5 + 0.3 = 0.8m$$

- The minimum value is

$$S_{EM} = 0.7 + 0.005l$$
$$= 0.7 + 0.005 \times 30 = 0.85$$

- Consequently, $S_E = 0.85m$

Restrainers (桁間連結装置、桁と下部構造を結ぶ構造)

1) Connection of a bridge to a substructure

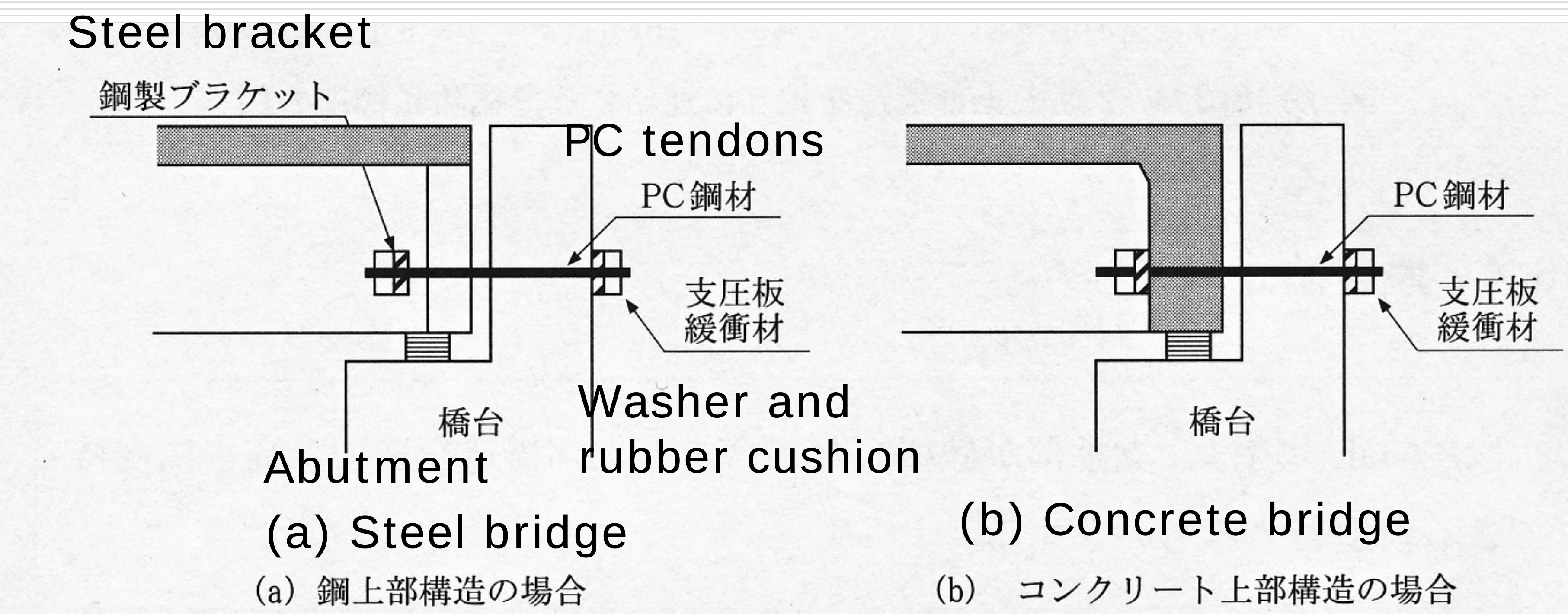
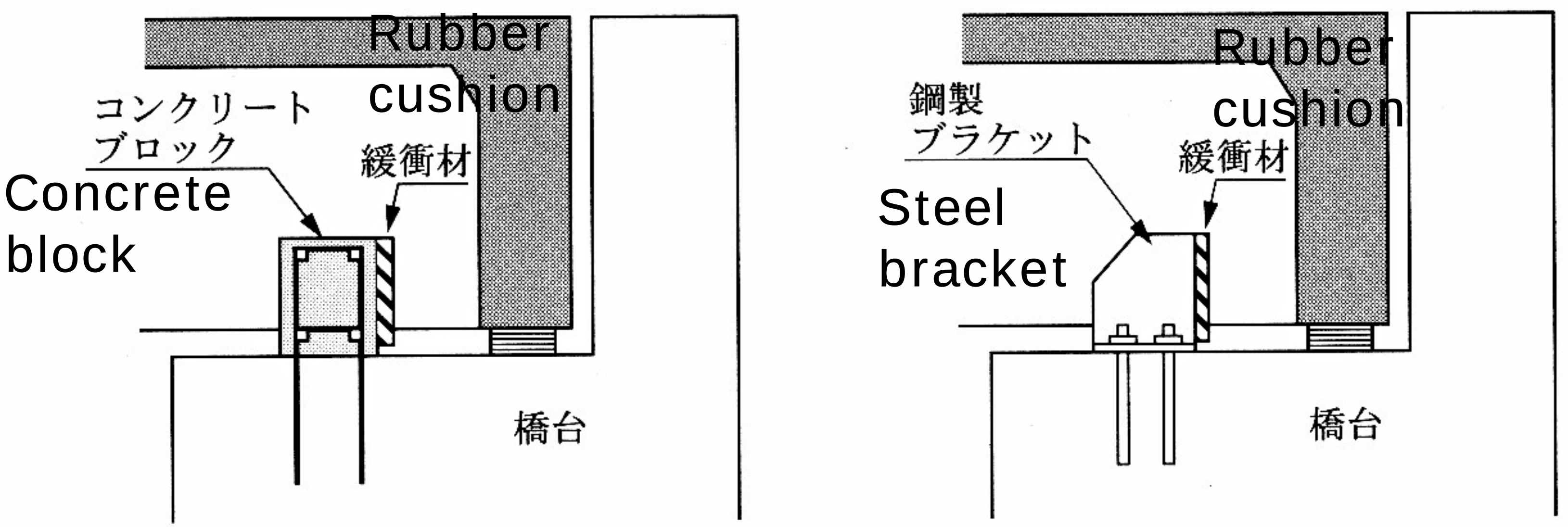


図-解 16. 3. 1　上部構造と下部構造を連結する落橋防止構造の例

2) Set of Stoppers



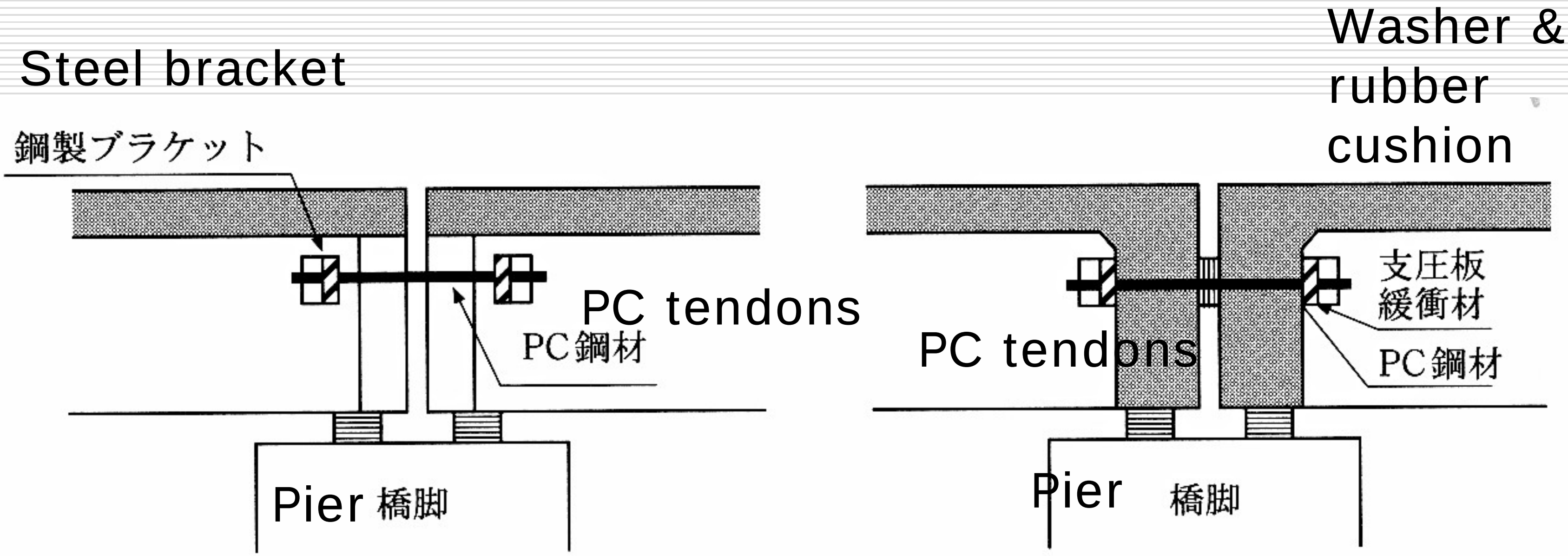
(a) Concrete block

(b) Steel bracket

(a) コンクリートブロックを用いる落橋防止構造　　(b) 鋼製ブラケットを用いる落橋防止構造

図-解 16.3.2　上部構造及び下部構造に突起を設ける落橋防止構造の例

3) Connection of adjacent bridges



(a) Steel bridges
(a)　鋼上部構造の場合

(b) Concrete bridges
(b)　コンクリート上部構造の場合

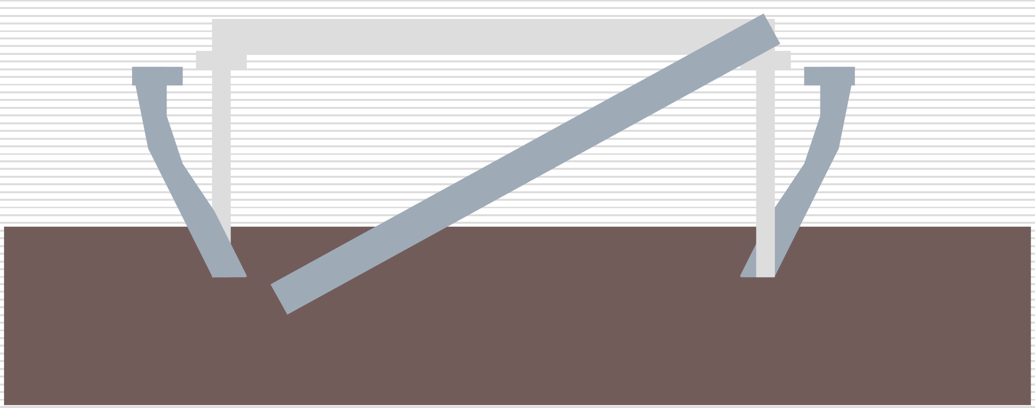
図-解 16.3.3　2 連の上部構造を相互に連結する落橋防止構造の例

Examples of Unseating Prevention Device

Stage II: Damage which occurred until the importance of soil liquefaction and unseating prevention devices was recognized

- Consideration to soil liquefaction and unseating prevention devices were not recognized in the seismic design practice prior to 1964

- Excessive relative displacement of decks resulted from soil liquefaction



Collapse

1971 Guide Specifications on Seismic Design of Highway Bridges (道路橋耐震設計指針)

- Modified seismic coefficient method (修正震度法) which incorporated natural period, soil condition and importance dependence of the seismic coefficient （設計震度） was introduced
- Evaluation of vulnerability for liquefaction was first included in the world
- Unseating prevention devices were first included in the world

Modified Seismic Coefficient (**修正震度**)

$$k_h = c_I \cdot c_Z \cdot c_{GC} \cdot c_T \cdot k_{h0}$$

Standard seismic coefficient (**標準設計震度**)

$$k_{h0} = 0.2$$

Importance factor(**重要度別補正係数**)

$$c_I = \begin{cases} 1.0 & \text{Important} \\ 0.8 & \text{Less Important} \end{cases}$$

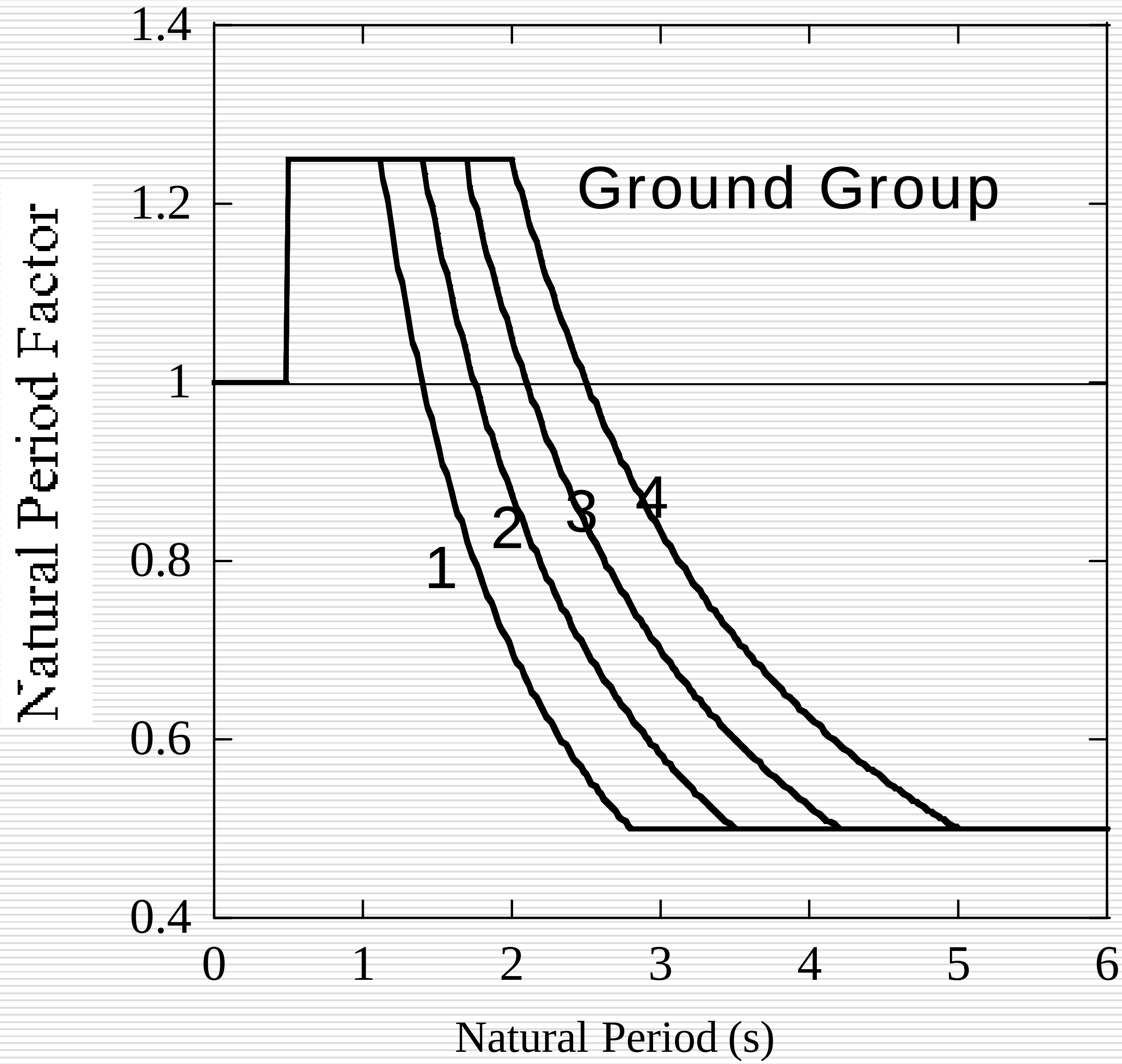
Zoning Factor (**地域別補正係数**)

$$c_Z = \begin{cases} 1.0 & \text{A-zone} \\ 0.85 & \text{B-zone} \\ 0.7 & \text{C-zone} \end{cases}$$

Ground condition factor（地盤別補正係数）

$$c_{GC} = \begin{cases} 0.7 & \text{Ground group 1} \\ 0.8 & \text{Ground group 2} \\ 0.9 & \text{Ground group 3} \\ 1.0 & \text{Ground group 4} \end{cases}$$

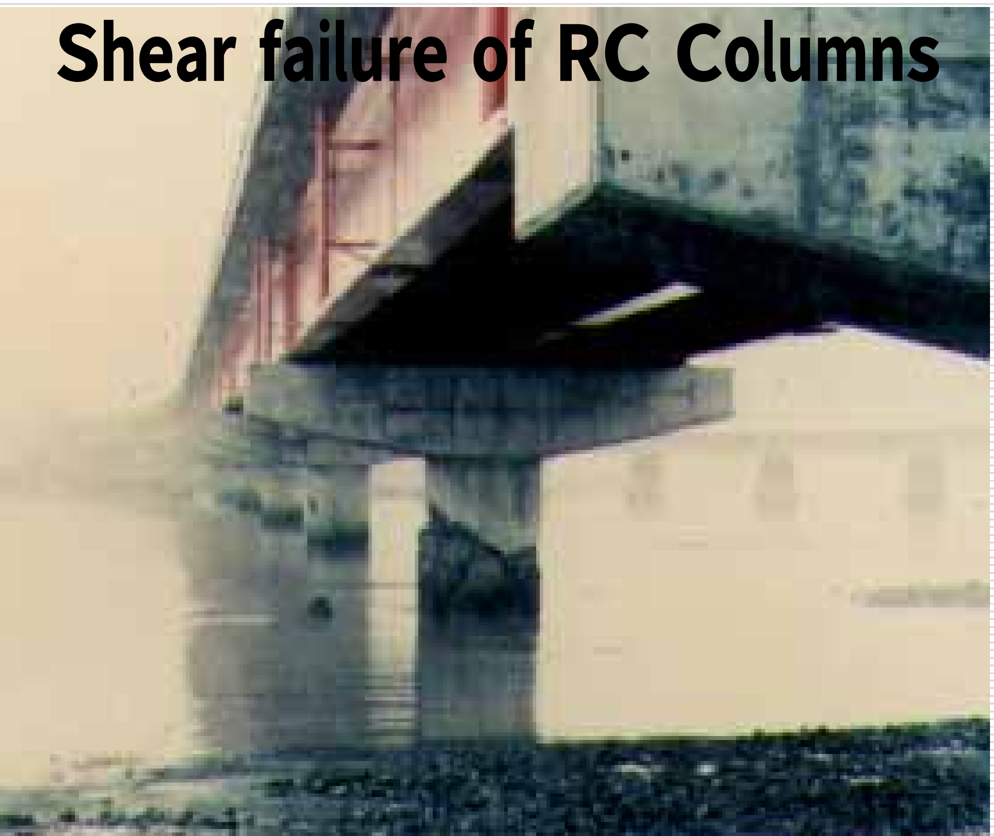
Natural period factor (固有周期補正係数)



2.2.3 Stage-III: Damage resulting from insufficient ductility of columns and strength of bearings (橋脚の不十分なじん性と支承の強度不足)

1982 Urakawa-oki Earthquake
浦河沖地震

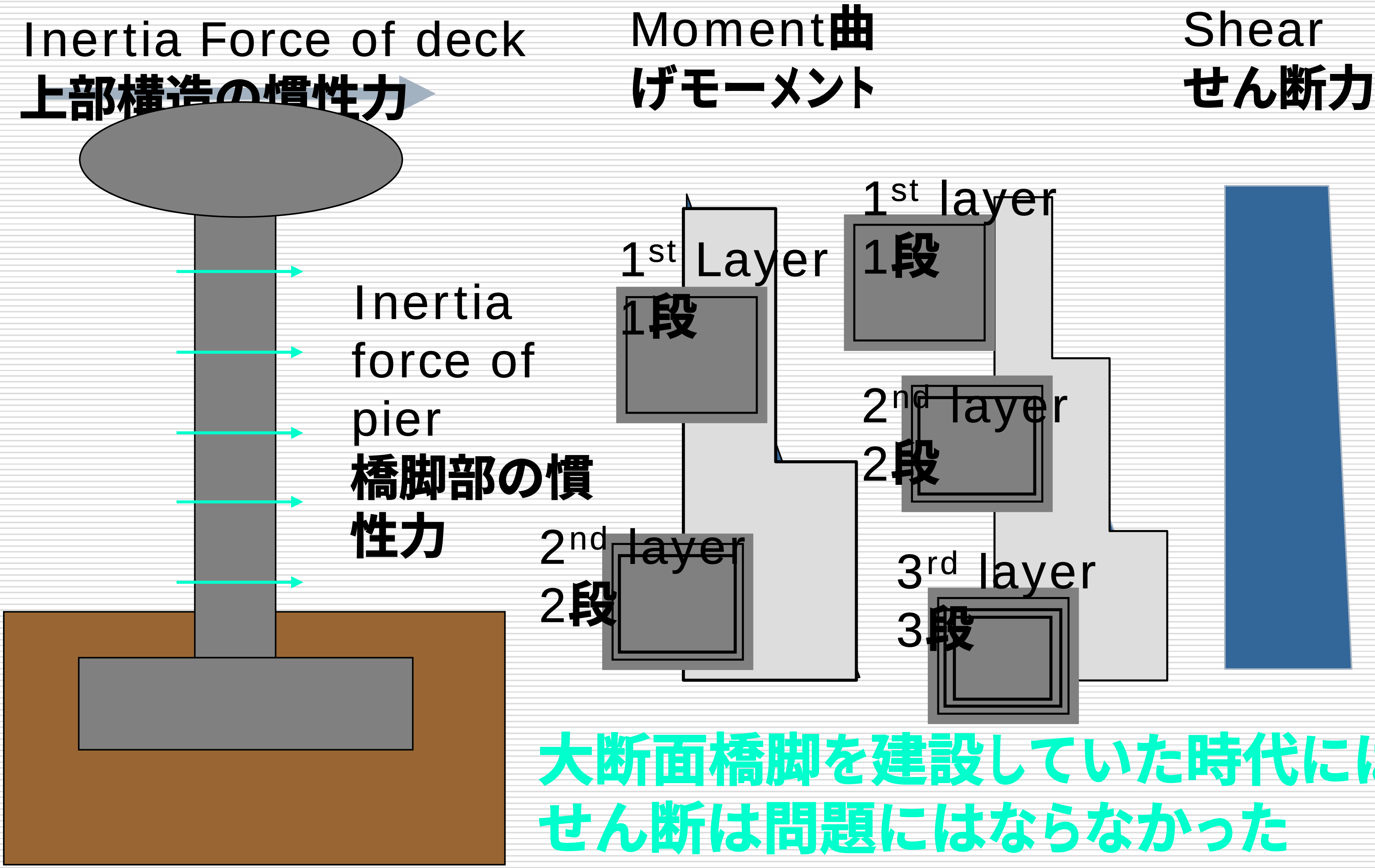
Shear failure of RC Columns



Shizunai Bridge
静内橋

Premature Shear Failure of RC Piers
橋脚のせん断破壊

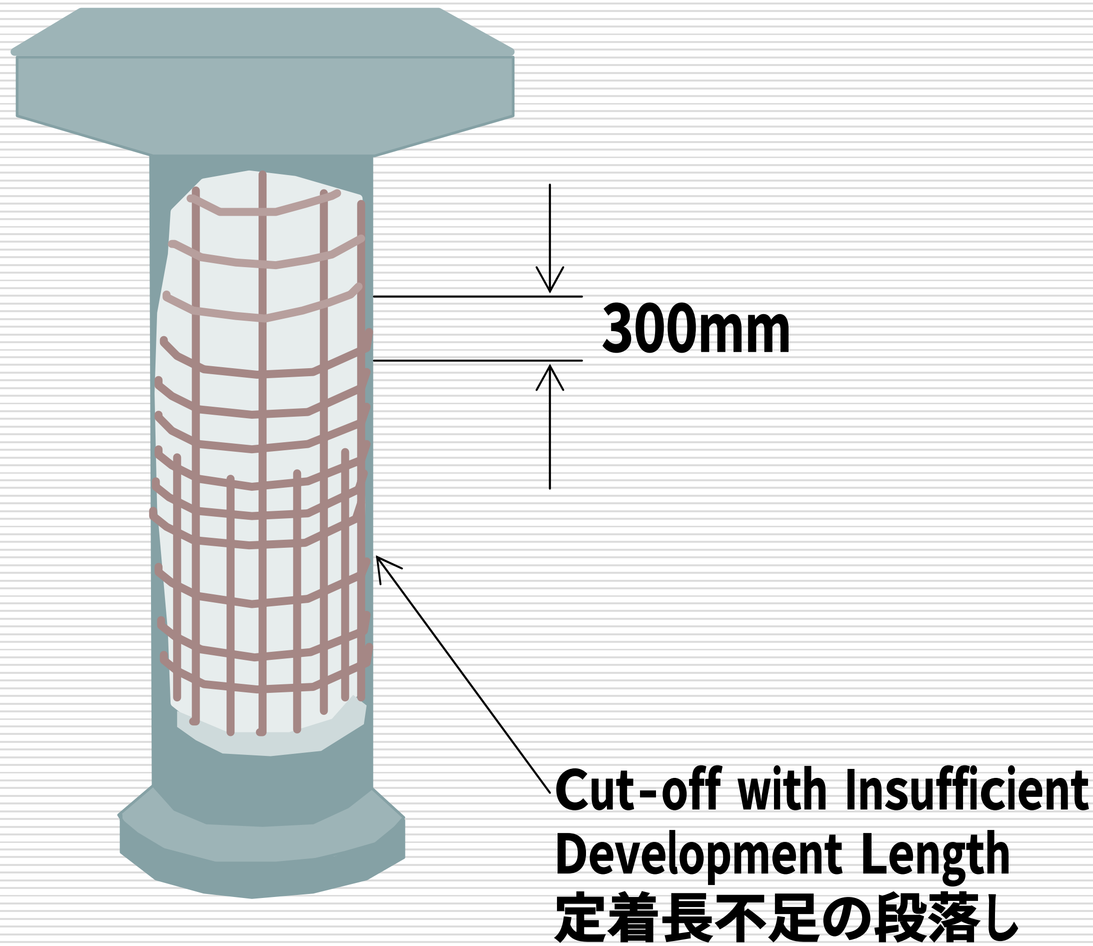
Why is the number of longitudinal bars reduced at the upper part of a pier?



大断面橋脚を建設していた時代には、せん断は問題にはならなかった

Shear Failure of Reinforced Concrete Piers Resulting from Insufficient Development Length
定着長不足の主鉄筋段落しに起因するせん断破壊

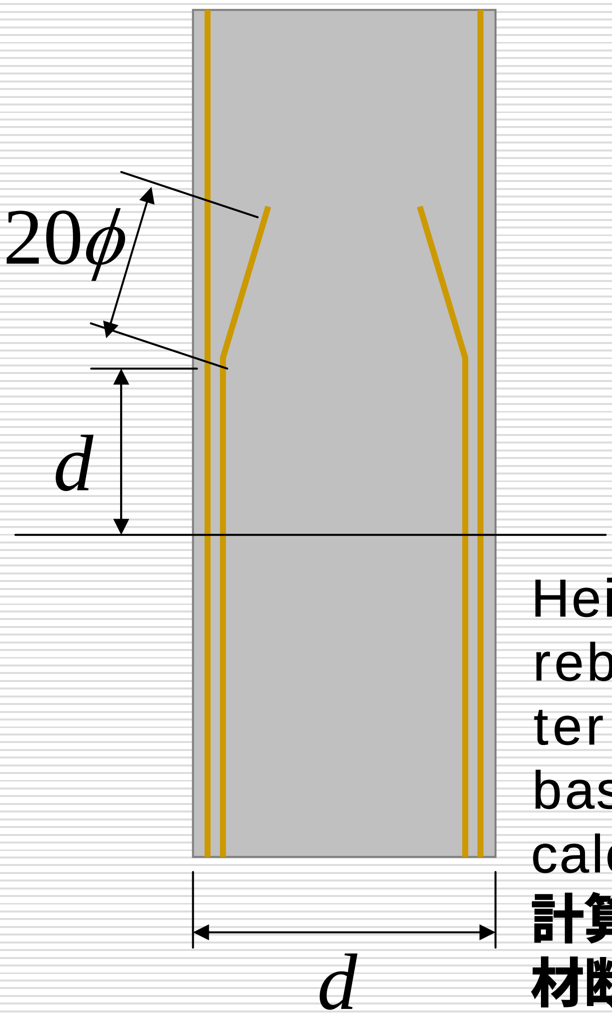
Common Design Practice prior to 1985

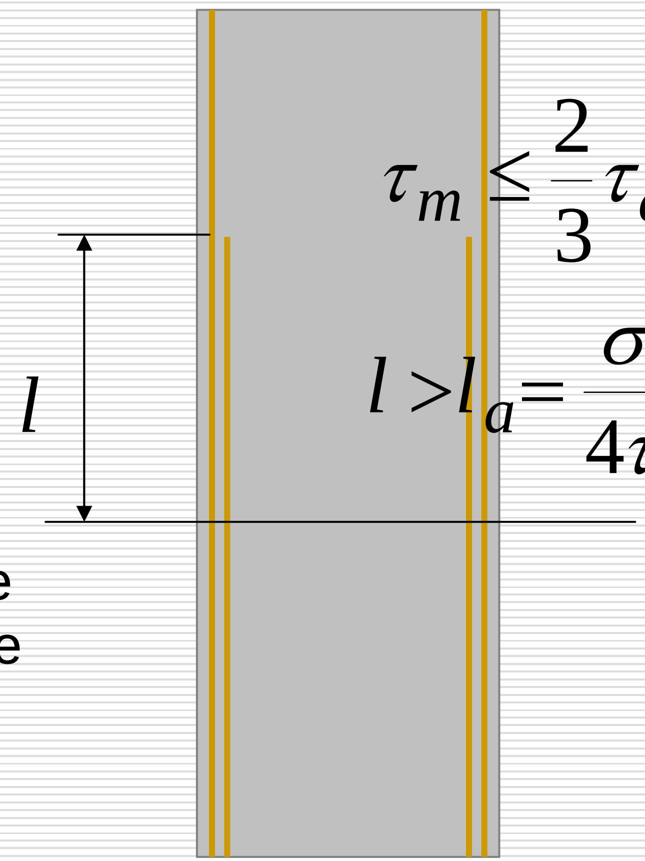


Current Practice for Development

Developed by bending inside
主鉄筋を内側に曲げて定着

Develop by extending bars with a certain length
主鉄筋をまっすぐ定着する場合



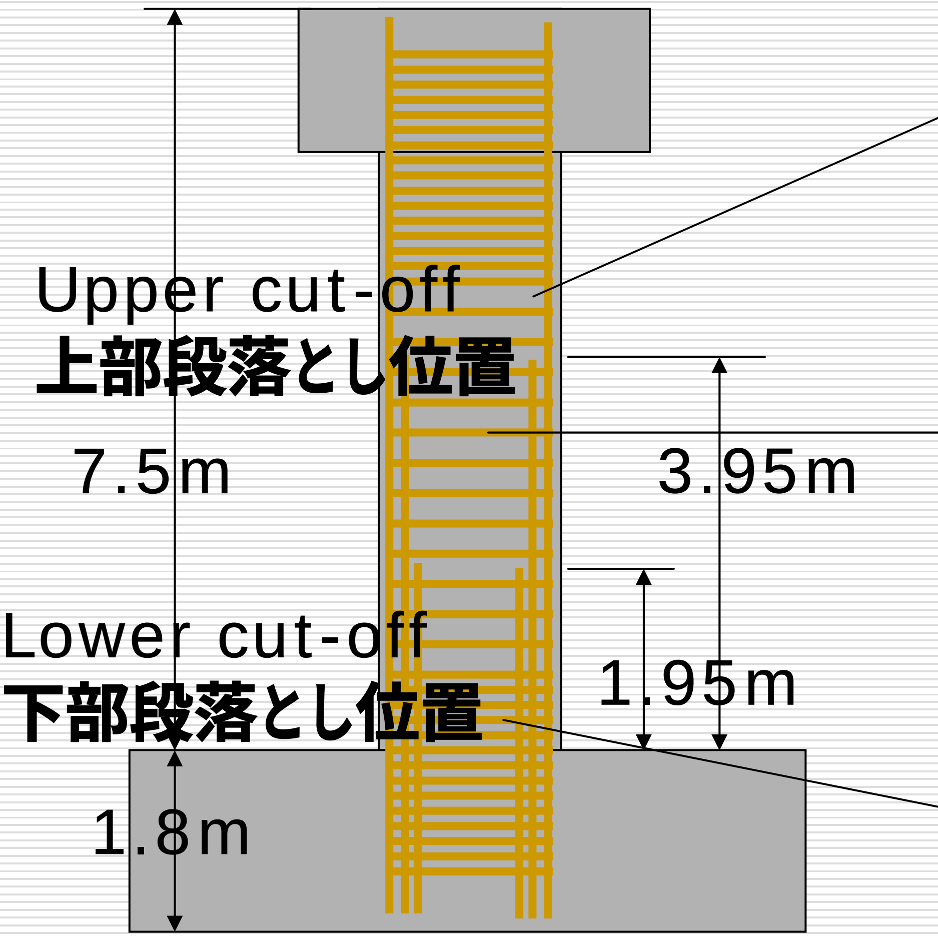


$$\tau_m \leq \frac{2}{3}\tau_{a1}$$

$$l > l_a = \frac{\sigma_{sa}}{4\tau_{0a}}\phi$$

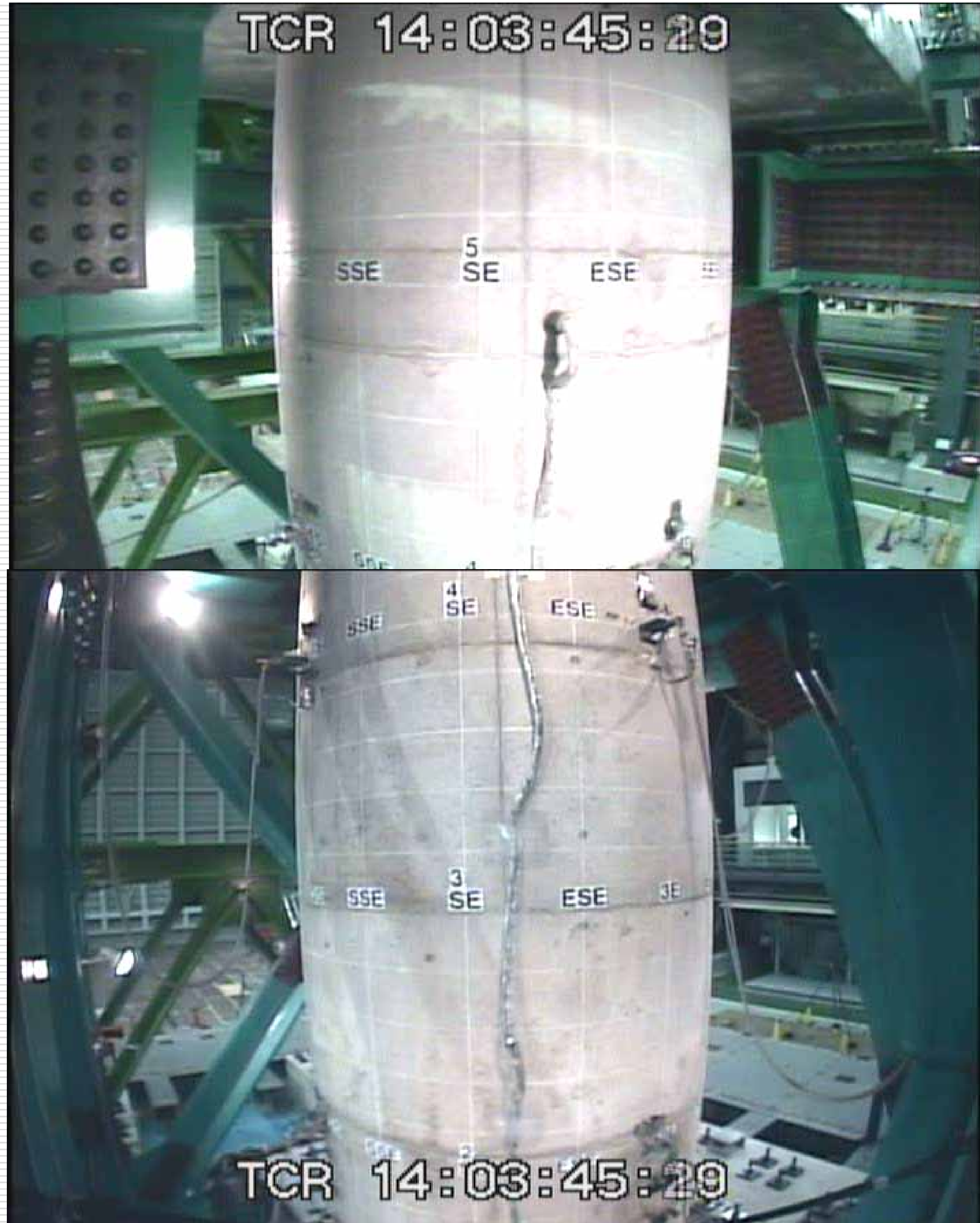
Height where rebars can be terminated based on calculation
計算上不要となる部材断面

How does a column with
段落しのある橋脚 (C1-2橋脚)



Shear Failure of RC Column due to Insufficient Termination of Longitudinal Bars
Shake Table Experiment at E-Defense

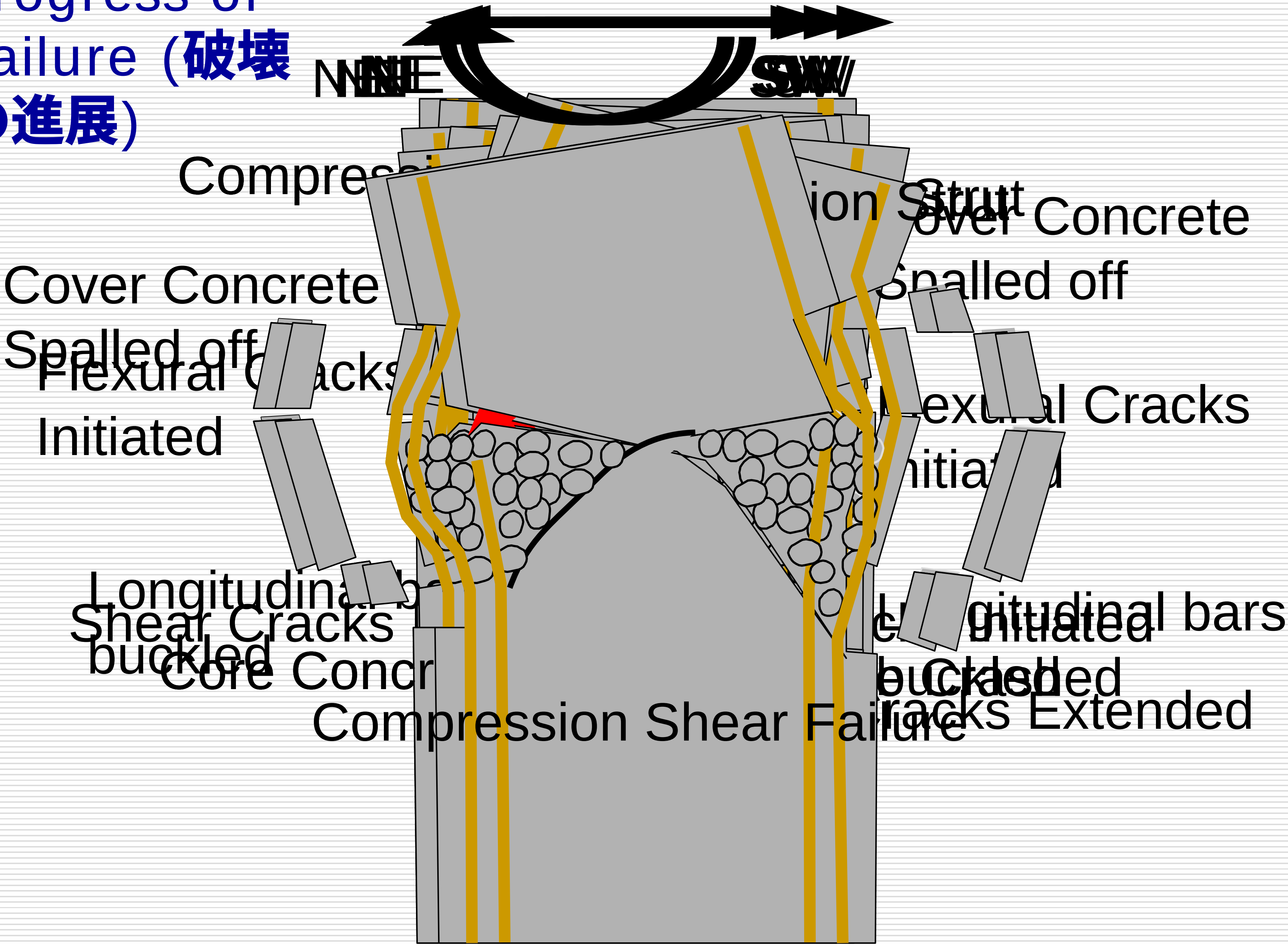
Shear Failure of RC Column due to Insufficient Termination of Longitudinal Bars



Shear Failure of RC Column due to Insufficient Termination of Longitudinal Bars

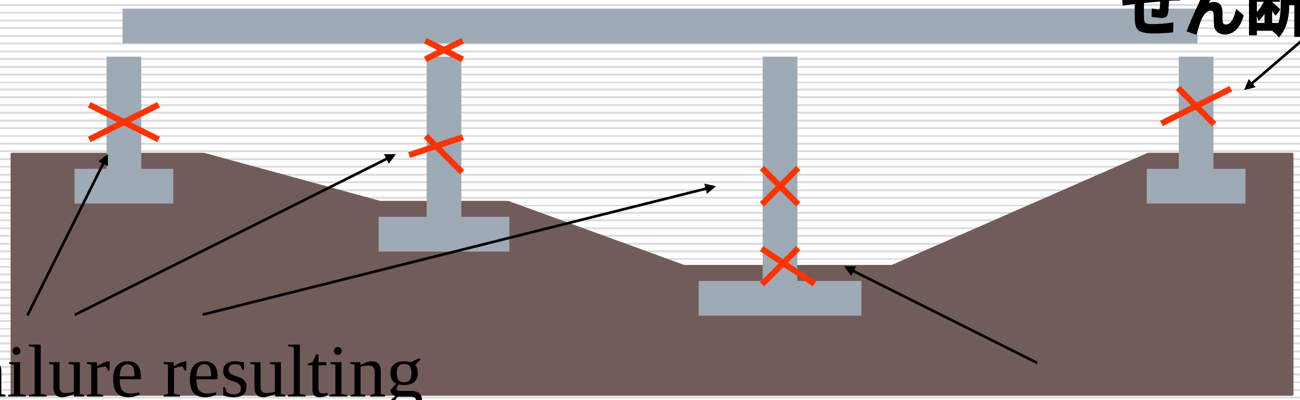
Flexural Failure of a RC Column (RC橋脚の曲げ破壊)

Progress of Failure (破壊の進展)



Stage-III: Damage resulted from insufficient ductility of columns and strength of bearings

1978 Miyagi-ken-oki Earthquake
1982 Urakawa-oki Earthquake
1993 Hokkaido-toho-oki Earthquake

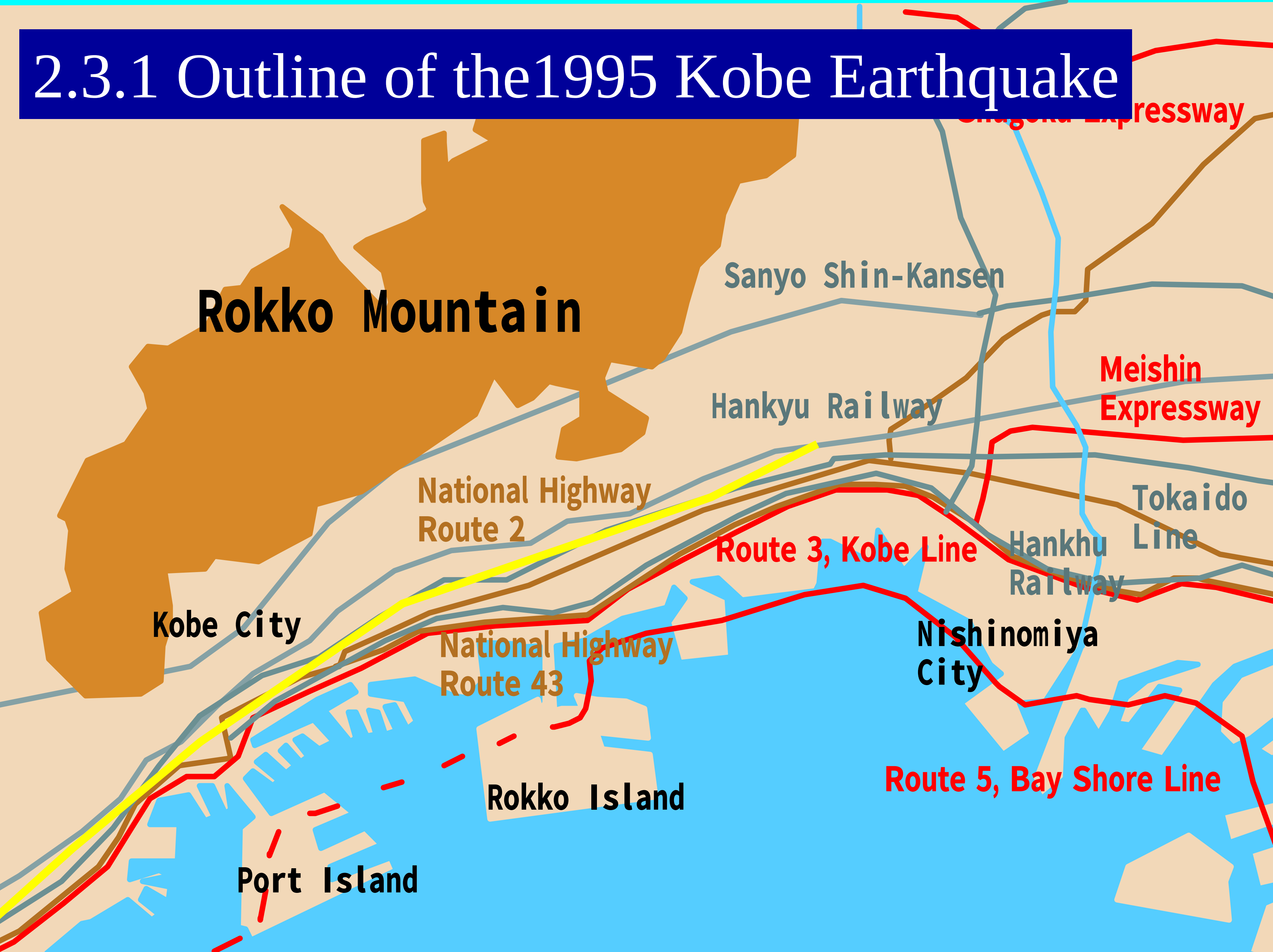


2.2.4 Features of Japanese Seismic Design Practice Prior to the 1995 Kobe Earthquake

- A number of experience for seismic damage in the past
- Larger seismic design force
- Development and early implementation of Unseating Prevention Devices since 1971
- Countermeasures for Liquefactions since 1971

2.3 Impact of the 1995 Kobe, Japan Earthquake

2.3.1 Outline of the1995 Kobe Earthquake



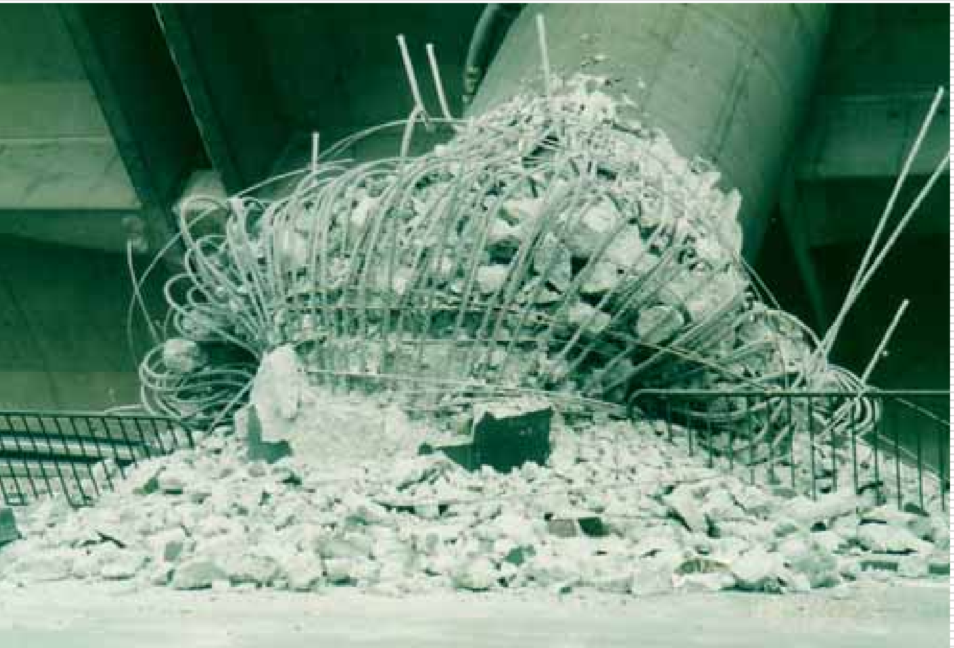

Bridges are vital component of urban areas

2.3.2 Various Damage

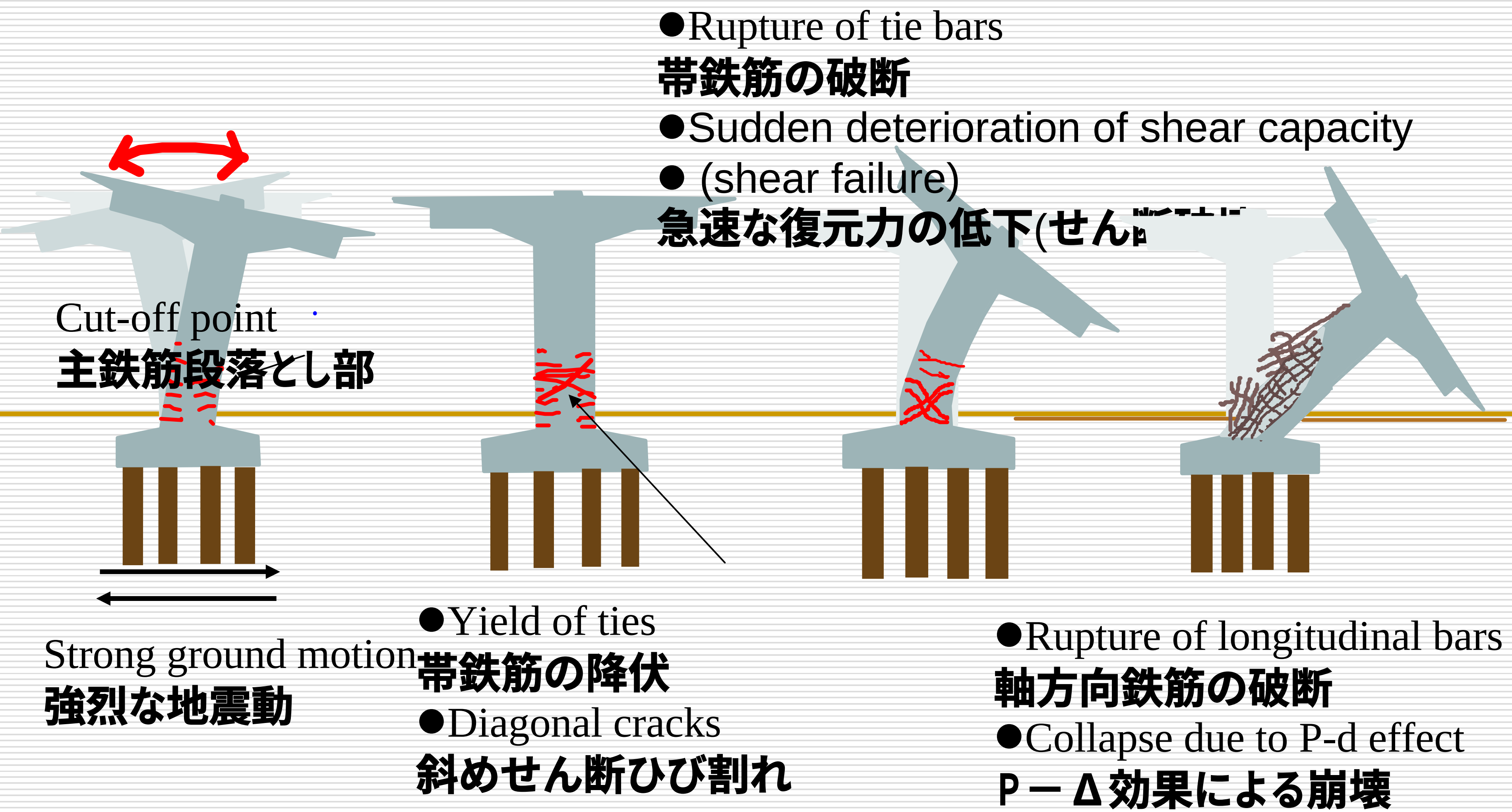
1) Shear Failure of RC Columns due to Termination of Longitudinal bars with Insufficient Development

The extensive damage occurred at a 18-span viaduct. The viaduct collapsed due to failure of RC columns resulting from the shear failure at terminated zone of longitudinal reinforcements with insufficient development (定着長不足の軸方向鉄筋段落し部のせん断破壊によって多数の橋が落橋した).

18 Span collapsed at Fukae Viaduct
18径間連続橋
深江高架橋(ピルツ橋)



Collapse Mechanism of Fukae Viaduct



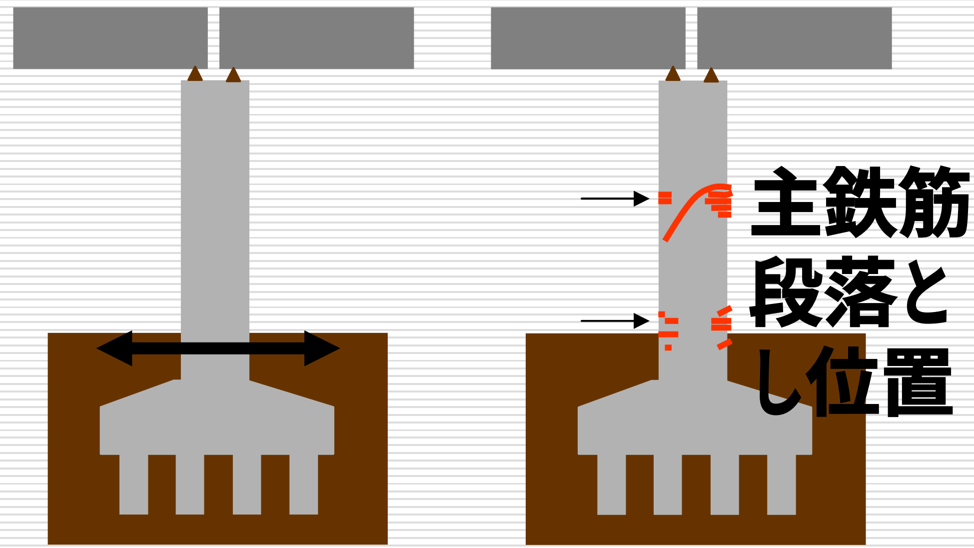
建設省震災調査委員会報告（平成7年）

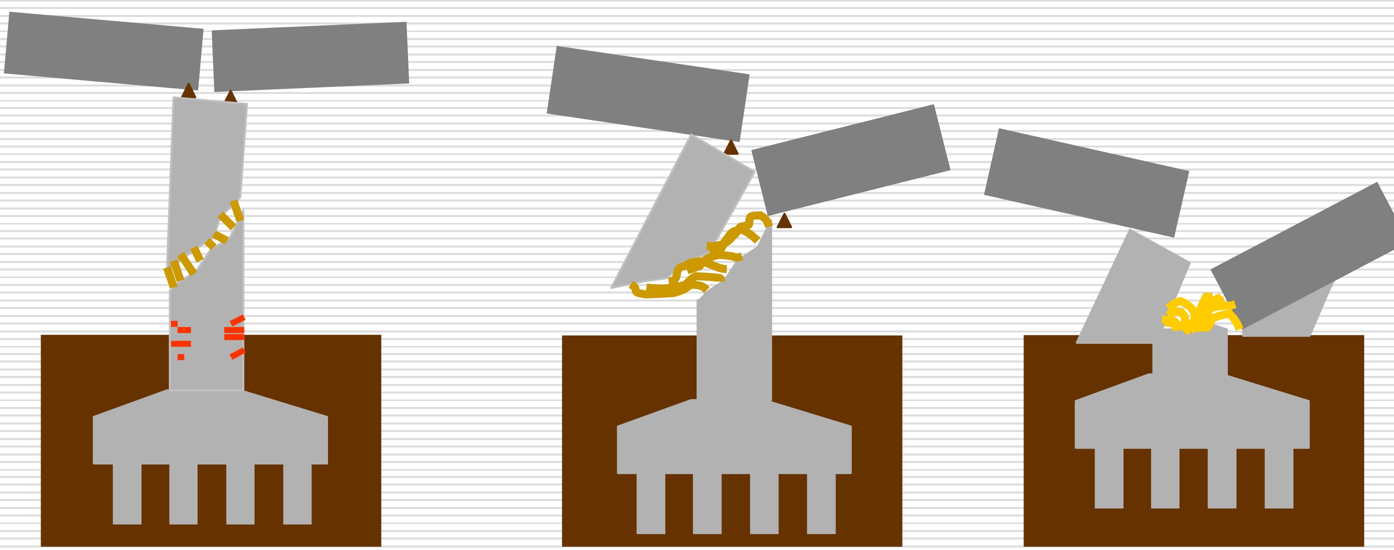
The importance of lateral confinement was not known (コンクリートの横拘束の重要性が知られていなかった)

Takashio Viaduct
高潮高架橋

Failure Mechanism of Takashio Viaduct

推定された被害メカニズム





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Collapse of a 18-span viaduct in the 1995 Kobe Earthquake

NHK

Enhancement of Ductility Capacity
じん性率の向上

Prior to 1980 After 1995 Kobe earthquake