[11-1-1]

# Deteriorations of Road Infrastructures

**Institute of Industrial Science**
**The University of Tokyo**
**Kohei NAGAI**

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## Minneapolis I-35W bridge

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## "America in Ruins"

Silver Bridge (1967)

- 46 killed
- 9 injured

Mianus River Bridge (1983)

- 90,000 of traffic/day
- 3 killed
- 5 injured

Minneapolis I-35W bridge (2007)

- 13 killed
- 145 injured

Monitored

Many bridges in the US built in New Deal in 1930s were damaged for inappropriate maintenance.

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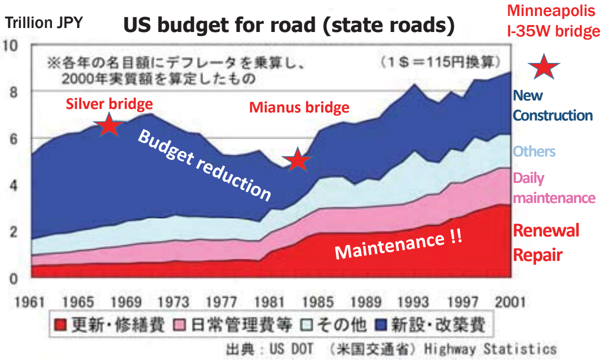
## Background - America in Ruins



出典：US DOT （米国交通省）Highway Statistics

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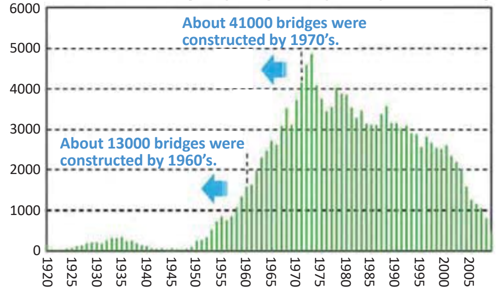
## Aging of Infrastructures in Japan

Number of construction (bridges, span length longer than 15 m)



*By Ministry of Land, Infrastructure, Transport and Tourism of Japan*

Right now, number of aged structures (over 50 years old) is rapidly increasing.

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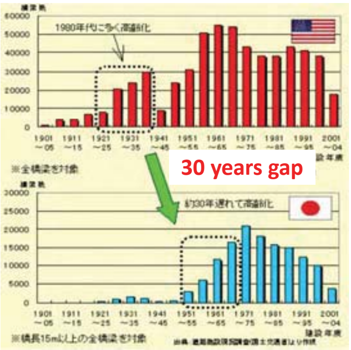
## Japan in ruins?



Will Japanese bridges also collapse?
Maintenance becomes important.

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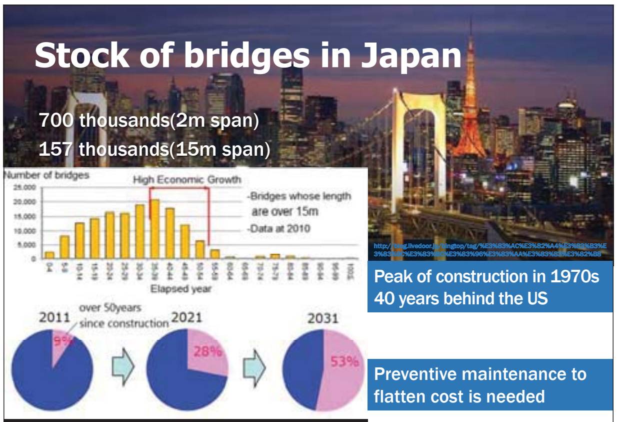
## Stock of bridges in Japan



700 thousands(2m span)
157 thousands(15m span)

http://blog.livedoor.jp/bingtop/tag/%E3%83%AC%E3%82%A4%E3%83%B3%E3%83%9C%E3%83%BC%E3%83%96%E3%83%AA%E3%83%83%E3%82%B8

Peak of construction in 1970s
40 years behind the US

Preventive maintenance to flatten cost is needed

The number of bridges over 50 years old is increasing rapidly in two decades

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## Examples of damage and repair

- ･Corrosion
- ･Fatigue
- ･Replacement (shoe, slab)
- ･Fire and so on

# Corrosion

-damage-

-repair-

At bridge end

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-bridge Full view-

- State of the outer surface of the girder -

-Corrosion state of the girder inner surface-

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# Corrosion

-damage-

-repair-

Fillet welding between web and lower flange

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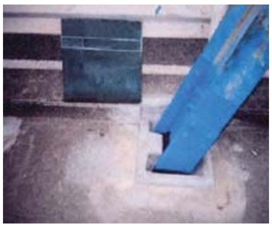
# Corrosion

Breakage of diagonal Member in truss bridge

-damage-



-repair-

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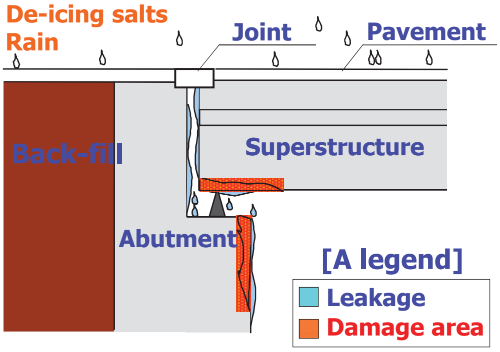
# Water Leakage From Expansion Joint



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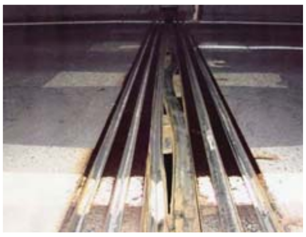
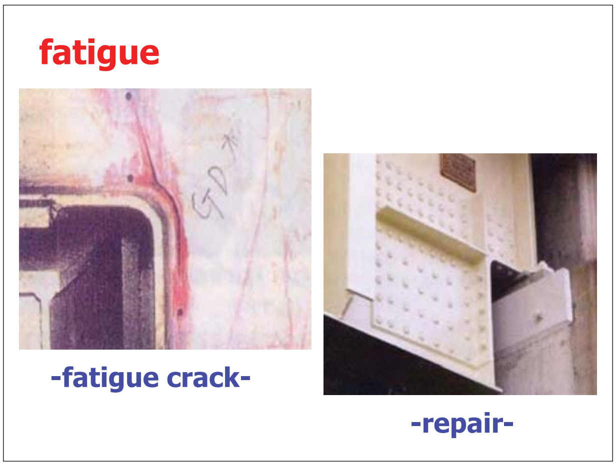
# Damage of Expansion Joint





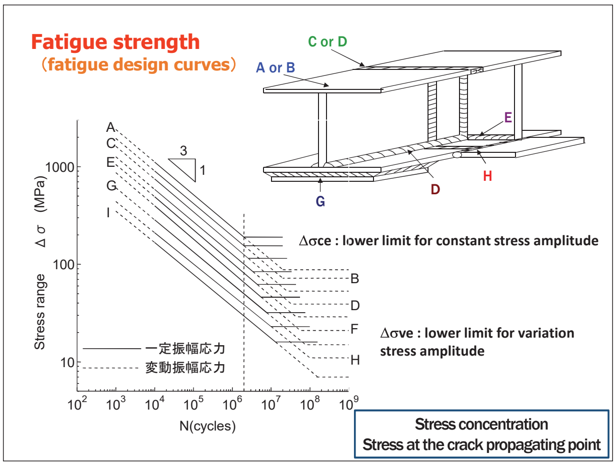
## fatigue

-fatigue crack-

-repair-

2006年10月2日発見、名阪国道「山添橋」

中央線第2平等川橋梁（山梨県石和町）2004年



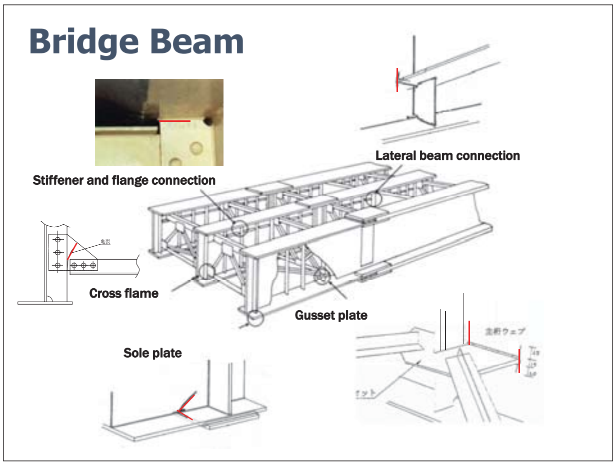
## Fatigue strength

（fatigue design curves）

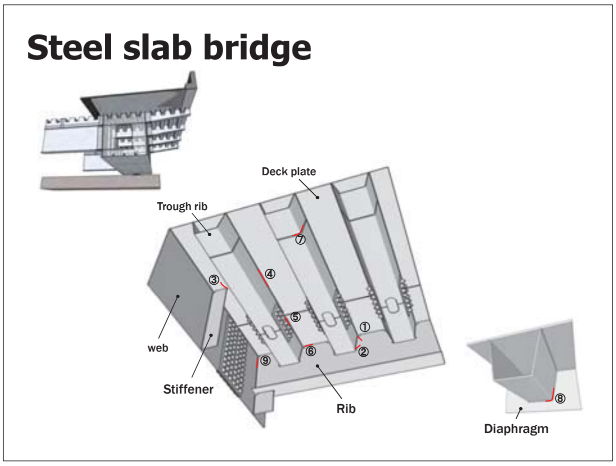
Δσce : lower limit for constant stress amplitude

Δσve : lower limit for variation stress amplitude

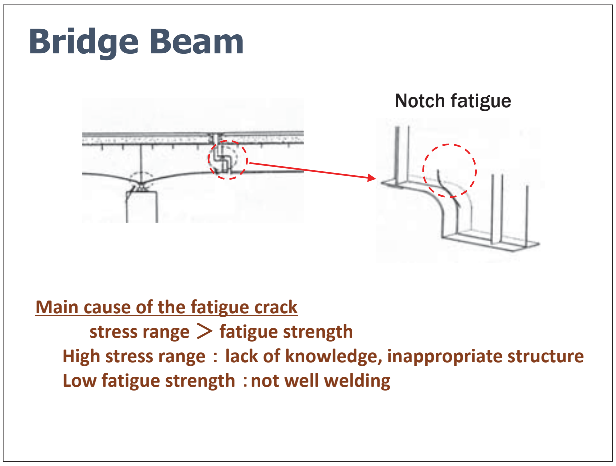
Stress concentration
Stress at the crack propagating point



## Bridge Beam



## Steel slab bridge



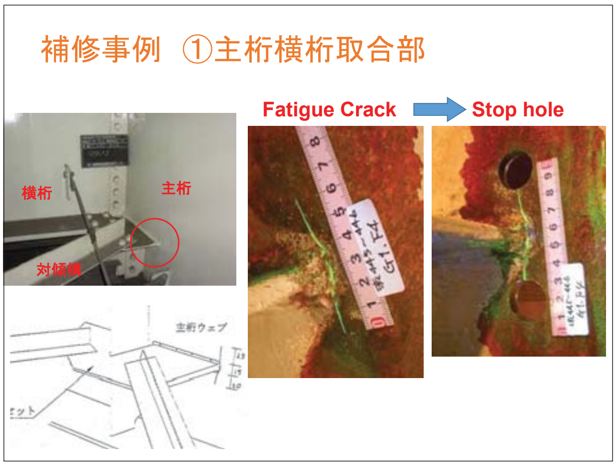
## Bridge Beam

Main cause of the fatigue crack

stress range ＞ fatigue strength

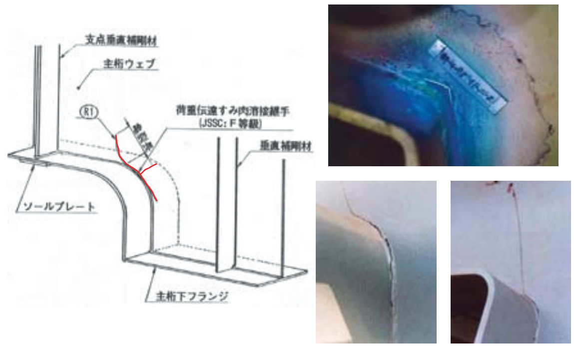
High stress range： lack of knowledge, inappropriate structure

Low fatigue strength ：not well welding



## 補修事例　①主桁横桁取合部

## ②主桁の桁端切欠き部



**Cause ：stress concentration, welding condition**

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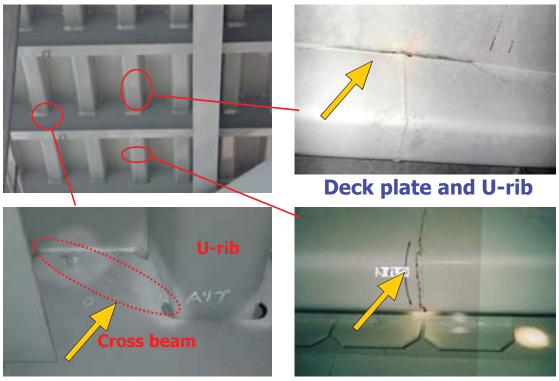
## fatigue　Steel plate deck



Deck plate and U-rib

Cross of cross beam and U-rib

Welding of U-rib

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## fatigue

-fatigue crack-

-repair-

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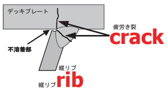

strengthening by（SFRC pavement）

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## fatigue

Repair by steel plate (bolting)
Rigid frame type steel piers

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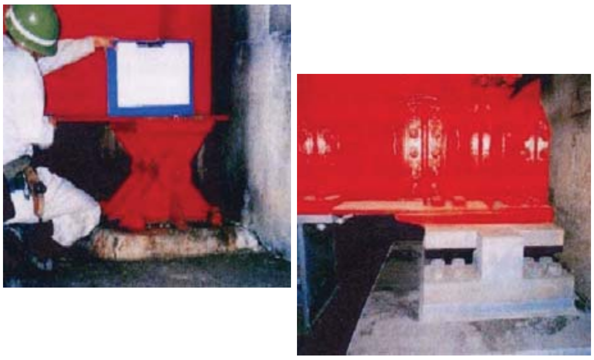
## Replacement of shoe (metal to rubber)



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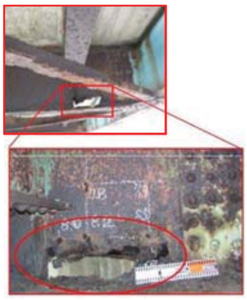

著しく腐食し支承としての機能を失っている

著しく腐食し支承としての機能が損なわれている

## 橋梁の劣化

（主に国交省HP）

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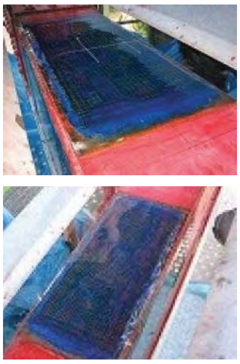
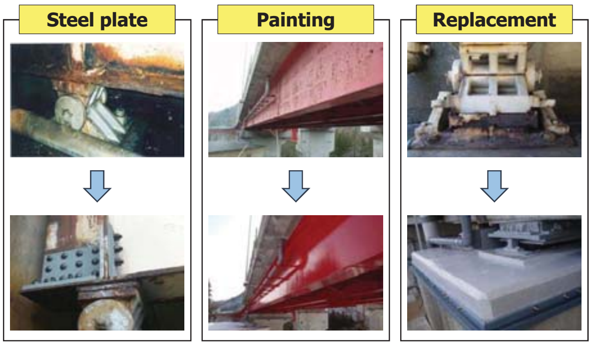
Asari river bridge
(Central-Nippon Expressway)

## CFRP bonding



## Repair work for steel structures



## Can we use or stop??

## Damages of concrete structures

Rupture of cables

# Classification of Deteriorations

- Steel structure
  - Corrosion
  - Fatigue
- Concrete structure
  - Corrosion
  - Alkali-silica reaction
  - Drying shrinkage
  - Frost damage
  - Sulfate attack
  - Creep
  - Fatigue
  - Abrasion

*Green: deteriorations caused by environmental actions
*Red: deteriorations caused by mechanical actions

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# Corrosion mainly-induced damages

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# CAESAR's Clinical Research Cases

CAESAR (Center for Advanced Engineering Structural Assessment and Research), Public Works Research Institute (Incorporated Administrative Agency) in Japan conducts clinical research on infrastructure management.

Some representative cases are introduced in this presentation.

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# Kuratanibashi Bridge

## Bridge specifications

| | |
|---|---|
| Route and location | City road, Gotsu City, Shimane Prefecture |
| Bridge type | RC slab bridge |
| Bridge length | 10.2 m |
| Span length | 5.1 m (x2) |
| Year of completion | 1959 (51 years elapsed) |

Kuratanibashi bridge

## Observed damage (bottom of the slab)

Delamination of concrete

Significant corrosion of main rebars

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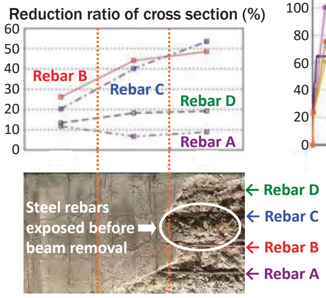
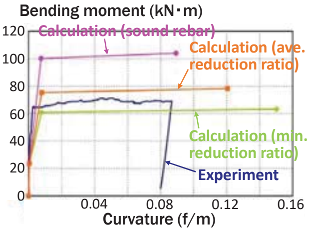
# Kuratanibashi Bridge

Correlation between average reduction ratio of cross section of rebar and slab undersurface



Result of the loading test



- ✓ Delamination affects the corrosion.
- ✓ About 32% reduction of load capacity is expected.

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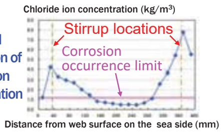
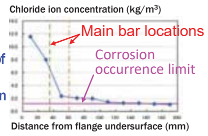
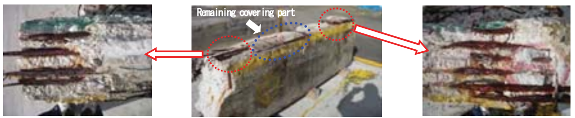
# Tsuhabashi Bridge

Horizontal distribution of chloride ion concentration



Vertical distribution of chloride ion concentration



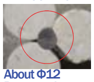


Exposed rebars / Removed for dismantling investigation / Rebars of the support part

State of rebar before and after derusting

About Φ25
Relatively sound rebar

About Φ18
Decrease in cross section: 49%



About Φ12
Decrease in cross section: 77%

✓ Remarkable $Cl^-$ ingress and corrosion resulted bridge fell down.

---

# Tsuhabashi Bridge

## Bridge specifications

| | |
|---|---|
| Route and location | Old National Rounte 58, Ogimi Village, Okinawa Prefecture |
| Bridge type | RCT girder bridge |
| Bridge length | 10.2 m |
| Span length | 9.4 m |
| Year of completion | 1931 (79 years elapsed) |

Tsuhabashi bridge

This bridge fell down due to the exposure to the environment subjected to the salt damage for a long period.

Before the bridge fell down

After the bridge fell down

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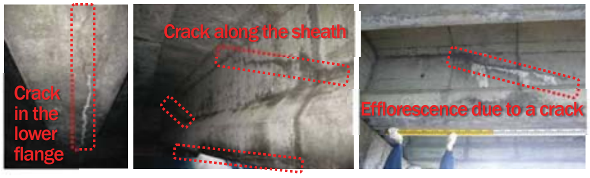
# Uenaibashi Bridge

## Bridge specifications

| | |
|---|---|
| Route and location | Uenai Station Road, Tomakomai City, Hokkaido Prefecture |
| Bridge type | Simple post-tensioned PCT girder bridge |
| Bridge length | 38.0 m |
| Span length | 18.3 m (x2) |
| Year of completion | 1961 (49 years elapsed) |

Uenaibashi bridge

## Damage observed from the outer side



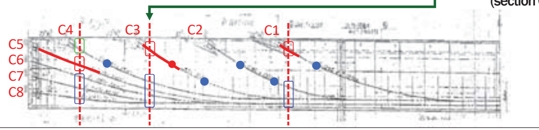
## Uenaibashi Bridge

### Grout filling survey by cutting and core collection


Cutting by a dry wire saw


Dry core collection

State of grout filling on the cross section (section C3)



- - - - : Cutting point
● : Sufficiently filled core
● : Insufficiently filled core
☐ : Cross section of grout filled portion
☐ : Cross section with insufficient filling
☐ : Unknown cross section
▬ : Cable with insufficient filling

✓ Grout filling is insufficient near the upper anchorage part.

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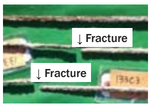
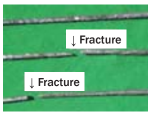
## Nou-Ohashi Bridge

The girder was removed and damage was investigated.



Before derusting (G3-C3)



After derusting (G3-C3)

✓ Remarkable damage including rupture of PC cables was observed.

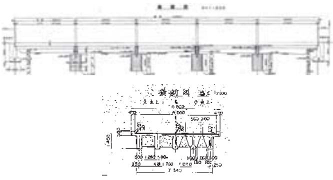
---

## Nou-Ohashi Bridge

### Bridge specifications

| | |
|---|---|
| Route and location | National highway 8, Itoigawa City, Niigata Prefecture |
| Bridge type | Simple PC girder bridge |
| Bridge length | 141.0 m |
| Span length | 28.1 m (x5) |
| Year of completion | 1966 (44 years elapsed) |


Nou-Ohashi bridge



General view
(upper: side view, lower: cross section)


Cracking in the girder

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## Osahashi Bridge

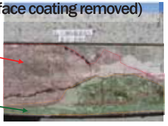
### Bridge specifications

| | |
|---|---|
| Route and location | National highway 352, Kashiwazaki City, Niigata Prefecture |
| Bridge type | Simple RCT girder bridge |
| Bridge length | 8.56 m |
| Span length | 8.1 m |
| Year of completion | 1965 (45 years elapsed) |


Osahashi bridge

### Observed damage

(After surface coating removed)



✓ Deteriorated again despite the repair and strengthening
✓ Chloride ion level at the location of steel rebar: 5.7 kg/m$^3$

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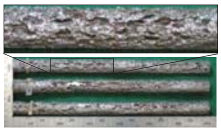
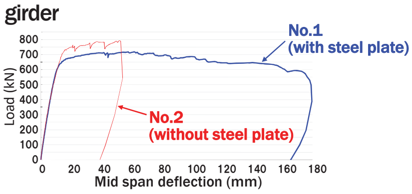
## Osahashi Bridge

### Dismantling survey of the floating part



### Loading test of the girder



✓ Less effect of the steel plate on load capacity

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## ASR

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## ASR mainly-induced damages

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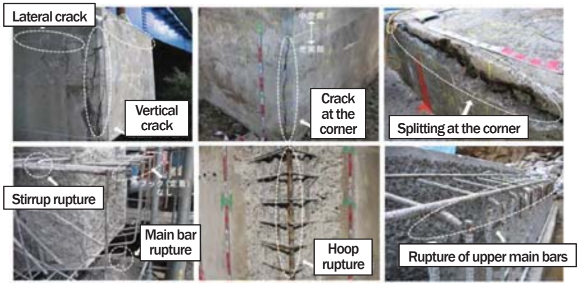
## ASR

（網目状のひび割れ）

# Structures Damaged by ASR

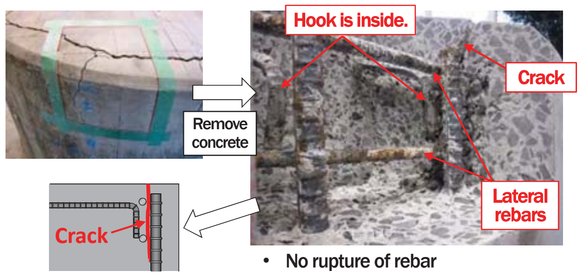
Examples of damage by ASR (Alkali Silica Reaction)



T. Okumura, T. Minato and T. Sakai, "Detailed survey of bridges damaged by ASR in Ishikawa prefecture and countermeasures", The 1st workshop on bridge maintenance in Hokuriku region, A-12. (in Japanese)

✓ ASR induces rather random shape crack pattern.
✓ Rebar rupture frequently occurs at the bent portion.

# Structures Damaged by ASR



Schematic of rebar detailing and crack

- No rupture of rebar
- Hook of rebars is inappropriately provided at the inside of lateral bars.

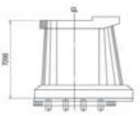
✓ Inappropriate structural detailing can also affect the progress of deteriorations.

# Structures Damaged by ASR

Bridge specifications

| | |
|---|---|
| Bridge name | NA |
| Bridge type | Simple steel-composite box girder bridge |
| Span length | 51.6 m (x5) |
| Year of completion | 1964 (46 years elapsed) |

Complete view of the bridge



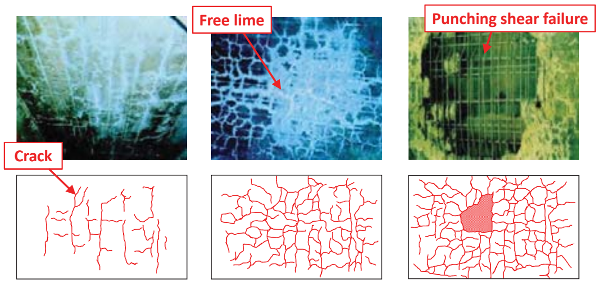
P1 pier

Damage by ASR (Alkali Silica Reaction)

Coring sample of concrete

# Fatigue mainly-induced damages

# Process of Fatigue Failure in Slabs



Two-directional cracks occur.

Progress of the two-directional cracks and occurrence of free lime and water leakage.

After the increase of the water leakage, punching shear failure occurs.

# Disaggregation of Concrete

Before pavement removed

After pavement removed

Closeup pictures of the inside concrete

*By East Nippon Expressway Co., Ltd.*

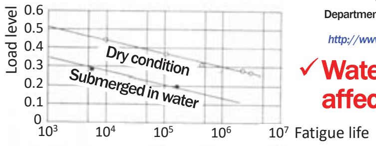
✓ Concrete was disaggregated in the severe environment.

# Effect of Water on Fatigue

Wheel-trucking fatigue loading test

Failure (upper surface **submerged in water**)

Website of Concrete Engineering Laboratory,
Department of Civil Engineering, Faculty of Engineering,
Nihon University.
*http://www.civil.ce.nihon-u.ac.jp/~concrete/index.html*



*By Matsui et al.*

✓ Water significantly affects the fatigue life.

## Repair for concrete structures

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## Situations in Other Asian Countries (1)

Bridge type: RC structures with span length around 40 m (approaching part).
Location: Rayong Province, Thailand
Age: 7 to 10 years
Environmental condition: Around 500 m away from the sea but sea water comes to the bottom of the structure.


Severe corrosion cracks along the reinforcement in the intermediate beams

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## Situations in Other Asian Countries (1)

Bridge type: RC structures with span length around 40 m (approaching part).
Location: Rayong Province, Thailand
Age: 7 to 10 years
Environmental condition: Around 500 m away from the sea but sea water comes to the bottom of the structure.

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## Situations in Other Asian Countries (1)

Bridge type: RC structures with span length around 40 m (approaching part).
Location: Rayong Province, Thailand
Age: 7 to 10 years
Environmental condition: Around 500 m away from the sea but sea water comes to the bottom of the structure.

Strange shape observed in the columns

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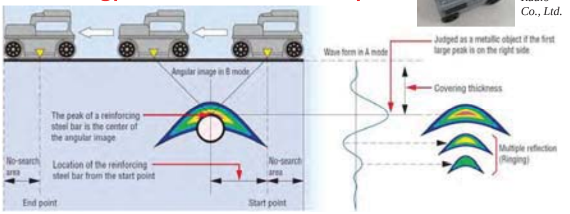
## Conducted Non-destructive Tests

Ground Penetrating Radar (GPR)
*For measuring position of rebars and cover depth*


*NJJ-105 Japan Radio Co., Ltd.*



Radiate electromagnetic waves through concrete and receive reflected signals

Air Permeability Test

Electric Resistivity Test

*For evaluating surface concrete quality*

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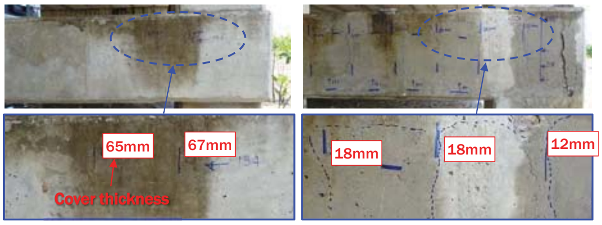
## Situations in Other Asian Countries (1)

Bridge type: RC structures with span length around 40 m (approaching part).
Location: Rayong Province, Thailand
Age: 7 to 10 years
Environmental condition: Around 500 m away from the sea but sea water comes to the bottom of the structure.



Significant scattering in cover thickness. Corrosion occurs in thinner cover condition.

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By moving GPR in the direction of member axis, positions and depth of stirrups can be measured.

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## Situations in Other Asian Countries (2)

Severe corrosion of concrete bridges in Rakhine, Myanmar

Bridge investigation was conducted on 8th and 9th September, 2015.
*Only 10 years passed after the bridge completion.

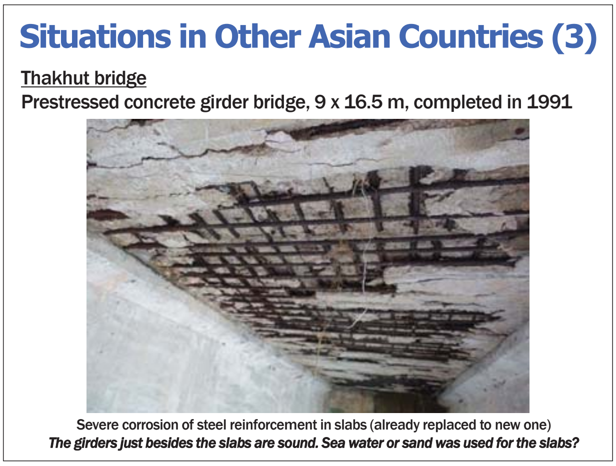

Corrosion of steel reinforcement in the pier


Corrosion of tensile reinforcement in the girder


Corrosion of stirrups (shear reinforcement ) in the girder


Re-deterioration after the patch repair


Corrosion of RC slabs and repaired portions



## Situations in Other Asian Countries (3)

<u>Thakhut bridge</u>
Prestressed concrete girder bridge, 9 x 16.5 m, completed in 1991

Severe corrosion of steel reinforcement in slabs (already replaced to new one)
***The girders just besides the slabs are sound. Sea water or sand was used for the slabs?***

## Situations in Other Asian Countries (3)

<u>Thakhut bridge</u>
Prestressed concrete girder bridge, 9 x 16.5 m, completed in 1991

Problem 1: Corrosion of slabs
Problem 2: Movement of piers (explained in the next presentation)

## Situations in Other Asian Countries (3)

Repair work of the slabs, 11th Dec., 2016
***Mix proportion of concrete...???***

[11-1-2]

# Maintenance of Road Infrastructures

Institute of Industrial Science
The University of Tokyo
Kohei NAGAI

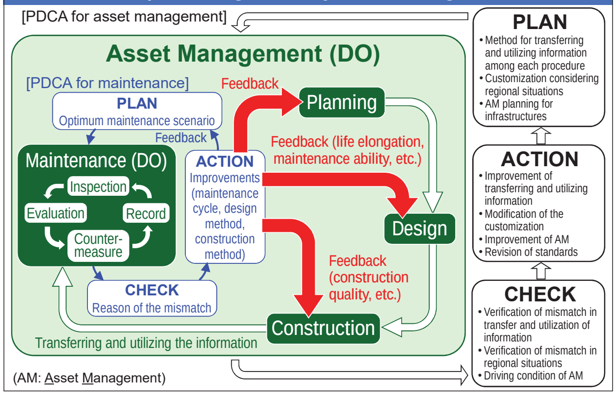
---

## Inspection (cyclic) &
## Diagnosis (correctly)

## are very important
## [key factors]

---

## Infrastructure Asset Management Incorporating Life Cycle Management



[PDCA for asset management]

**Asset Management (DO)**

[PDCA for maintenance]

**PLAN**
Optimum maintenance scenario

Feedback

**Maintenance (DO)**
Inspection
Record
Counter-measure
Evaluation

**ACTION**
Improvements (maintenance cycle, design method, construction method)

**CHECK**
Reason of the mismatch

Feedback

Planning

Feedback (life elongation, maintenance ability, etc.)

Design

Feedback (construction quality, etc.)

Construction

Transferring and utilizing the information

**PLAN**
- Method for transferring and utilizing information among each procedure
- Customization considering regional situations
- AM planning for infrastructures

**ACTION**
- Improvement of transferring and utilizing information
- Modification of the customization
- Improvement of AM
- Revision of standards

**CHECK**
- Verification of mismatch in transfer and utilization of information
- Verification of mismatch in regional situations
- Driving condition of AM

(AM: Asset Management)

---

## Inspection Procedure

1) Preliminary investigation for efficiency
(check drawings, former records and etc.)
↓
2) Make inspection plan
↓
3) (visual)inspection by access and record
↓
4) Diagnosis (Evaluation of performance)

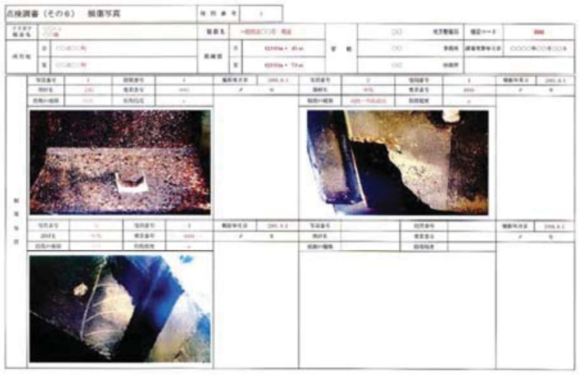
---

## Inspection Type

| Type | Aim |
|---|---|
| Initial | Grasp the structural damage just after completion or repair work |
| Daily | For user's safety (by patrol) |
| Regular | Grasp the structural damage |
| Special | Grasp the structural damage just after earthquake attack, typhoon attack, collision, fire and so on |
| Detail | Grasp the degree and cause of the damage |

---

## Record (example)



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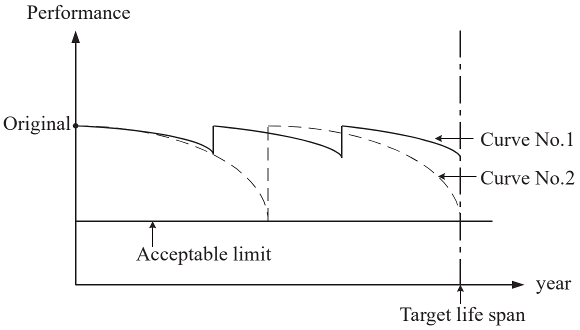
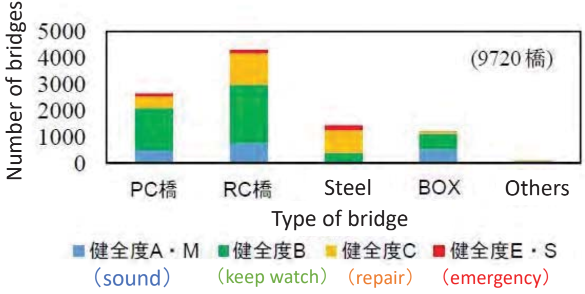
## Deterioration curve of performance



# Inspection data

---

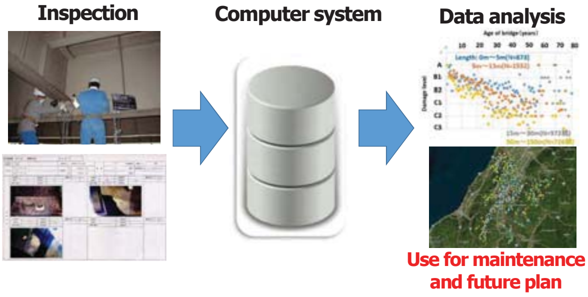
## Damage level



**Damage of steel structure is worse than concrete bridges.**

---

## Database is important for management

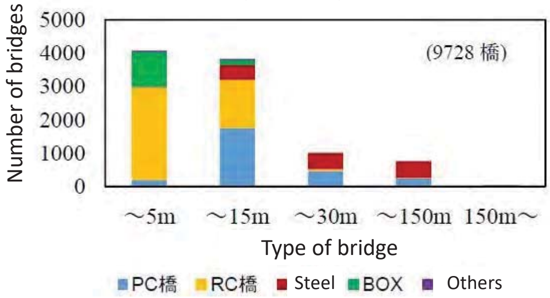


**Use for maintenance and future plan**

**Database is a basis for rational management of infrastructures**

---

## Bridge length

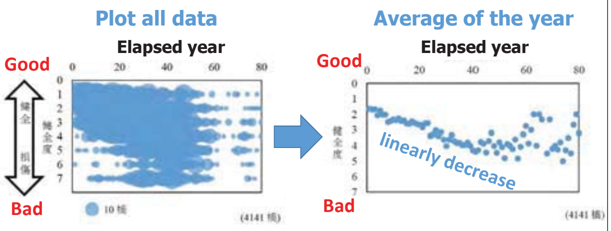


**Steel bridges are longer** (heavy traffic etc. ?)

**Difficult to judge, steel or concrete.**

---

## Damage by year

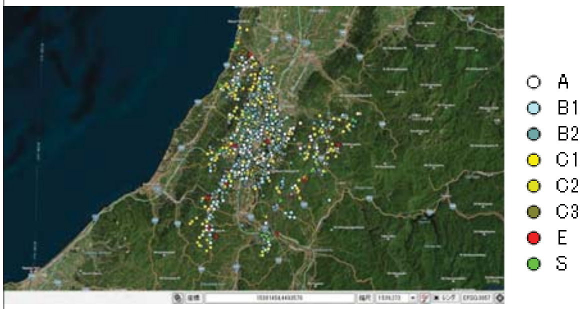


Damage is recovered after 50 years??

- less number of data
- Already repaired bridges are included
- old bridges are constructed carefully (no concrete mixer truck)

---

## Damage level



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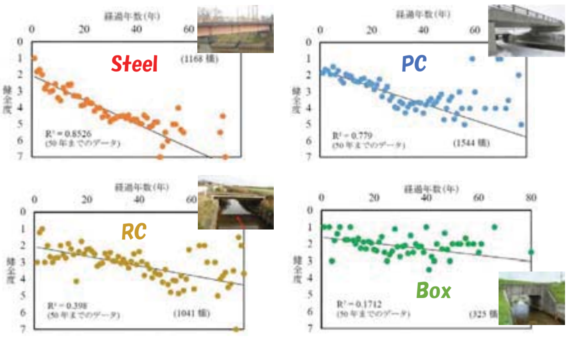
## Damage progress

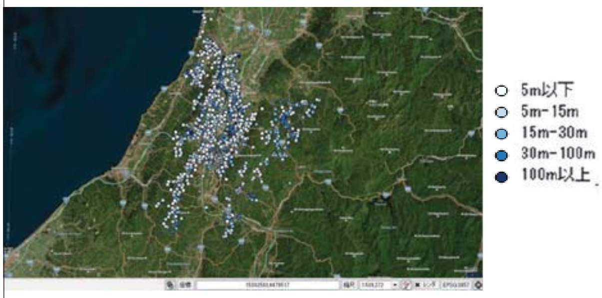


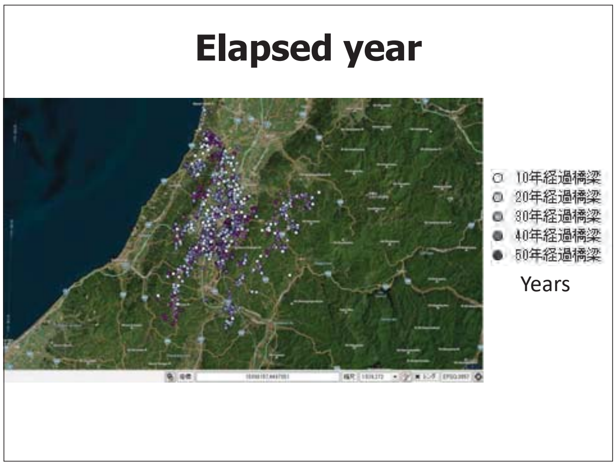
**Damage speed: Steel, PC, RC, Box**

**Steel structure is not maintained well (painting…)?**

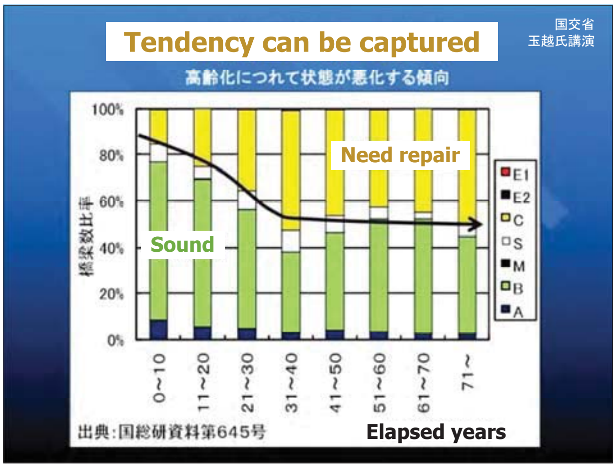
---

## Length of bridge





## Elapsed year



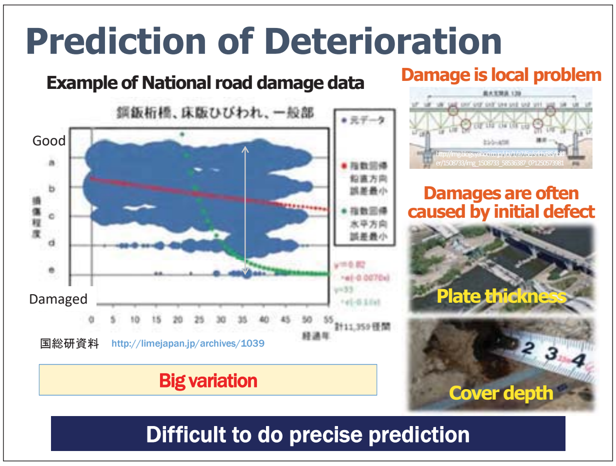
## Tendency can be captured

国交省
玉越氏講演

高齢化につれて状態が悪化する傾向

出典:国総研資料第645号

Elapsed years



## Prediction of Deterioration

**Example of National road damage data**

国総研資料 http://limejapan.jp/archives/1039

**Big variation**

**Damage is local problem**

**Damages are often caused by initial defect**

**Difficult to do precise prediction**

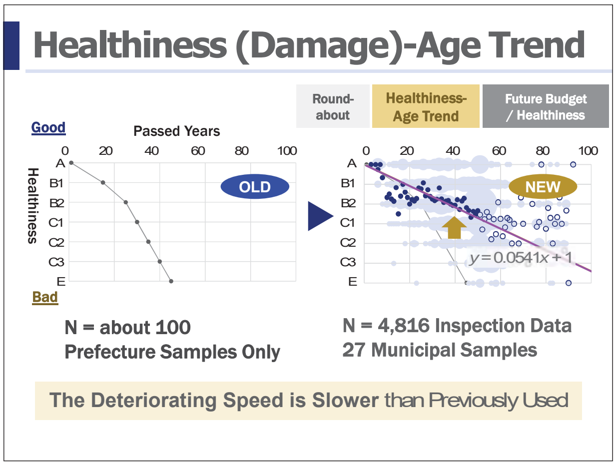
## Example of damage level estimation and budget estimation for the future

## In Niigata city, JAPAN

## How to estimate

1. **Bridge information (database)**
   Bridge type, length, damage level, construction year etc.
2. **Damage progress speed**
   Deterioration speed by bridge type etc.
3. **Repair timing and cost**
   Roughly set the repair cost for each type of bridge

Calculate conditions of all bridges on by one every year.
If repair is necessary, cost is counted,

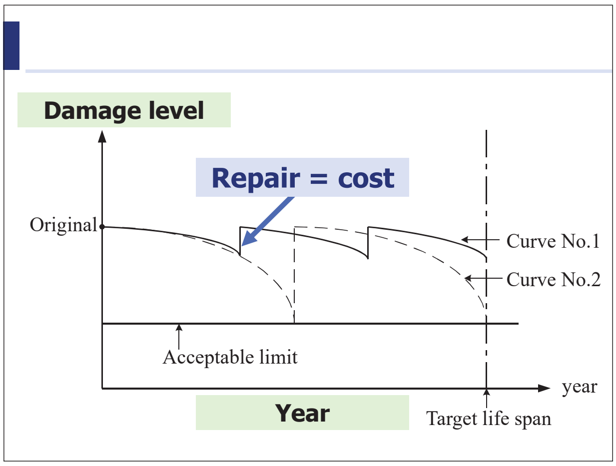


## Healthiness (Damage)-Age Trend

N = about 100
Prefecture Samples Only

N = 4,816 Inspection Data
27 Municipal Samples

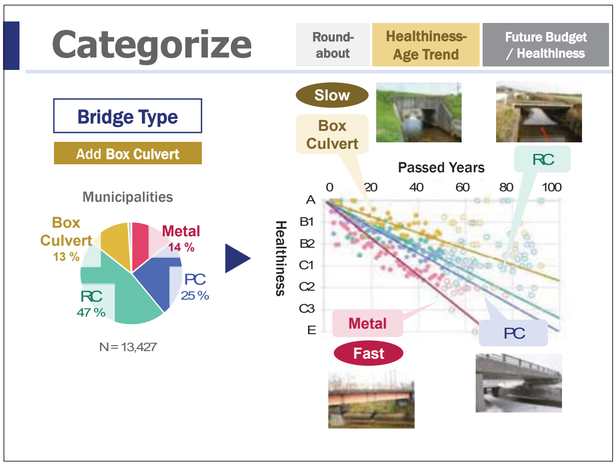
The Deteriorating Speed is Slower than Previously Used



## Categorize

Round-about | Healthiness-Age Trend | Future Budget / Healthiness

**Bridge Type** × **Length** × **Salt Environment**



## Categorize

Round-about | Healthiness-Age Trend | Future Budget / Healthiness

**Bridge Type**

**Add Box Culvert**

Municipalities

Box Culvert 13 %

Metal 14 %

PC 25 %

RC 47 %

N = 13,427

Slow

Box Culvert

RC

Passed Years

0 20 40 60 80 100

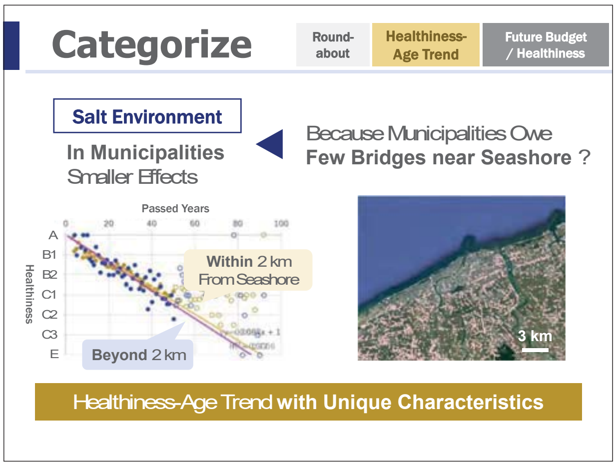
A B1 B2 C1 C2 C3 E

Healthiness

Metal

Fast

PC



## Categorize

Round-about | Healthiness-Age Trend | Future Budget / Healthiness

**Salt Environment**

In Municipalities
Smaller Effects

Because Municipalities Owe
Few Bridges near Seashore ?

Passed Years

0 20 40 60 80 100

A B1 B2 C1 C2 C3 E

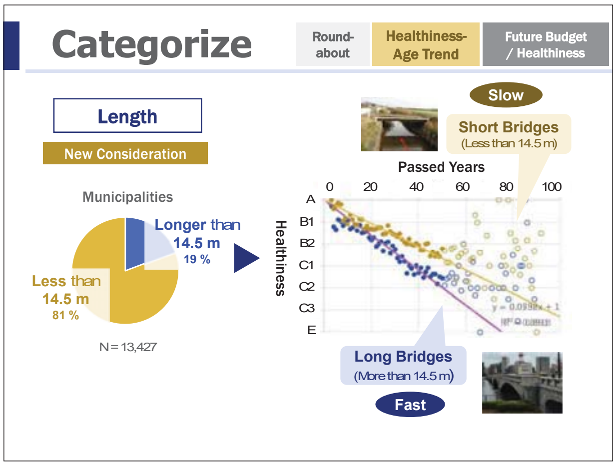
Healthiness

Within 2 km
From Seashore

Beyond 2 km

3 km

**Healthiness-Age Trend with Unique Characteristics**



## Categorize

Round-about | Healthiness-Age Trend | Future Budget / Healthiness

**Length**

**New Consideration**

Municipalities

Longer than 14.5 m 19 %

Less than 14.5 m 81 %

N = 13,427

Slow

Short Bridges
(Less than 14.5 m)

Passed Years

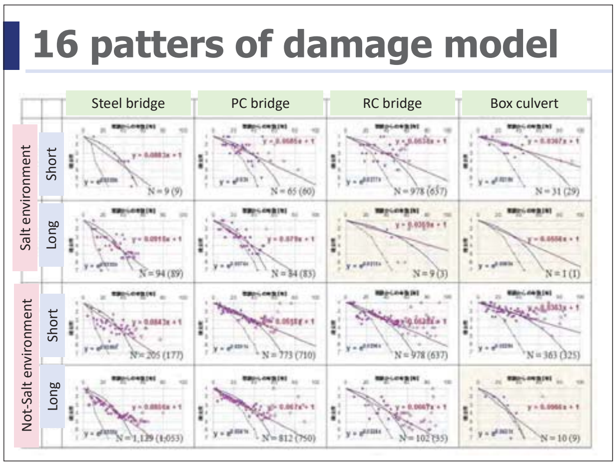
0 20 40 60 80 100

A B1 B2 C1 C2 C3 E

Healthiness

Long Bridges
(More than 14.5 m)

Fast

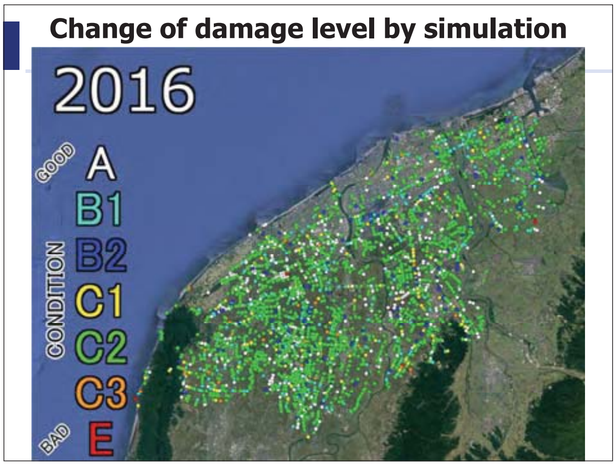


## 16 patters of damage model

Steel bridge | PC bridge | RC bridge | Box culvert

Salt environment: Short, Long

Not-Salt environment: Short, Long



## Change of damage level by simulation

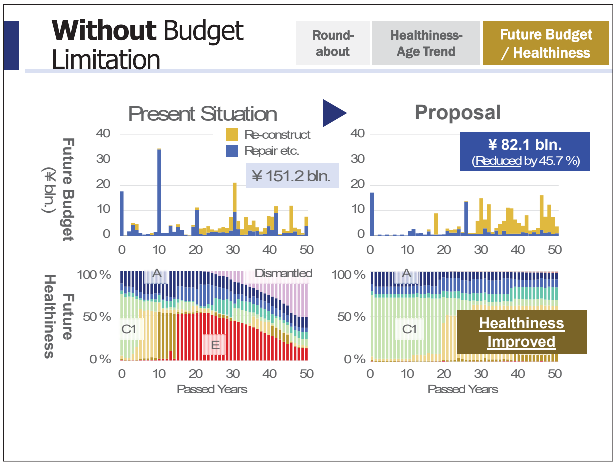
2016

GOOD

CONDITION

A B1 B2 C1 C2 C3 E

BAD



## Without Budget Limitation

Round-about | Healthiness-Age Trend | Future Budget / Healthiness

Present Situation

Proposal

Future Budget (¥ bln.)

Re-construct

Repair etc.

¥ 151.2 bln.

¥ 82.1 bln.
(Reduced by 45.7 %)

0 10 20 30 40

0 10 20 30 40 50

Future Healthiness

100 % 50 % 0 %

A

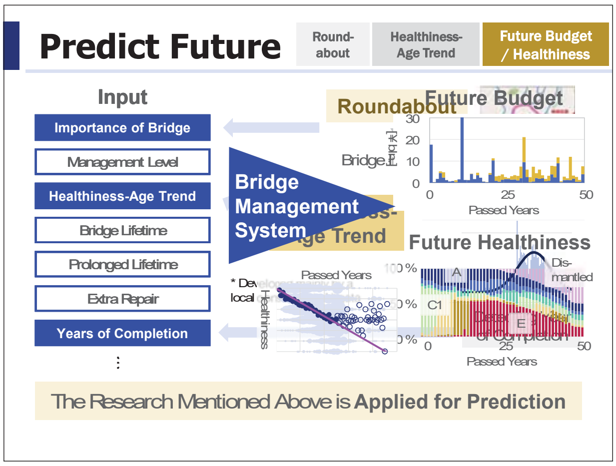
C1

E

Dismantled

Healthiness Improved

Passed Years



## Predict Future

Round-about | Healthiness-Age Trend | Future Budget / Healthiness

Input

- Importance of Bridge
- Management Level
- Healthiness-Age Trend
- Bridge Lifetime
- Prolonged Lifetime
- Extra Repair
- Years of Completion
- ⋮

Bridge Management System

Roundabout

Future Budget

Bridge

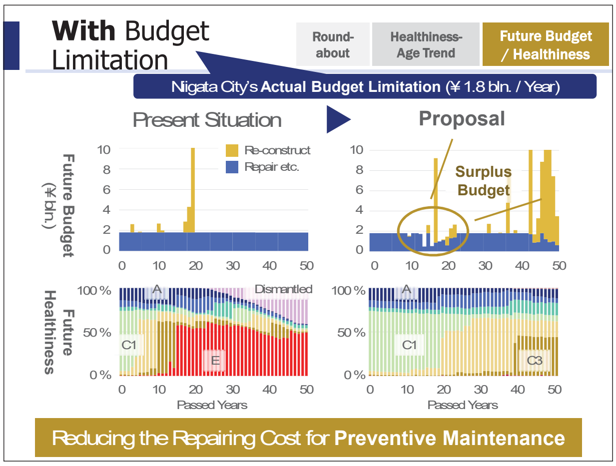
Healthiness-Age Trend

Future Healthiness

Passed Years

Dis-mantled

**The Research Mentioned Above is Applied for Prediction**



## With Budget Limitation

Round-about | Healthiness-Age Trend | Future Budget / Healthiness

Nigata City's Actual Budget Limitation (¥ 1.8 bln. / Year)

Present Situation

Proposal

Future Budget (¥ bln.)

Re-construct

Repair etc.

Surplus Budget

0 2 4 6 8 10

0 10 20 30 40 50

Future Healthiness

100 % 50 % 0 %

A

C1

E

C3

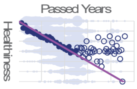
Dismantled

Passed Years

**Reducing the Repairing Cost for Preventive Maintenance**

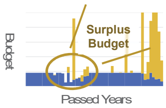
# Conclusions

To Propose **Application of Inspection Data / Spatial Information** so as to **Rationalize Bridge Managing**



**Roundabout Calculation**
for Judging Importance

**Healthiness-Age Trend**
Based on 4,816 Municipal Inspection Data



**Applied Bridge Inspection Data / Spatial Information**
for the Maintenance Plan of Niigata

▶ **Implemented Successfully**

Verified the Efficiency by **Budget / Healthiness Prediction**

# Application to Nigata-City

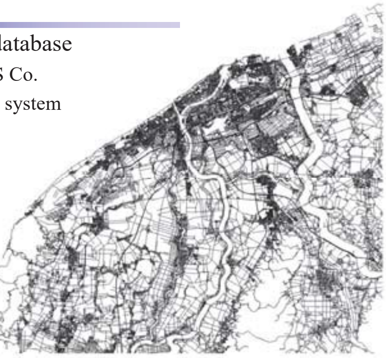
- Basic information of Nigata-city
  - One of prefectural capital
  - Population: 807,450
  - Area: 726.25km$^2$
  - Shinano-river, which is the longest in Japan
- About the bridges
  - Over 4000!
  - Nagai-sensei has very informative data about their condition :-)

# Example

Bridge data and Road network

# Road Network Data

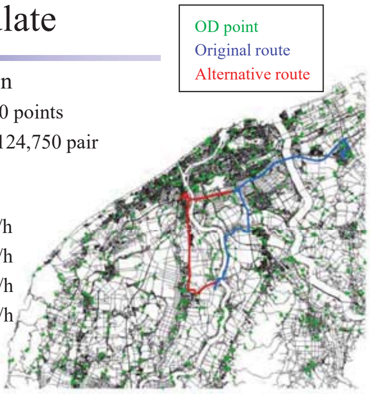
- Digital road map database
  - By Sumitomo ESS Co.
  - For car navigation system
- In the DB
  - Road class
  - Road width
  - # of lane, etc…
- NW Size
  - Node: 198,002
  - Link: 228,610



# How to Calculate

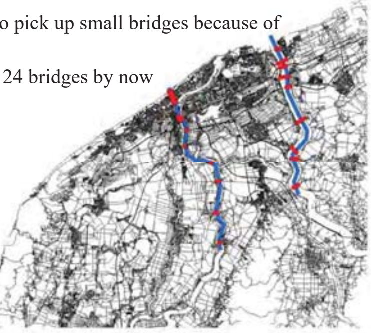
- Origin and destination
  - Randomly choose 500 points
  - Total (500×499)/2=124,750 pair
- Shortest time path

| | |
|---|---|
| Highways: | 80km/h |
| General roads: | 40km/h |
| Local roads: | 20km/h |
| others: | 10km/h |

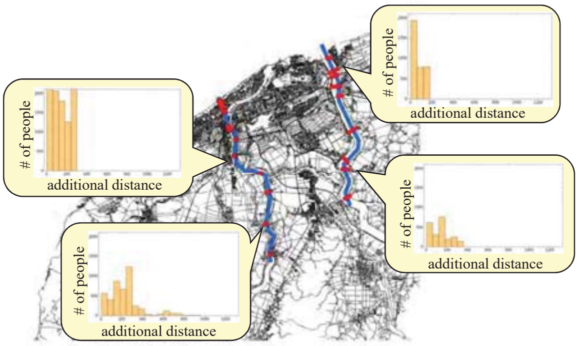


# Location of River and Bridge

- hand working to finalize…
  - Very difficult to pick up small bridges because of time limitation
  - Only analyzed 24 bridges by now



# Distribution of Additional Distance



## Example

## Open Data and Open Public

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2013 REPORT CARD for america's INFRASTRUCTURE ASCE

## BRIDGES


The average age of a bridge in the U.S. is 42 years old. Some bridges like this one in Virginia are beginning to show their age.

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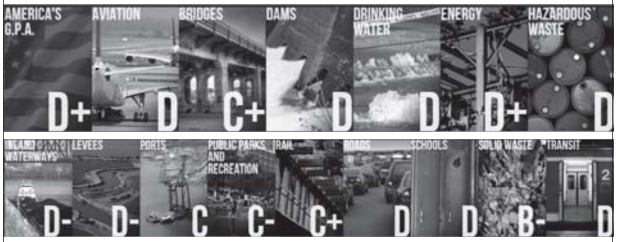
2013 REPORT CARD for america's INFRASTRUCTURE ASCE

## INFRASTRUCTURE REPORT 2013



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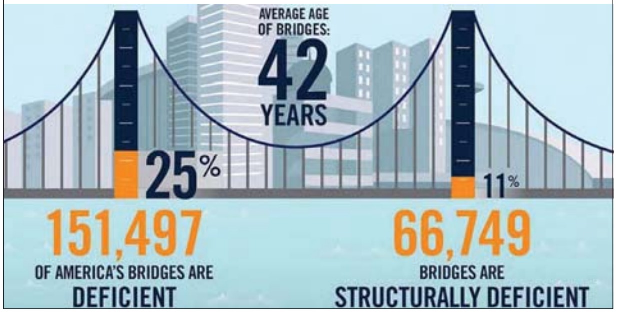
## *Overview:*

Click

- ☐ One in nine of the nation's bridges are rated as structurally deficient, while the average age of the nation's 607,380 bridges is currently 42 years
- ☐ estimates that to eliminate the nation's bridge deficient backlog by 2028
- ☐ need to invest $20.5 billion annually, while only $12.8 billion is being spent currently



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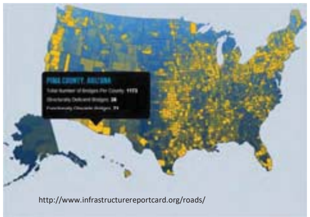
2013 REPORT CARD for america's INFRASTRUCTURE ASCE

RITA opens data


http://www.rita.dot.gov/bts/publications/pocket_guide_to_transportation/2013/transportation_system_and_equipment/figure_02_03

ASCE converts the data for public


http://www.infrastructurereportcard.org/roads/

Use for budget acquisition

---

## INVESTMENT IN ROADS, BRIDGES, TRANSIT IS NOT KEEPING UP WITH NEEDS:

Open data, Open public

[11-1-3]

# ICUS Projects
# for Infrastructure Management in Asia

## SIP Project

(supported by Cabinet office of Japan)

## SATREPS Project

(supported by Cabinet office of JICA)

**Kohei NAGAI**
Associate Professor
International Center for Urban Safety (ICUS)
The University of Tokyo

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## Cross-ministerial Strategic Innovation Promotion Program
by Cabinet Office of Japan since 2014

*To be the Top, Innovation !*
*Prime Minister Shinzo ABE*

**10 subjects are selected as significant and necessity challenges for society and growth of Japanese economy and industry.**
**(50 billion JPY= 400 million USD (2014))**

Council for Science and Technology Policy
(HP of Cabinet Office)

- Innovative Fuel Technology
- Future-generation Power Electronics
- Innovative Structural Material
- Energy Carrier
- Future-generation Marine Resource Survey Technology
- Auto Driving System
- Infrastructure Maintenance / Renewal / Management Technology
- Resilient Capability Development for Disaster Prevention and Reduction
- Future-generation Innovative Technology for Agriculture, Forestry and Fisheries
- Innovative Production Design Technology

---

## SIP Project (by Cabinet Office of Japan)
Cross-ministerial Strategic Innovation Promotion Program

### Research Project Title:
**Comprehensive research on development of road infrastructure management cycle and its application in Japan and abroad**

Project term: October 2014 – March 2019

Research budget: Approx. 150 million JPY = 1.25 million USD / Year
Head of the project:
Prof. Koichi Maekawa (University of Tokyo)

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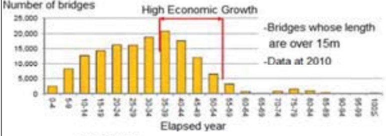
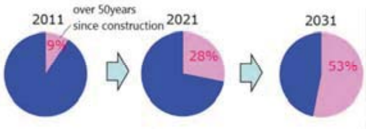
## Bridges in JAPAN

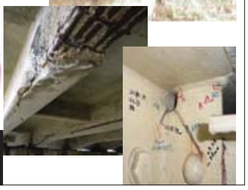
Pictures from MLIT HP etc.

**700,000 over 2m**
**157,000 over 15m**









Efficient and rational maintenance of infrastructure must be done immediately

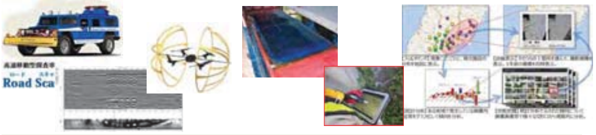
---

## Infrastructure Maintenance/Renewal/Management Technology

Project Director: Prof Yozo FUJINO
(Yokohama National University)

### Set Target Fields in this Subject

1. Inspection/Monitoring/Diagnosis Technology
2. Structural Material/Damage Progress/Retrofit/Repair Technology
3. Information and Communication Technology (ICT)
4. Robot Technology
5. Asset Management Technology ⇦ our project



**What is the innovation in our field in the project?**
**Prof. Fujino says *'Implementation is the innovation in this project'***

---

## Sub Project 4:
## Strategic International Expansion of Asset Management

### Establishment of international hub of asset management

**IIS, U Tokyo (Japan)**
International Center for Urban Safety Engineering（ICUS）
- Collection and transmission of information
- Development of suitable management technology and system

**SIIT, Thammasat U (Thailand)**
Construction and Maintenance Technology Research Center (CONTEC)
Prof. Somnuk Tangtemsirikul
New Partner: Chulalongkorn U

**Implement Japanese technology and system for management of infrastructure assets**

Collaborative institutes
- Tokyo Metropolitan Expressway Company
- JSCE
- Public Works Research Institute, Port and Airport Research Institute

**Network in Asian Region**
- Vietnam: NUCE
- Indonesia
- Myanmar: SATREPS Project
- India: IIT
- Int'l Association: ACF,EASEC

### Activities:
Collection and transmission of information of asset management
Evaluation of infrastructure development and management system in Asian countries
Development and implementation of asset management technology and system
International standardization (ISO/TC71)

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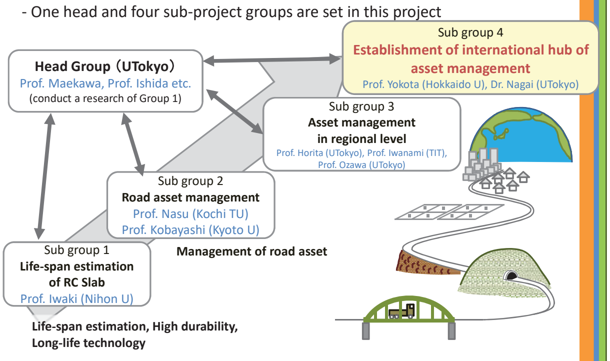
## Overall Research Scheme

- PDCA cycles of maintenance in the levels of Bridges, road, region.
- One head and four sub-project groups are set in this project



**Head Group（UTokyo）**
Prof. Maekawa, Prof. Ishida etc.
(conduct a research of Group 1)

Sub group 4
**Establishment of international hub of asset management**
Prof. Yokota (Hokkaido U), Dr. Nagai (UTokyo)

Sub group 3
**Asset management in regional level**
Prof. Horita (UTokyo), Prof. Iwanami (TIT), Prof. Ozawa (UTokyo)

Sub group 2
**Road asset management**
Prof. Nasu (Kochi TU)
Prof. Kobayashi (Kyoto U)

**Management of road asset**

Sub group 1
**Life-span estimation of RC Slab**
Prof. Iwaki (Nihon U)

**Life-span estimation, High durability, Long-life technology**

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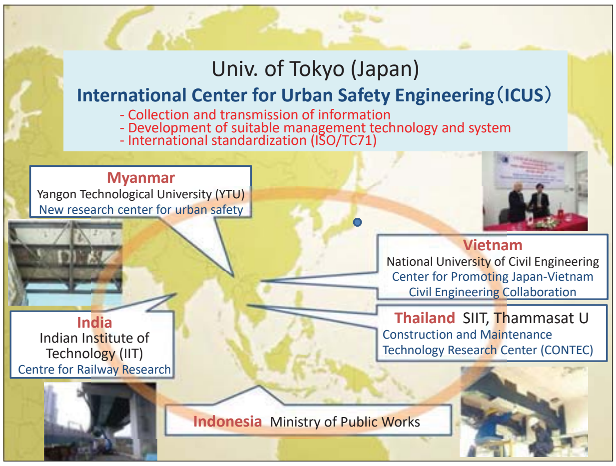


## Univ. of Tokyo (Japan)
### International Center for Urban Safety Engineering（ICUS）
- Collection and transmission of information
- Development of suitable management technology and system
- International standardization (ISO/TC71)

**Myanmar**
Yangon Technological University (YTU)
New research center for urban safety

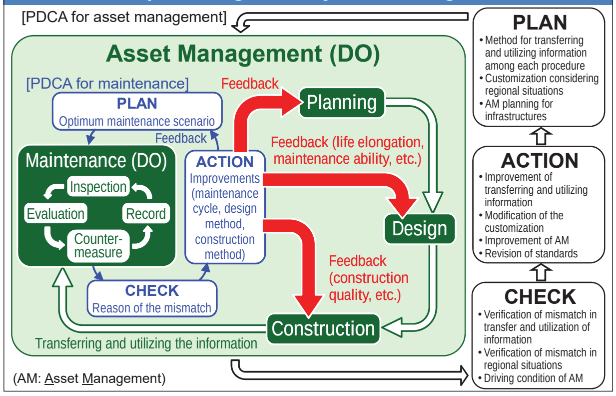
**Vietnam**
National University of Civil Engineering
Center for Promoting Japan-Vietnam Civil Engineering Collaboration

**India**
Indian Institute of Technology (IIT)
Centre for Railway Research

**Thailand** SIIT, Thammasat U
Construction and Maintenance Technology Research Center (CONTEC)

**Indonesia** Ministry of Public Works

# Infrastructure Asset Management Incorporating Life Cycle Management



**PLAN**
- Method for transferring and utilizing information among each procedure
- Customization considering regional situations
- AM planning for infrastructures

**ACTION**
- Improvement of transferring and utilizing information
- Modification of the customization
- Improvement of AM
- Revision of standards

**CHECK**
- Verification of mismatch in transfer and utilization of information
- Verification of mismatch in regional situations
- Driving condition of AM

(AM: Asset Management)

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SATREPS: JICA-JST Fund Research Project (FY2015-2019)

# Development of a Comprehensive Disaster Resilience and Collaboration Platform in Myanmar

Not technical cooperation, but research based project

Principal institutions: University of Tokyo, JAPAN
Yangon Technological University, MYANMAR

ANZEN SATREPS : MYANMAR NIPPON ZERO-CASUALTY ENGINEERING NETWORK 2014-2020

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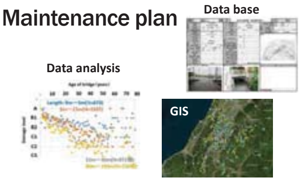
# Target area of the Project

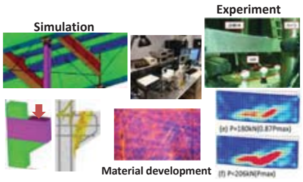
## Inspection
(Existing & New)

## Maintenance plan



## Repair

## Relating leading research



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# Project Office in YTU

Research Centre for Urban Safety

Meeting rooms

Equipment for measurement etc.

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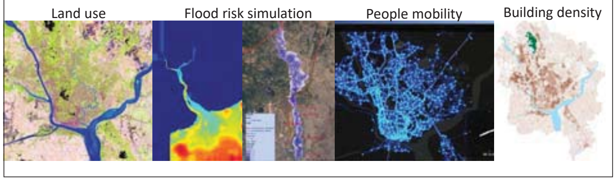
# Myanmar's steady economic growth by safe cities formation

## Hardware, Software, Human resources

### Project Groups

1. Water-related Disaster Group
2. Traffic / People mobility Group
3. Geo-spatial Technology Group
3. Earthquake-related Disaster Group
4. Infrastructure Management Group
5. Disaster Management Group



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# Way to the site (Last week)

Road infrastructure is important !

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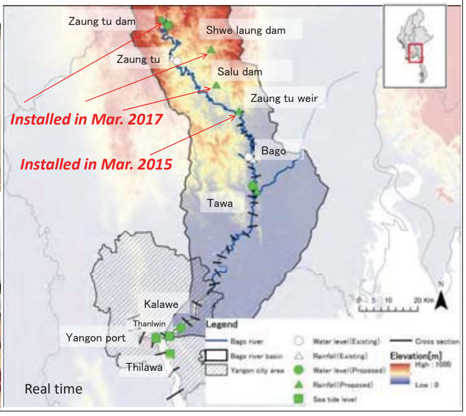
# Weather and hydrological observation site after SATREPS



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# Infrastructure Management Group / Members

## JAPAN

### Univ. of Tokyo

Dr. Kohei Nagai
Prof. Reiko Kuwano
Dr. Koji Matsumoto
Dr. Tsukasa Mizutani
Dr. Hiroshi Dobashi (Metropolitan Expressway)

### Hokkaido Univ.

Prof. Hiroshi Yokota Dr. Michael Henry

### Nagaoka Univ. of Tech.

Prof Eiji Iwasaki, Dr. Takeshi Miyashita

4 students from Japanese universities

## MYANMAR

### Yangon Technological University (YTU)

Prof. Khin Than Yu, YTU
Civil Engineering

### Ministry of Construction (MOC)

Daw Thein Nu, MOC
U Paing, MOC
U Tin Maung Htwe, MOC
U Myint Zaw, MOC
U Nyan Thar, YCDC

2 students from Myanmar universities

## Activities of Infrastructure Group

Installation of simple monitoring system in damaged bridges
Analysis of the structural performance
(Short-term countermeasure)

Estimation of cause of early fracture of bolts in steel-truss bridge
(Long-term maintenance)

Establishment of bridge database system
(Strategic bridge management)

Measurement of road surface roughness
(Pavement management)

[Future plan] Construction quality control
Installation of monitoring system to new bridges
Training for casting concrete (quality control)

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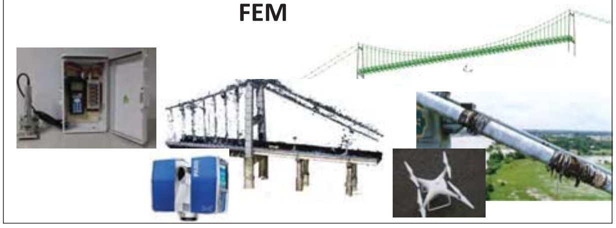
## Assessment of Damaged Suspension Bridge in Myanmar

1. Twantay Bridge
2. Pathein Bridge (on-going)

Key technologies: Simple monitoring
3D laser scanner, Drone
FEM



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## Proposal 1

When damage is found,
Install an appropriate
simple monitoring system.

Case studies
Twantay bridge
Thkatut bridge

In the future,
For new important structures, we suggest to install the simple monitoring system from the beginning to capture the initial deformation.

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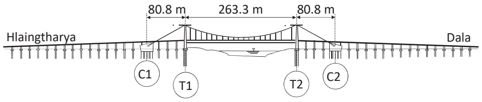
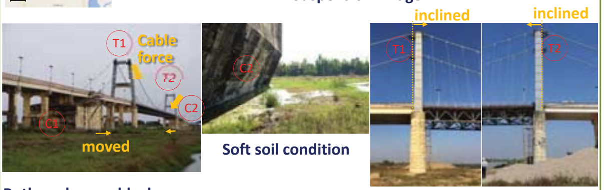
## Twantay Bridge (constructed 2006)

**Yangon Map**

**General view:**



Suspension Bridge



Soft soil condition

Both anchorage blocks move

Both main towers are inclined

Main tower T2 is the target for the monitoring

Both main towers are inclined in bridge axis direction

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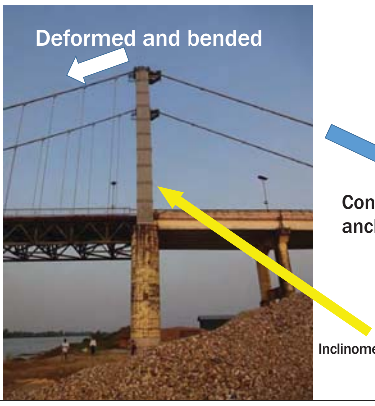
## Tower is inclined



Concrete anchorage

Inclinometer installed

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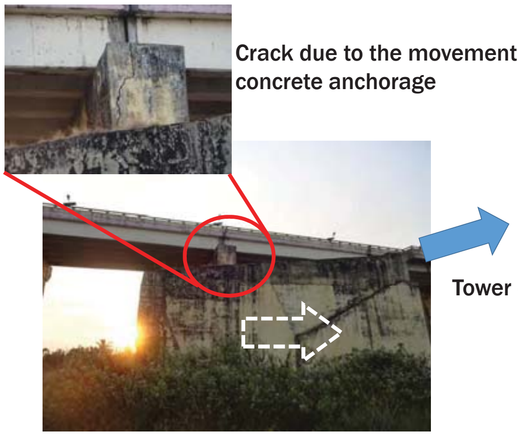

Crack due to the movement of concrete anchorage

Tower cable

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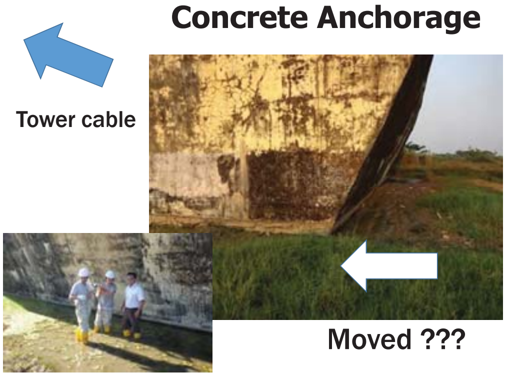
## Concrete Anchorage



Tower cable

Moved ???

Simple soil test

---

## Recent cracked anchorage survey

**South side anchorage**

Crack under anchorage — Crack in neighboring pier

This cracks are supposed to be occurred due to movement of anchorage

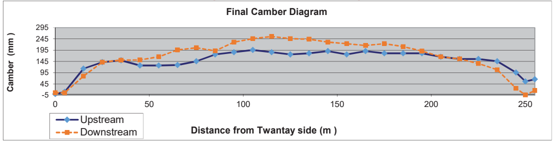
**North side anchorage**

No crack in north side

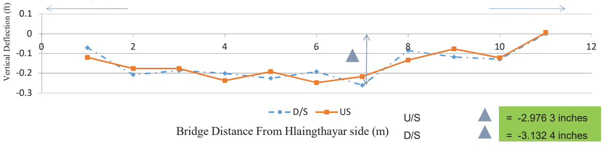
## Measurement of road level by MOC

TWANTY BRIDGE MAIN BRIDGE STEEL TRUSS LEVEL CHECKING HLAINGTHAYAR TO DALA

(Date: 9.6.2006)



(Date . 25.1.2012 , Time : 2:00 PM)



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## Is it still moving??

### Measure the tower inclination

### Monitoring plan

Equipment: inclinometer and data logger
Measurement interval: 3 hours
Measurement term: 1 year (check every 3 months)

Monitor the inclination in bridge axis direction of the main tower

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## Question

**Is this bridge safe ?**

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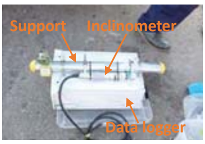
## Twantay Bridge

1. Prepared support for Inclinometer & data logger

2. Set inclinometer and data logger at support



3. Connected inclinometer to data logger

4. Crane was used to reach the position

5. Mounted monitoring system at the main tower

6. Monitoring system was set at the main tower

Set an inclinometer and a data logger at the height of 10 m above read level

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## Twantay Bridge

Monitoring system was set successfully at South Main Tower

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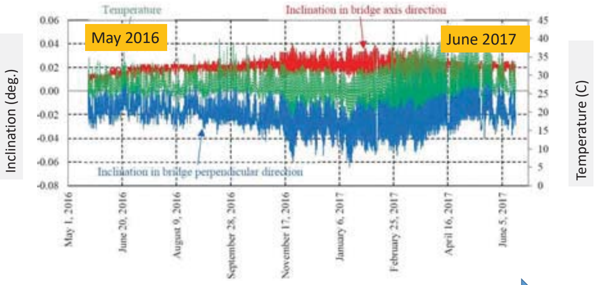
## Tower inclination



12 months data: No significant change
Twantay bridge is now almost stable

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**Correction of data**

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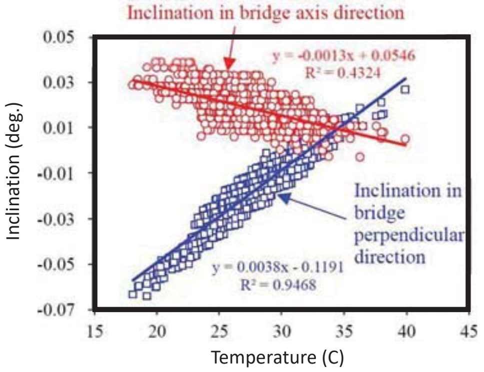
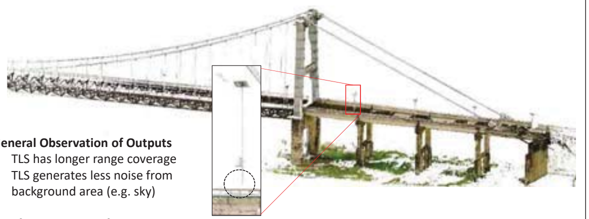
Data arranged by Dr. Miyashita



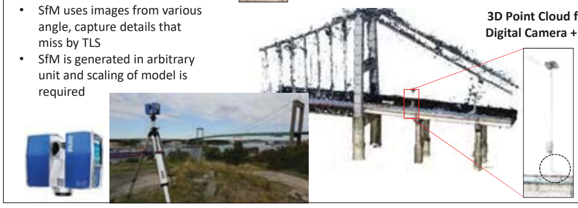
Tower inclination degree follows the temperature.
No damage is found from the data in 12 months.

## 3D Point Cloud from Terrestrial Laser Scanner



**General Observation of Outputs**
- TLS has longer range coverage
- TLS generates less noise from background area (e.g. sky)

- SfM uses images from various angle, capture details that miss by TLS
- SfM is generated in arbitrary unit and scaling of model is required



3D Point Cloud from Digital Camera + SfM

## Is this bridge still safe??

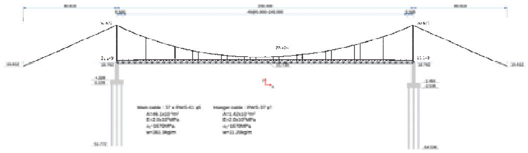
### Analysis of Twantay Bridge

長岡技術科学大学
Nagaoka University of Technology
Prof. Eiji IWASAKI
Member of SATREPS Project
Nagaoka University of Technology

### Simulation Procedure

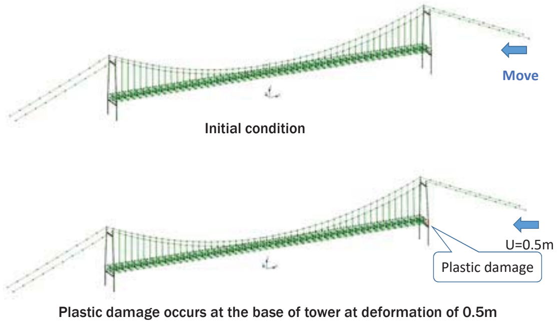
1.
Set the initial condition considering the tension force of cable and hangers.

2.
In the simulation, move the anchorage (right side only). In one step, 0.05m.
Then, check the deformation, plasticity damage, cable force etc.



## 3D Point Cloud from Terrestrial Laser Scanner

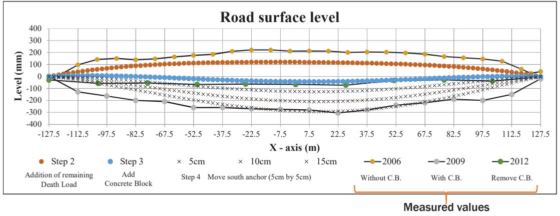
### Overall deformation



Plastic damage occurs at the base of tower at deformation of 0.5m

## Road Surface Level

**Step 3 & Step 4**



Measured values

- Step 3 : Addition of 300ton Concrete block railing
- With only concrete block addition, could not be reproduced the 2009 data values
- Apply a 5cm by 5cm displacement in the South Anchor

With this there is a possibility that the concrete anchor may moved about 15cm in the course of 6years

## Tower Shape

**Tower displacement results**

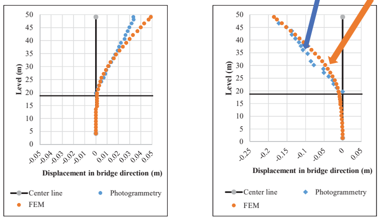
FEM results are from Step 5 ( Twantay Bridge current state )

Photogrammetry results were obtained after FEM analysis

Measurement at site

FEM simulation

**Tower North side**



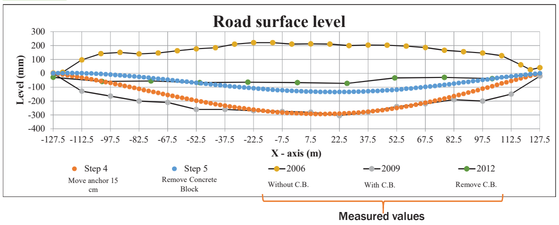
**Tower South side**

| | | | |
|---|---|---|---|
| Error: 1.7cm | 3D: top of tower disp = 3.3cm<br>FEM: top of tower disp = 5.0 cm | 3D: top of tower disp = 17.2cm<br>FEM: top of tower disp = 18.5cm | Error: 1.3cm |

Results from photogrammetry well matched FEM analysis

## Road Surface Level

**Step 5**



Measured values

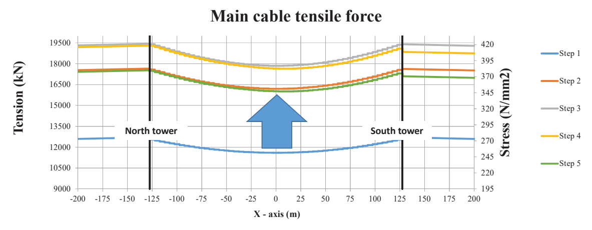
- Step 5 results of removing concrete block are close to 2012 data

After removing the bridge railing block, values well-matched and the current state of the bridge was reproduced

## Cable Stress Level

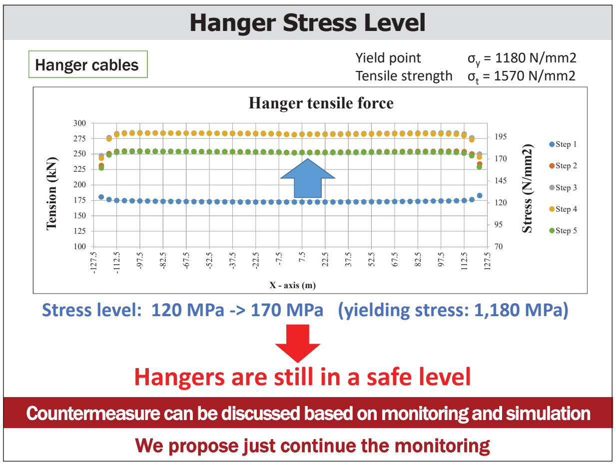
**Main cable**

Yield point $\sigma_y$ = 1180 N/mm2
Tensile strength $\sigma_t$ = 1570 N/mm2



Stress level: 270 MPa -> 370 MPa (yielding stress: 1,180 MPa)

Cables are still in a safe level



## Hanger Stress Level

**Hanger cables**

Yield point $\sigma_y$ = 1180 N/mm2
Tensile strength $\sigma_t$ = 1570 N/mm2

Stress level: 120 MPa -> 170 MPa (yielding stress: 1,180 MPa)

Hangers are still in a safe level

**Countermeasure can be discussed based on monitoring and simulation**

We propose just continue the monitoring

## Recent suspension bridge failure in Indonesia

### Kutai-Kartanegara Bridge in Indonesia

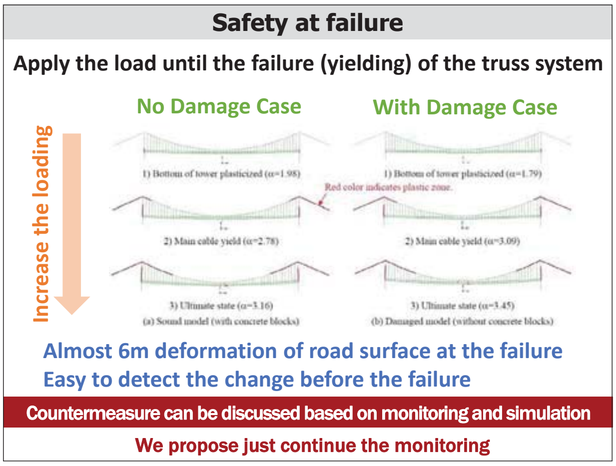
In November 26, 2011 Kutai-Kartanegara Bridge, the longest suspension bridge in Indonesia collapsed

In 2006, in order to an inspection by local government, the next damages was found:

- Anchorage block moved 13.5cm (from expansion device)
- Camber at center of main span was reduced by 55cm
- Towers inclined in bridge direction

Same problem as Twantay Bridge

**Camber restoration work was done.**
**During this work, one clamp broke leading to the bridge collapse !**



## Safety at failure

Apply the load until the failure (yielding) of the truss system

Almost 6m deformation of road surface at the failure
Easy to detect the change before the failure

**Countermeasure can be discussed based on monitoring and simulation**

We propose just continue the monitoring

## Based on this achievement

### Simple monitoring for long-span bridges

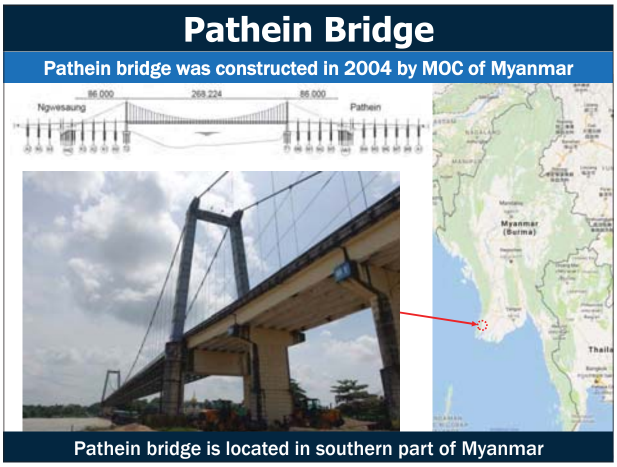
**New Bago River Bridge supported by JICA (planning)**

Install similar monitoring system to new bridge in Yangon to check an early deflection of cable-stayed bridge.

Now in design process

Set inclinometer at the tower top.
Monitor several years.

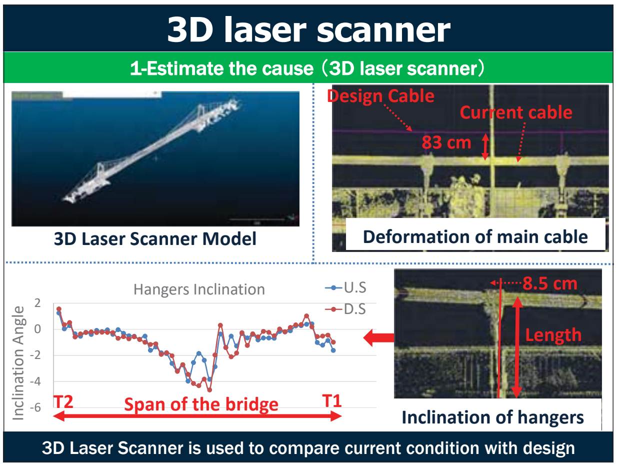
**We wish this concept for new long-span bridge becomes standard**



## Pathein Bridge

**Pathein bridge was constructed in 2004 by MOC of Myanmar**

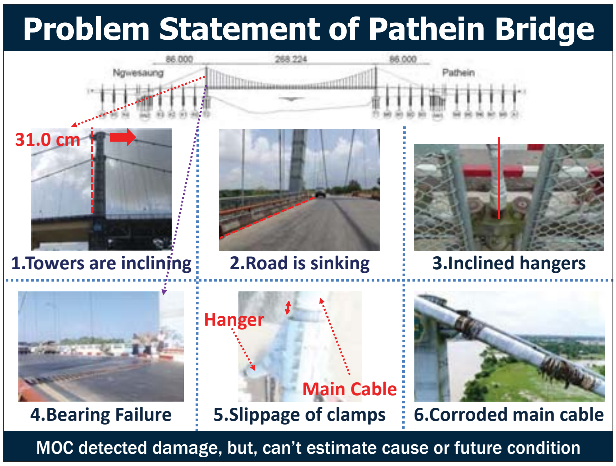
Pathein bridge is located in southern part of Myanmar



## 3D laser scanner

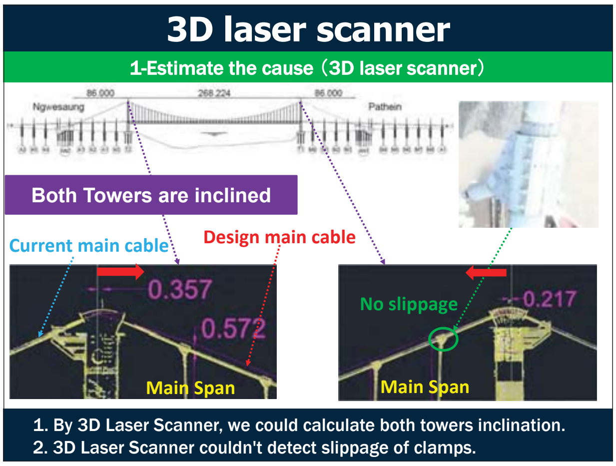
**1-Estimate the cause（3D laser scanner）**

3D Laser Scanner is used to compare current condition with design



## Problem Statement of Pathein Bridge

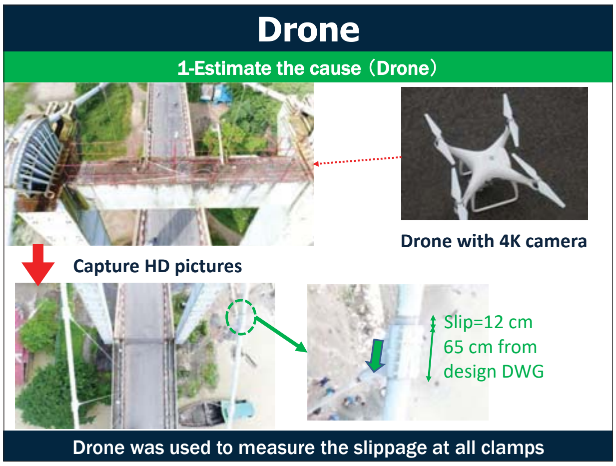
MOC detected damage, but, can't estimate cause or future condition



## 3D laser scanner

**1-Estimate the cause（3D laser scanner）**

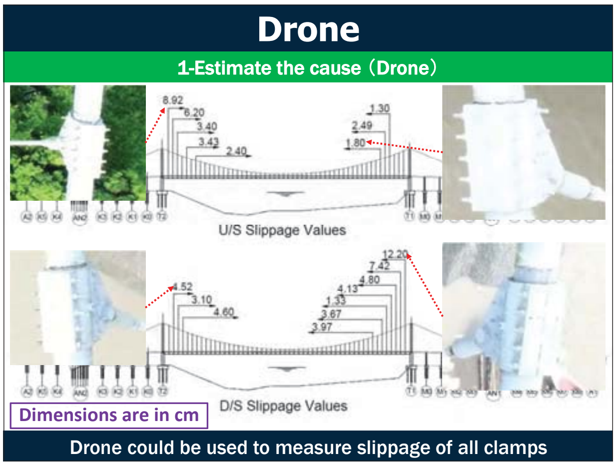
1. By 3D Laser Scanner, we could calculate both towers inclination.
2. 3D Laser Scanner couldn't detect slippage of clamps.



# Drone

**1-Estimate the cause（Drone）**

Drone was used to measure the slippage at all clamps

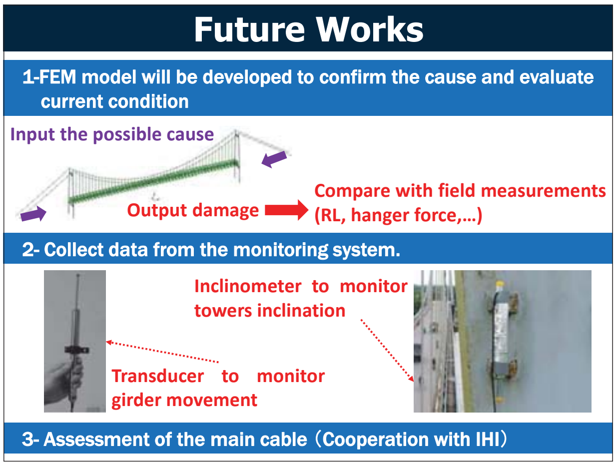


# Drone

**1-Estimate the cause（Drone）**

Dimensions are in cm

Drone could be used to measure slippage of all clamps

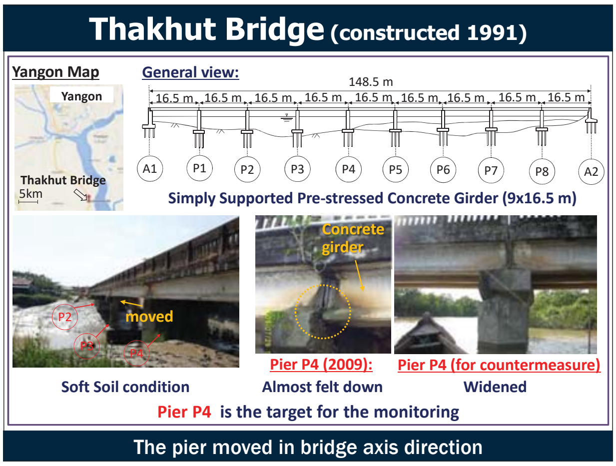


# Future Works

**1-FEM model will be developed to confirm the cause and evaluate current condition**

**2- Collect data from the monitoring system.**

**3- Assessment of the main cable（Cooperation with IHI）**



# Thakhut Bridge (constructed 1991)

Simply Supported Pre-stressed Concrete Girder (9x16.5 m)

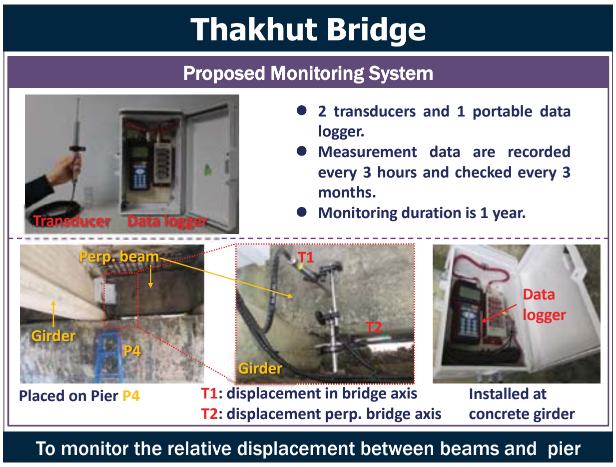
Soft Soil condition

Pier P4 (2009): Almost felt down

Pier P4 (for countermeasure) Widened

Pier P4 is the target for the monitoring

The pier moved in bridge axis direction



# Thakhut Bridge

**Proposed Monitoring System**

- 2 transducers and 1 portable data logger.
- Measurement data are recorded every 3 hours and checked every 3 months.
- Monitoring duration is 1 year.

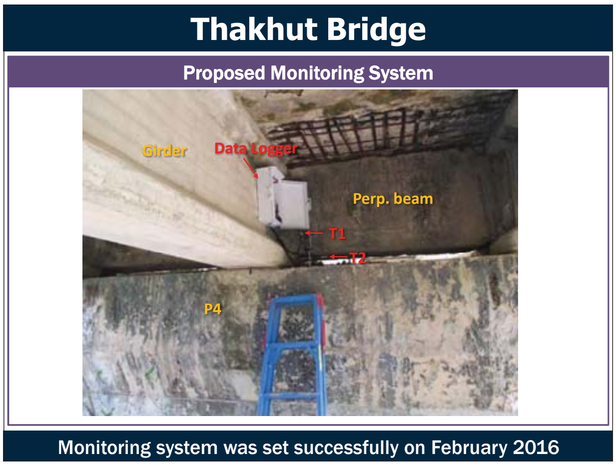
Placed on Pier P4

T1: displacement in bridge axis

T2: displacement perp. bridge axis

Installed at concrete girder

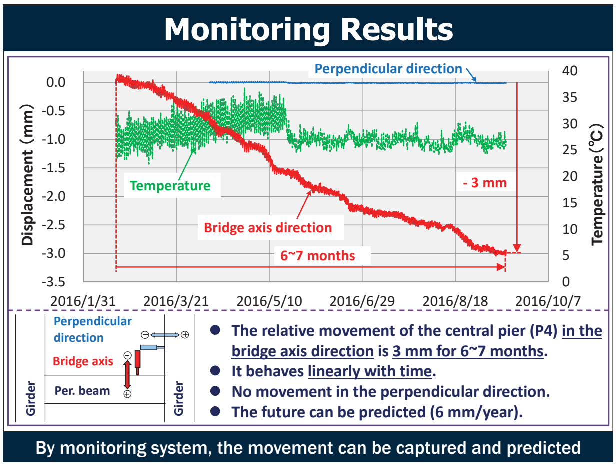
To monitor the relative displacement between beams and pier



# Thakhut Bridge

**Proposed Monitoring System**

Monitoring system was set successfully on February 2016



# Monitoring Results

- The relative movement of the central pier (P4) in the bridge axis direction is 3 mm for 6~7 months.
- It behaves linearly with time.
- No movement in the perpendicular direction.
- The future can be predicted (6 mm/year).

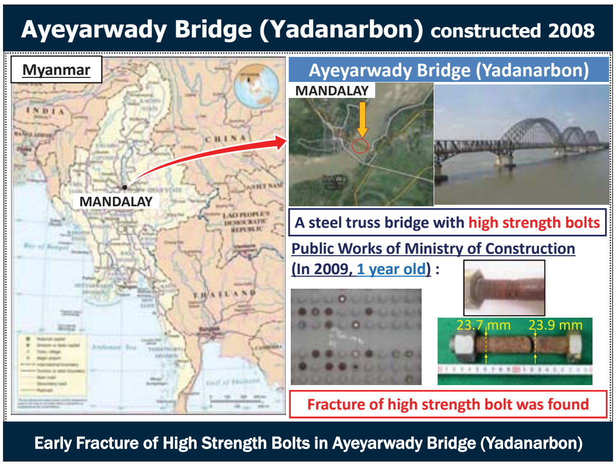
By monitoring system, the movement can be captured and predicted

# Proposal 2

Maintenance strategy should be established for damaged bridges (long term planning)

Case study

Ayeyarwady Bridge (Yadanarbon)



## Ayeyarwady Bridge (Yadanarbon) constructed 2008

**Ayeyarwady Bridge (Yadanarbon)**

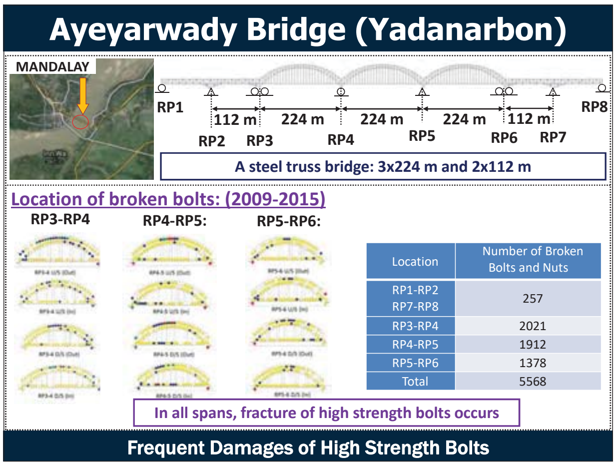
A steel truss bridge with high strength bolts

Public Works of Ministry of Construction

(In 2009, 1 year old) :

Fracture of high strength bolt was found

**Early Fracture of High Strength Bolts in Ayeyarwady Bridge (Yadanarbon)**



## Ayeyarwady Bridge (Yadanarbon)

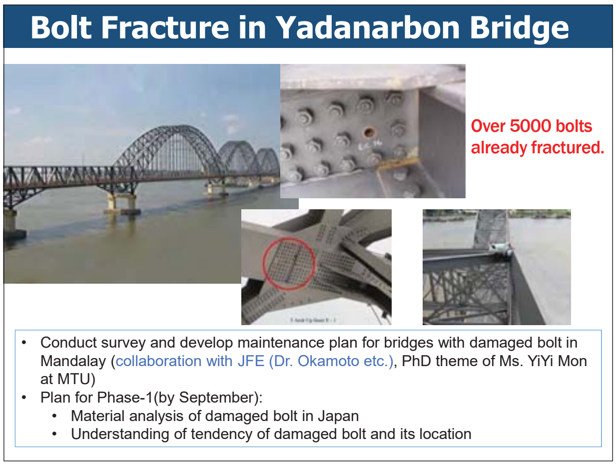
A steel truss bridge: 3x224 m and 2x112 m

**Location of broken bolts: (2009-2015)**

| Location | Number of Broken Bolts and Nuts |
|---|---|
| RP1-RP2<br>RP7-RP8 | 257 |
| RP3-RP4 | 2021 |
| RP4-RP5 | 1912 |
| RP5-RP6 | 1378 |
| Total | 5568 |

In all spans, fracture of high strength bolts occurs

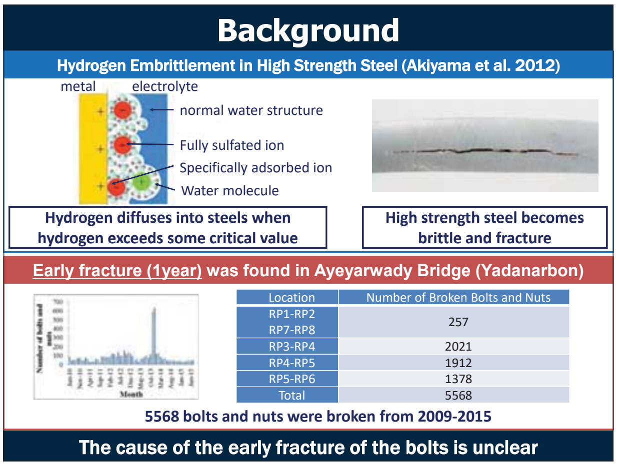
**Frequent Damages of High Strength Bolts**



## Bolt Fracture in Yadanarbon Bridge

Over 5000 bolts already fractured.

- Conduct survey and develop maintenance plan for bridges with damaged bolt in Mandalay (collaboration with JFE (Dr. Okamoto etc.), PhD theme of Ms. YiYi Mon at MTU)
- Plan for Phase-1(by September):
  - Material analysis of damaged bolt in Japan
  - Understanding of tendency of damaged bolt and its location



## Background

**Hydrogen Embrittlement in High Strength Steel (Akiyama et al. 2012)**

Hydrogen diffuses into steels when hydrogen exceeds some critical value

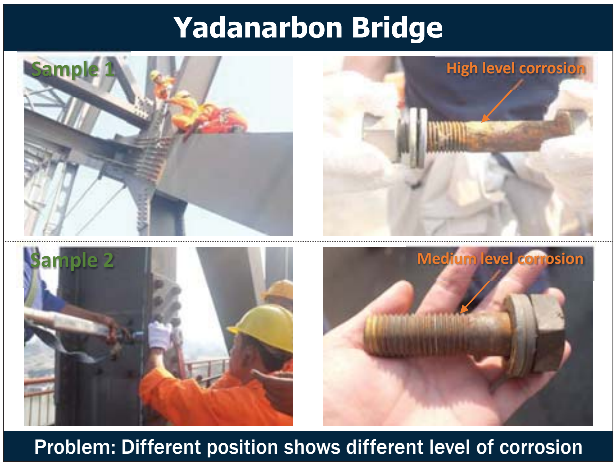
High strength steel becomes brittle and fracture

**Early fracture (1year) was found in Ayeyarwady Bridge (Yadanarbon)**

| Location | Number of Broken Bolts and Nuts |
|---|---|
| RP1-RP2<br>RP7-RP8 | 257 |
| RP3-RP4 | 2021 |
| RP4-RP5 | 1912 |
| RP5-RP6 | 1378 |
| Total | 5568 |

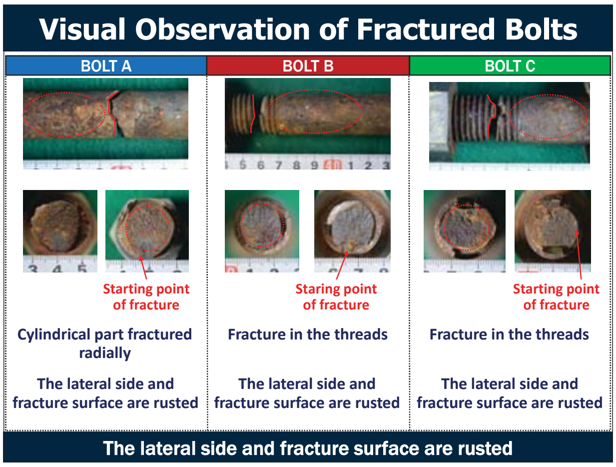
5568 bolts and nuts were broken from 2009-2015

**The cause of the early fracture of the bolts is unclear**



## Yadanarbon Bridge

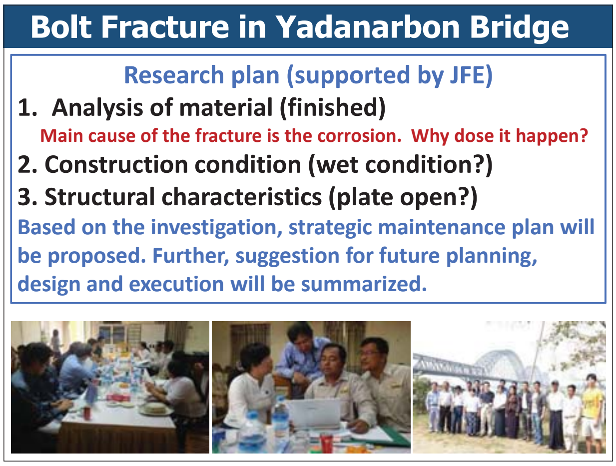
**Problem: Different position shows different level of corrosion**



## Visual Observation of Fractured Bolts

| BOLT A | BOLT B | BOLT C |
|---|---|---|
| Starting point of fracture | Staring point of fracture | Starting point of fracture |
| Cylindrical part fractured radially | Fracture in the threads | Fracture in the threads |
| The lateral side and fracture surface are rusted | The lateral side and fracture surface are rusted | The lateral side and fracture surface are rusted |

**The lateral side and fracture surface are rusted**



## Bolt Fracture in Yadanarbon Bridge

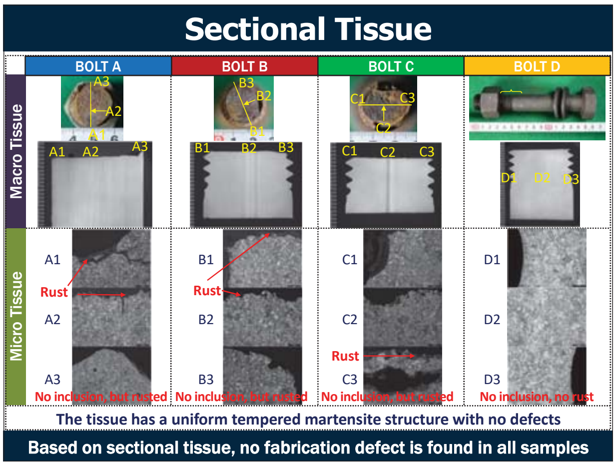
**Research plan (supported by JFE)**

1. Analysis of material (finished)

   Main cause of the fracture is the corrosion. Why dose it happen?

2. Construction condition (wet condition?)
3. Structural characteristics (plate open?)

Based on the investigation, strategic maintenance plan will be proposed. Further, suggestion for future planning, design and execution will be summarized.



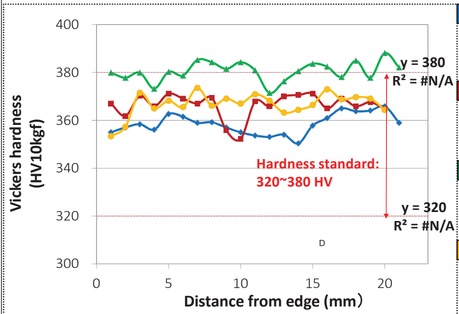
## Sectional Tissue

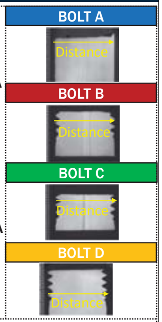
| BOLT A | BOLT B | BOLT C | BOLT D |
|---|---|---|---|
| No inclusion, but rusted | No inclusion, but rusted | No inclusion, but rusted | No inclusion, no rust |

The tissue has a uniform tempered martensite structure with no defects

**Based on sectional tissue, no fabrication defect is found in all samples**

# Hardness Testing





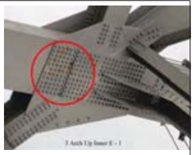
| | |
|---|---|
| Hardness values: Bolt A = 350~366 HV | BOLT C = 371~388 HV |
| Bolt B = 352~371 HV | BOLT D = 353~374 HV |
| Standard value : 320~380 HV | |

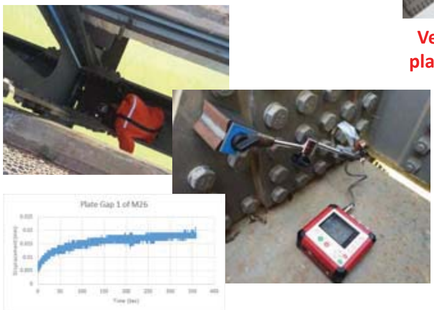
**Hardness of all samples are still within acceptable value**

---

## Step 2: Plate gap measurement

**Now done by Ms. Yi Yi Mon of MTU**



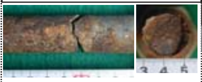
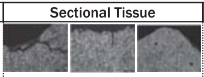
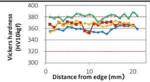


**Very complicated plate arrangement.**

**Local stress and deformation ??**

---

# Conclusion

| Visual Observation | Sectional Tissue | Hardness Testing |
|---|---|---|
|  |  |  |
| Bolts are rusted | No inclusion or defect | Values are acceptable |

**There is no defect in the raw material**

**Other mechanisms may take place and cause the early fracture of high strength bolts:**

- **Corrosion of bolts and nuts due to water entering, very corrosive environment, or**
- **The vibration is so large, and the bolts are gradually loosened.**

**These possible causes are still being investigated.**

---

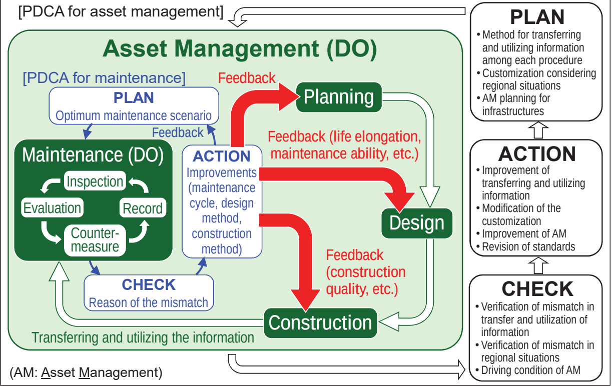
# Proposal 3

Bridge management system
should be established.
For the first step,
create a bridge database with GIS.

Target area
Yangon region, for the first step

---

# Infrastructure Asset Management Incorporating Life Cycle Management



**PLAN**
- Method for transferring and utilizing information among each procedure
- Customization considering regional situations
- AM planning for infrastructures

**ACTION**
- Improvement of transferring and utilizing information
- Modification of the customization
- Improvement of AM
- Revision of standards

**CHECK**
- Verification of mismatch in transfer and utilization of information
- Verification of mismatch in regional situations
- Driving condition of AM

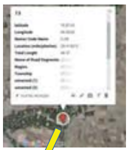
(AM: Asset Management)

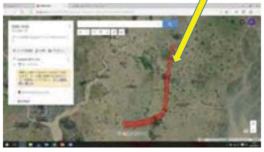
---

# Now.....

**In May 2016, with YTU students and MOC staff, investigation has started.**
**80 are finished by student.**

YTU student





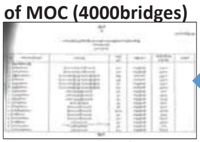
In Myanmar, 4000. In Yangon region, 700. How to do....
Collaboration with JICA Technical Cooperation Project

---

# Bridge Database with GIS

## Development of infrastructure database in Myanmar

**Bridge inventory list of MOC (4000bridges)**



Now, translating from Myanmar to English

Link

**GIS data of bridges in Yangon region. (900 bridges) From another JICA project**

- Translation of bridge data in progress (Yangon is completed)
- linking of locational info (for Yangon only) with other JICA project
- Plan for Phase-1(by September): Construction of GIS database for Yangon city

**How to link the information.**
**Origin (starting point) of the roads is necessary.**

---

# Additional investigation

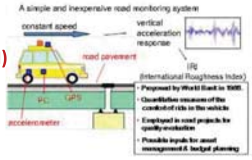
Road surface roughness condition
in Yangon is surveyed by simple
measurement system.

Survey area
Yangon city area

# Road roughness in Yangon

## Dynamic Response Intelligent Monitoring System (DRIMS)

Convert the data into
International Roughness index (IRI)

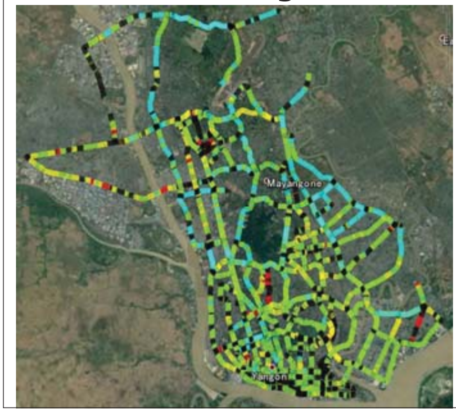


Very easy and simple !!

HP: http://vims.sakura.ne.jp/

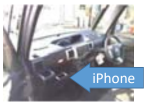
---

# Trial run in Yangon



Condition
Good
🟦
🟩
🟨
🟥
Bad

---



To calculate IRI, over 30 km/h is necessary.

---

# Next step

Star from this year

Quality control of
new infrastructure is important
For Myanmar

Proposal 1
Simple monitoring for long-span bridge

Proposal 2
Training for good casting of concrete

---

# Proposal 1

## Simple monitoring for long-span bridges

*New Bago River Bridge*
*supported by JICA (planning)*

Install similar monitoring system to new bridge in Yangon to check an early deflection of cable-stayed bridge.

JICA has agreed this concept

Set inclinometer at the tower top.
Monitor several years.

3D laser scan also is planned

We wish this concept for new long-span bridge becomes standard

---

# Example of precast concrete factory in Japan

All procedures can be learnt

---

# Proposal 2

## Training for good casting of concrete

Training at Precast Factory is very efficient for training of casing a concrete

I&H Engineering Co.,Ltd (MOC (Myanmar) and IHI (Japan))

Open April 2017 near Yangon

HP of IHI

- Material quality
- Casting and Prestressing
- Curing
- Testing

All controlled in the factory !!

*“ Learn the best quality first. Then, go to construction sites “*

We wish Training Course is established based on
JICA Technical Cooperation Project

---

SATREPS Project continues by 2020 !

Thank you for your kind attention

Kohei NAGAI
Associate Professor
International Center for Urban Safety (ICUS)
The University of Tokyo

ANZEN SATREPS : MYANMAR NIPPON ZERO-CASUALTY ENGINEERING NETWORK 2014-2020

# Contents of November Lecture

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[11-1-1,2,3] Maintenance, Monitoring

[11-2-1] Influence line of girders
[11-2-2] Influence line of trusses
[11-2-3] Design method

[11-3-1] Buckling and its strength design of columns (DVD)
[11-3-2] Buckling and its strength design of beam and beam columns (DVD)
[11-3-3] Buckling and its strength design of unstiffened and stiffened plates (DVD)

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[11-4-1] Connection (Bolt) (DVD)
[11-4-2] Connection (Welding) (DVD)
[10-4-3] Fatigue and its design

[11-5-1] Design of slabs
[10-5-2] Design of girder bridges
[10-5-3] Design of truss bridges

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## Contents of December Lecture

[1] Vibration DVD and basic theory

[2~5] Cable-stayed bridges

History
Name of members and structure
Design parameters and selection
Estimation of stress resultants
Design of Girder, Tower and Cable
Erection, DVD
Wind-resistance design
Super long-span bridges

---

[11-2-1]

# Influence line of Beam (Girder)

---

At the design,
We need to know the stress resultants (N, M, Q) and stress ($\sigma$, $\tau$) of structures under loading to check the safety.

$M_D$ ($\sigma_D$) under dead loading
$M_{[L+I]}$ ($\sigma_{[L+I]}$) under live loading
$M_T$ ($\sigma_T$) under temperature change
$M_W$ ($\sigma_W$) under wind load
.....
.....

---

## Safety check

$\Sigma f_i \leqq h \cdot f_a = h \cdot [\{f_{cr}(\leqq f_y)\}/v]$
$f : \sigma, \tau$
$v$ : safety factor
$h(\geqq 1.0)$ : incremental factor

(or)

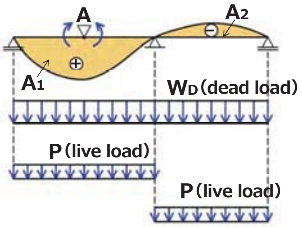
$\Sigma(\gamma_i S_i) \leqq \Phi R_n$
$S : N, M, Q$
$R_n : N_{cr}, M_{cr}, Q_{cr}$
$\gamma_i, \Phi$ : partial factor

---

In case of live loading,
(↑moving vehicle)
in order to evaluate action (f, S),

influence line is used.

## Influence line (bending moment at A )



$A_1, A_2$ : area

$M_D = W_D (A_1 + A_2)$

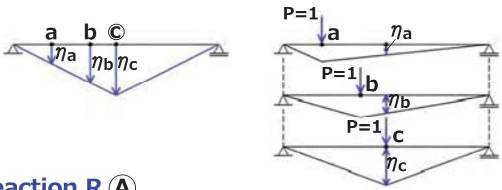
$M_{L, max.} = PA_1$

$M_{L, min.} = PA_2$

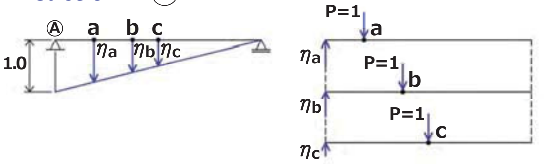
$M_{max.} = M_D + M_{L, max.}$

$M_{min.} = M_D - M_{L, min.}$

## Bending moment (at Ⓒ)

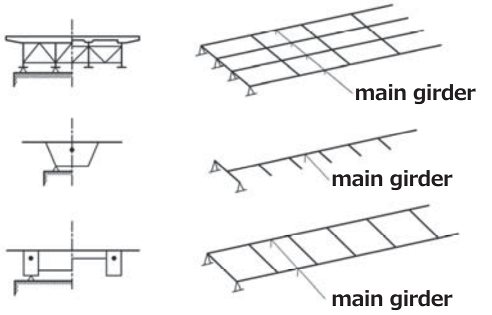


## Reaction R Ⓐ

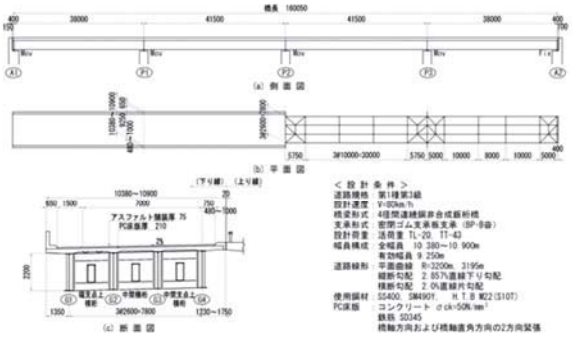


## Fish-bone and Grid models

(for obtaining stress resultants [N, M, Q] and deflection[δ])



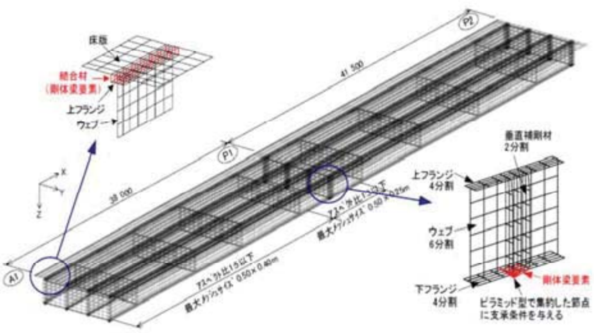
## Steel multi-I-girder bridge



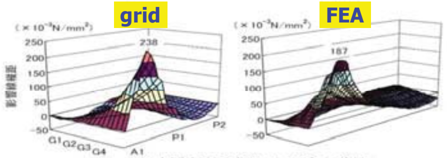
< 設 計 条 件 >
道路規格：第1種第3級
設計速度：V=80km/h
橋梁形式：4径間連続鋼非合成鈑桁橋
支承形式：密閉ゴム支承板支承（BP-B型）
設計荷重：活荷重 TL-20、TT-43
幅員構成：全幅員 10.380～10.900m
有効幅員 9.250m
道路線形：平面曲線 R=3200m、3195m
縦断勾配 2.857%直線下り勾配
横断勾配 2.0%直線片勾配
使用鋼材：SS400、SM490Y、H.T.B M22(S10T)
PC床版：コンクリート σck=50N/mm²
鉄筋 SD345
橋軸方向および橋軸直角方向の2方向緊張

---

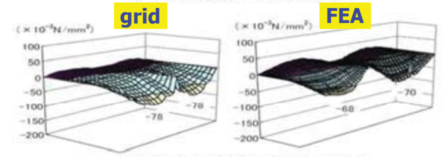
## Girder bridge modelled by Shell Element



## Ex. of influence line analysis



(a) 支間中央G4桁下フランジσx着目

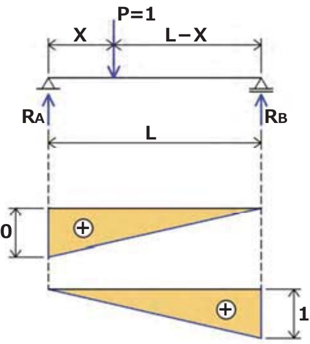


(b) 中間支点上G4桁下フランジσx着目

**3D influence volume (FEA vs GRID)**

# Simple beam

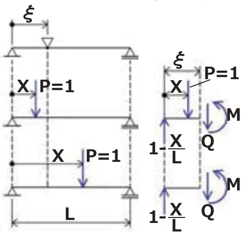
## Reactions RA, RB



$R_A = 1 - \frac{X}{L}$

$R_B = \frac{X}{L}$

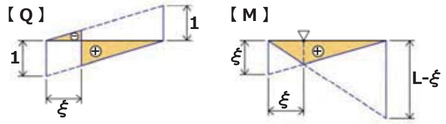
## Moment and Shear at ξ



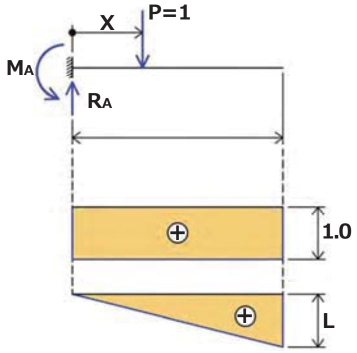
$Q=1-\frac{X}{L}-1=-\frac{X}{L}$ , $M=\left(1-\frac{X}{L}\right)\xi-(\xi-X)$

$=X\left(1-\frac{\xi}{L}\right)$

$Q=1-\frac{X}{L}$ , $M=\left(1-\frac{X}{L}\right)\xi$



## Reactions $M_A$, $R_A$

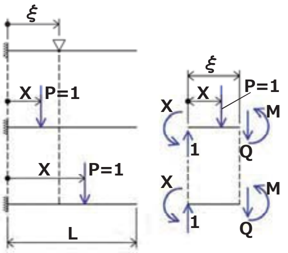


$R_A = 1$

$M_A = X$

## Cantilevered beam

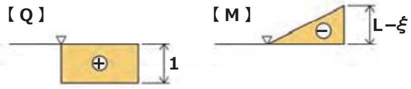
## Moment and shear at ξ



$Q=1-1=0, M=\xi-(\xi-X)-X$

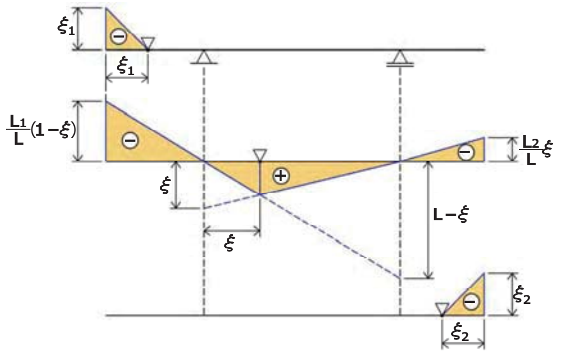
$=0$

$Q=1$ , $M=\xi-X$

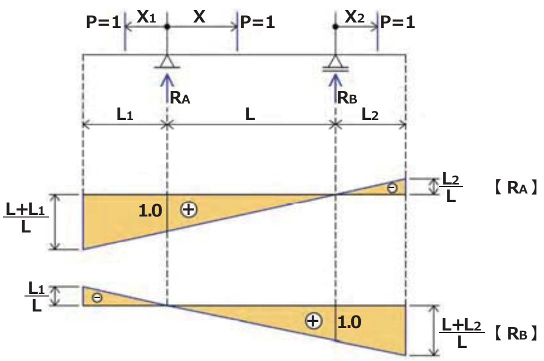


## Simple beam with wings

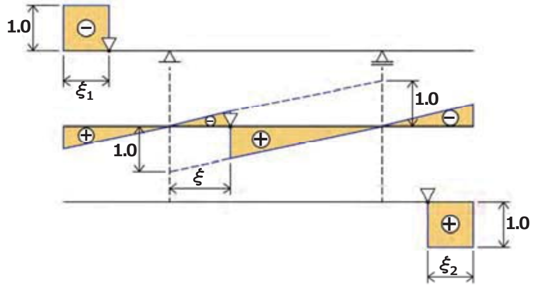
## Moment at $\xi_1$, ξ, $\xi_2$



## Reactions $R_A$, $R_B$



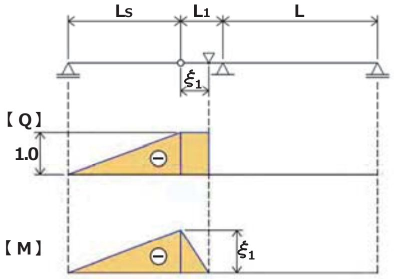
## Shear at $\xi_1$, ξ and $\xi_2$



# Gerber beam

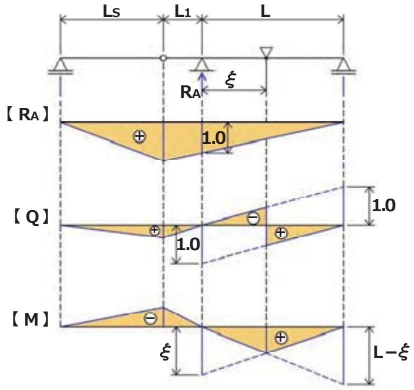
---

## Moment and shear at $\xi_1$



---

## $R_A$, moment and shear at $\xi$

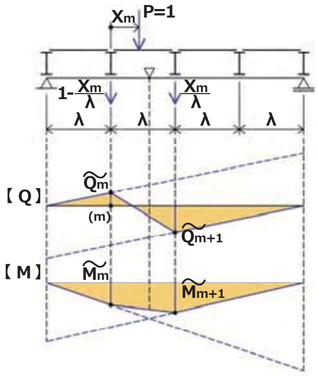


---

## Simple beam
## (Indirect loading)

---

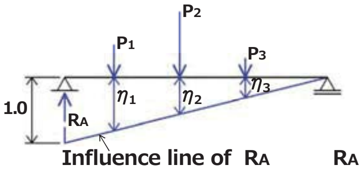
## Moment and shear at point (▽)



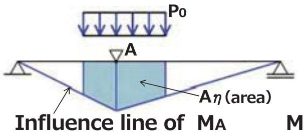
$\widetilde{Q}_m$, $\widetilde{M}_m$ are shear force and moment at (▽) position when unit load is applied to point (m) .

---

## Reaction and moment using influence line



Influence line of $R_A$ $\quad R_A = \sum_i^3 P_i \eta_i$



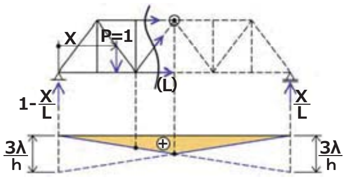
Influence line of $M_A$ $\quad M_A = P_0 A_\eta$

[11-2-2]

# Influence line of Truss

---

## Lower chord member (L)



( $0 \le X \le 2X$ )

$$Lh + (3\lambda - X) = \left(1-\frac{X}{L}\right)3\lambda$$

$$L = \frac{1}{h}\left(1-\frac{3\lambda}{L}\right)X = \frac{1}{h}\frac{3\lambda}{L}X$$

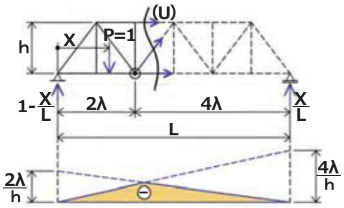
( $3\lambda \le X \le L\ (=6\lambda)$ )

$$Lh = \left(1-\frac{X}{L}\right)3\lambda$$

$$L = \frac{3\lambda}{h}\left(1-\frac{X}{L}\right)$$

---

## Upper chord member (U)



( $0 \le X \le 2\lambda$ )

$$Uh + \left(1-\frac{X}{L}\right)(2\lambda) = 2\lambda - X$$

$$U = -\frac{1}{h}\left(1-\frac{2\lambda}{L}\right)X = -\frac{1}{h}\frac{4\lambda}{L}X$$

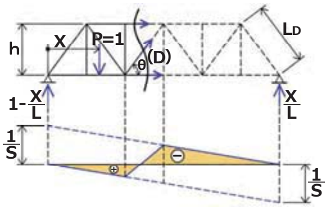
( $2\lambda \le X \le L\ (=6\lambda)$ )

$$Uh + \left(1-\frac{X}{L}\right)(2\lambda) = 0$$

$$U = -\frac{2\lambda}{h}\left(1-\frac{X}{L}\right)$$

---

## Diagonal member (D)



$$S = \sin\theta = \frac{h}{L_D}$$

$$L_D = \sqrt{\lambda^2 + h^2}$$

( $0 \le X \le 2\lambda$ )

$$D \cdot S + \left(1-\frac{X}{L}\right) = 1 \implies D = \frac{1}{S}\frac{X}{L}$$

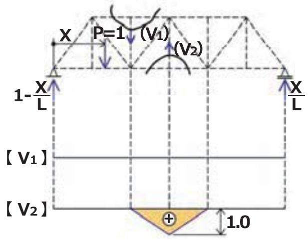
( $3\lambda \le X \le L\ (=6\lambda)$ )

$$D \cdot S + \left(1-\frac{X}{L}\right) = 0 \implies D = -\frac{1}{S}\left(1-\frac{X}{L}\right)$$

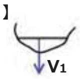
---

## Vertical members

Vertical member（ $V_1, V_2$ ）



【 $V_1$ 】



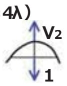
$V_1 = 0$

【 $V_2$ 】

( $0 \le X \le 2\lambda$ ) and ( $4\lambda \le X \le 6\lambda$ )

$V_2 = 0$

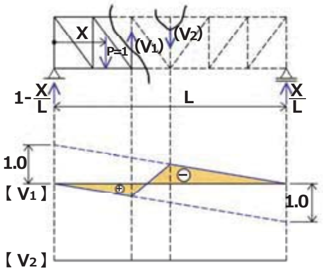
( $X = 4\lambda$ )



$V_2 = 1$

---

## [lower deck type]



【 $V_1$ 】

( $0 \le X \le 2\lambda$ )

$$V_1 + \left(1-\frac{X}{L}\right) = 1 \rightarrow V_1 = \frac{X}{L}$$

( $3\lambda \le X \le L\ (=6\lambda)$ )

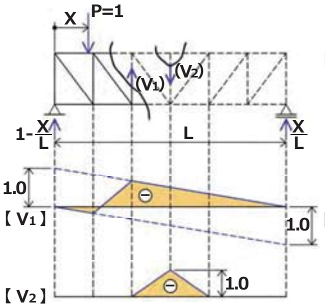
$$V_1 + \left(1-\frac{X}{L}\right) = 0 \rightarrow V_1 = -\left(1-\frac{X}{L}\right)$$

【 $V_2$ 】

$V_2 = 0$

---

## [upper deck type]



【 $V_1$ 】

( $0 \le X \le \lambda$ )

$$V_1 + \left(1-\frac{X}{L}\right) = 1 \rightarrow V_1 = \frac{X}{L}$$

( $2\lambda \le X \le L\ (=6\lambda)$ )

$$V_1 + \left(1-\frac{X}{L}\right) = 0 \rightarrow V_1 = -\left(1-\frac{X}{L}\right)$$

【 $V_2$ 】

( at $X = 3\lambda$ )

$$V_2 + 1 = 0 \quad \rightarrow V_2 = -1$$

# Bridge Design (Japan, AASHTO LRFD, EC)

---

# Process of the Planning

Necessity of the Project (Feasibility Study)

↓

**Road Design**

↓

**Basic Design of Structure**
(Selection of Bridge Type)

← Natural Condition Surveys

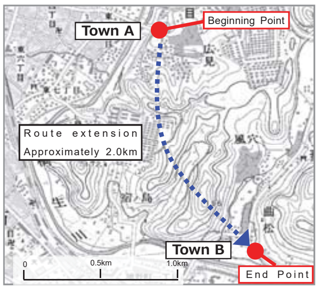
---

# Road-Building Plan

## From the study of a new road location to the selection of Bridge Type

---

# Road Design



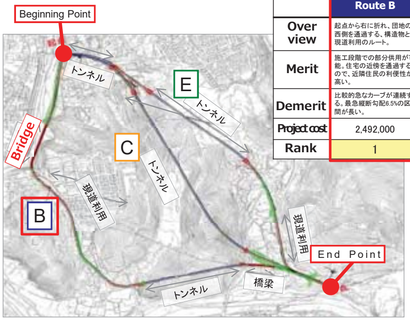
Connect from the beginning point to the end point.
Design a route of 2.0km

1) Road Division : 3 Type 4 Grade
2) Design Speed : V=40 km/h
3) Configuration of Road : Round Trip 2 Lanes

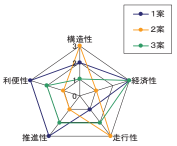
Component of Road Cross Section

---

# Comparison of Road Routes



| | Route B | Route C | Route E |
|---|---|---|---|
| Over view | 起点から右に折れ、団地の西側を通過する、構造物と現道利用のルート。 | 起点を直進してトンネル区間となり、緩やかな線形で終点に接続するルート。 | 起点を直進してトンネル区間となり、緩やかな線形で県道に接続するルート。 |
| Merit | 施工段階での部分供用が可能。住宅の近傍を通過するので、近隣住民の利便性が高い。 | バイパスとしての機能を重視したため、走行性が高く、区間長が短い。 | 現道部分は施工段階での部分供用が可能。 |
| Demerit | 比較的急なカーブが連続する。最急縦断勾配6.5%の区間が長い。 | ほとんどが構造物であるため施工に時間を要し、コストも高い。 | 用地買収が必要となる。全線供用までには時間がかかる。 |
| Project cost | 2,492,000 | 3,701,000 | 2,491,000 |
| Rank | 1 | 3 | 2 |



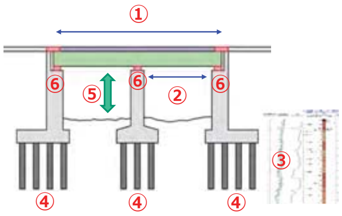
⇒ Select the Route B that excels in terms of economical, progress of the project and neighborhood convenience.

---

# Basic Design of Bridge

**Study Items**

① Bridge Length
② Span Length
③ Bearing Layer
④ Embedment of Foundation
⑤ Clearance under Girder
⑥ Support Condition



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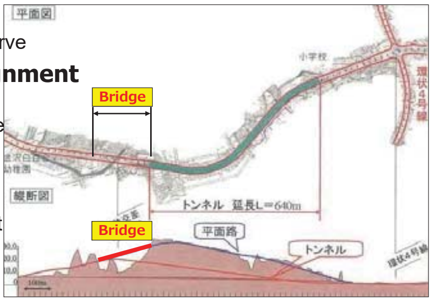
# Road Plan –Alignment-

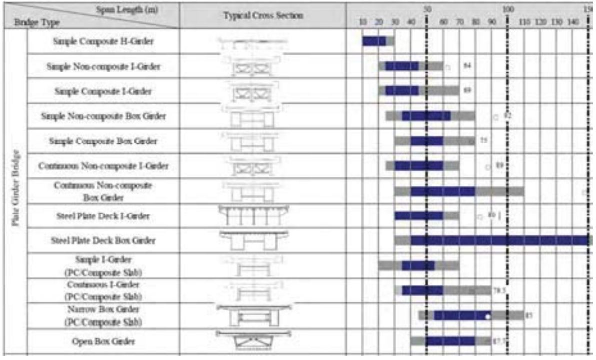
- **Horizontal Alignment**
  - ・Straight Line
  - ・Circle Curve
  - ・Transition Curve
- **Vertical Alignment**
  - ・Straight Line
  - ・Vertical Curve



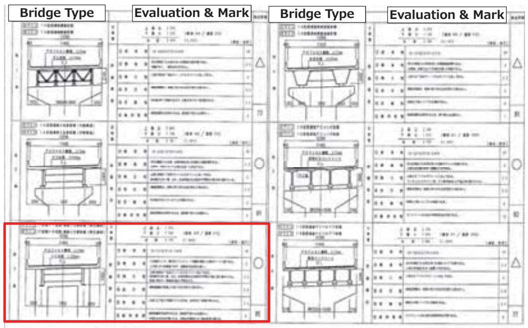
3-D Road Alignment
（Harmony of Horizontal & Vertical Alignment）

---

# Applicable bridge type at each span



## Comparison of Bridge Type



## After design of the slab

**[1] Assumption of steel weight**

**[2] Using fiber model (beam element), Influence line analysis**

↓

**Loading (live load includes impact)**

↓

**Stress resultants (M, Q) and deflection (δ)**

## Evaluation method in Japan

**Life-Cycle Cost（LCC）**

Total cost required for Life-cycle process

Calculation formula of Life-Cycle cost

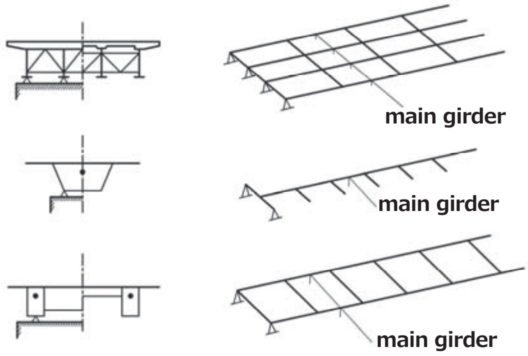
**LCC＝I＋ΣM＋ΣR**

I ：Initial Construction Cost
ΣM ：Total Maintenance Cost
ΣR ：Total Cost to Replacing

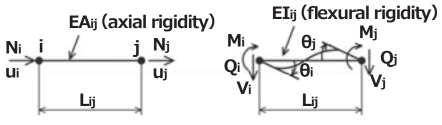
The request in recent years

→ **Minimum Maintenance Structure**

## Fish-bone and Grid models



## Truss and beam elements



$$\begin{Bmatrix} N_i \\ N_j \end{Bmatrix} = \frac{EA_{ij}}{L_{ij}} \begin{bmatrix} 1 & -1 \\ -1 & 1 \end{bmatrix} \begin{Bmatrix} u_i \\ u_j \end{Bmatrix}$$

↑Stiffness matrix

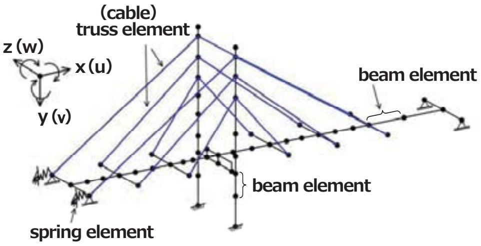
[truss element]

$$\begin{Bmatrix} Q_i \\ M_i \\ Q_j \\ M_j \end{Bmatrix} = \frac{EI_{ij}}{L_{ij}^3} \begin{bmatrix} 12 & 6L_{ij} & -12 & 6L_{ij} \\ & 4L_{ij}^2 & -6L_{ij} & 2L_{ij}^2 \\ & & 12 & -6L_{ij} \\ \text{sym.} & & & 4L_{ij}^2 \end{bmatrix} \cdot \begin{Bmatrix} V_i \\ \theta_i \\ V_j \\ \theta_j \end{Bmatrix}$$

↑Stiffness matrix

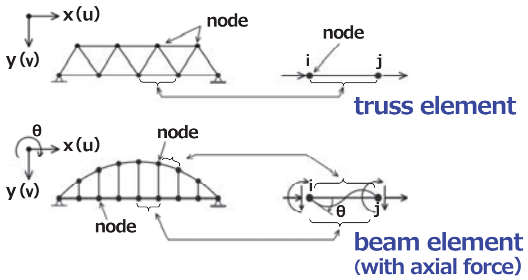
[beam element]

## Spatial（3D）model

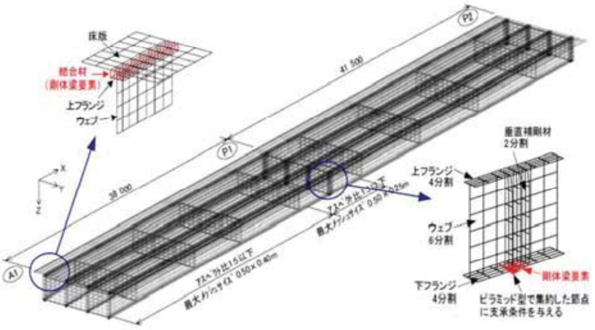


a cable-stayed bridge modeled by truss and beam elements

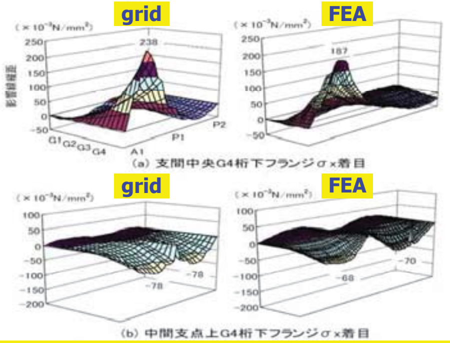
## Plane（2D）model
## （by truss and beam elements）



## Girder bridge modelled by Shell Element



## Ex. of influence line analysis



**3D influence volume (FEA vs GRID)**

---

If satisfied,

Check assumed steel weight and designed steel weight

If the difference is large (5% more)

Go to step [1],
{repeat until satisfied} ← basis of design

---

[3] safety check (stress check)

$\Sigma\sigma \leqq \sigma_a (= \sigma_{cr}/\gamma)$ γ : safety factor

$\Sigma\tau \leqq \tau_a (= \tau_{cr}/\gamma)$

$\sigma_e(\Sigma\sigma, \Sigma\tau) \leqq 1.1\sigma_a (= \sigma_y/\gamma)$

& deflection(δ) check

$\delta$(without impact) $\leqq \delta_a$

where, $\sigma_a$, $\tau_a$ and $\delta_a$ are allowable values

If not OK (not satisfied),
change section size and safety check.
{repeat until satisfied}

---

# Design method (Japan)

## - Allowable stress design method -

---

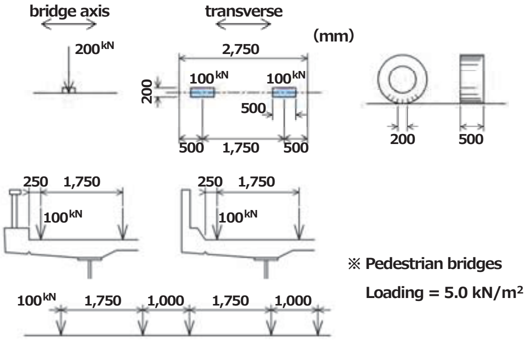
## Loading

（ ）: notation

| Primary Loading（P） | Sub Loading（S） |
|---|---|
| ・Dead load（D）<br>・Live load（L）<br>・Impact（I）<br>・Pre-stress force（PS）<br>・Concrete creep effect（CR）<br>・Concrete shrinkage effect（SH）<br>・Earth pressure（E）<br>・Hydraulic pressure（HP）<br>・Buoyancy or up-lift force（U） | ・Wind load（W）<br>・Temperature change（T）<br>・Earthquake（EQ） |
| **Special Loading Corresponding to load（PP）** | **Special Loading Corresponding to load（PA）** |
| ・Snow load（SW）<br>・Ground deformation（GD）<br>・Support settlement（SD）<br>・Wave pressure（WP）<br>・Centrifugal load（CF） | ・Braking force（BK）<br>・Erection load（ER）<br>・Collision load（CO）<br>・Others |

---

## For the design of Slabs and Floor systems（T-load）



※ Pedestrian bridges
Loading = 5.0 kN/m²

---

## Live Loading

1) Motor vehicle loading （T-load & L-load）
2) Pedestrian loading
3) Tram car （Surface car）

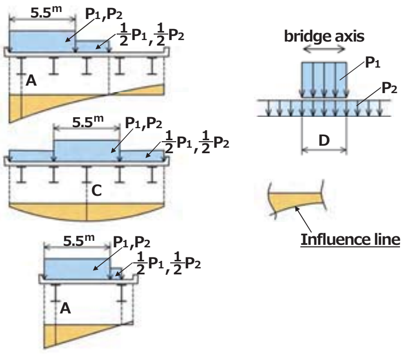
Depending on number of heavy vehicle volume , classified 【 A-live load and B-live load 】

B-live load : Running frequency of heavy trucks with a total weight of 245kN is large.

A-live load : the above frequency is small.

---

## For the design of main girder（L-load）



## Distributed-load intensity

| | Main loading | | | | | | sub loading |
|---|---|---|---|---|---|---|---|
| | $P_1$ (kN/m²) | | | $P_2$ (kN/m²) | | | |
| | loading length D (m) | For moment | For shear | L≦80 | 80＜L≦130 | 130＜L | |
| A-live load | 6 | 10 | 12 | 3.5 | 4.3−0.01L | 3.0 | 50% of main loading |
| B-live load | 10 | | | | | | |

L = span (m)

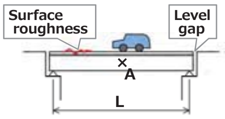
## Load intensity for pedestrian bridges

| | L≦80 | 80＜L≦130 | 130＜L |
|---|---|---|---|
| Load (kN/m²) | 3.5 | 4.3−0.01L | 3.0 |

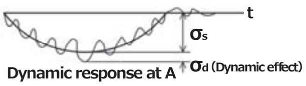
---

## Impact (i)

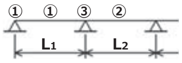
$i = \frac{20}{50+L}$ (for T-load and L-load ; steel bridges)



$\sigma = \sigma_s + \sigma_d = \sigma_s\ (1 + \underline{\ i\ })$

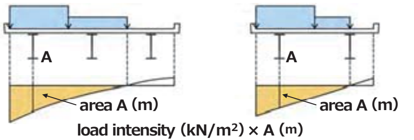


### Span (L) for Impact (i)

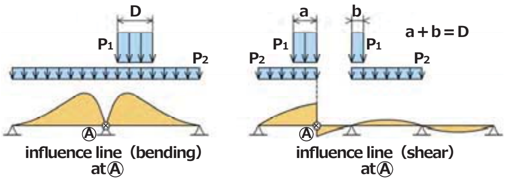
| | | |
|---|---|---|
| Continuous girder bridges (∗) |  | ① $L_1$<br>② $L_2$<br>③ ( $L_1$ + $L_2$ ) ／ 2 |
| Suspension bridges | Hanger | Span of floor system |
| Cable-stayed bridges | Main girder | (∗) |
| | Cable | Support in (∗) |

---

## Calculation of stress resultants (M, Q )



load intensity (kN/m²) × A (m)
→(kN/m) distributed load.



influence line (bending) atⒶ

influence line (shear) atⒶ

---

## Wind Load

$$P_W = \frac{1}{2}\rho V_d^2 C_d G A_W\ \ (N/m)$$

ρ : air density ( = 1.23 kg/m³)

$V_d$ : design wind velocity* ( = 40 m / s)

* mean value of random wind velocity.

$C_d$ : drag coefficient

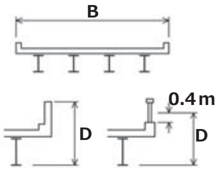
G : gust factor** ( = 1.9 )

** to take into account of effect of random wind velocity.

$A_W$ : projected area (m²/m)

### Wind load for girder bridges

| Section | Wind load (kN/m) |
|---|---|
| 1 ≦ B/D ＜ 8 | [4.0−0.2(B/D)] D ≧ 6.0 |
| 8 ≦ B/D | 2.4 D ≧ 6.0 |



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## Combination of loading and h* ( ≥ 1.0 )

* to take into account of probability of simultaneous loading

| Combination of loading | h |
|---|---|
| P + PP | 1.00 |
| P + PP + T | 1.15 |
| P + PP + W | 1.25 |
| P + PP + T + W | 1.35 |
| P + PP + BK | 1.25 |
| P + PP + CO | 1.70 (steel) |
| P (Excluding live load and impact) + EQ | 1.50 |
| W | 1.20 |
| BK | 1.20 |
| ER | 1.25 |

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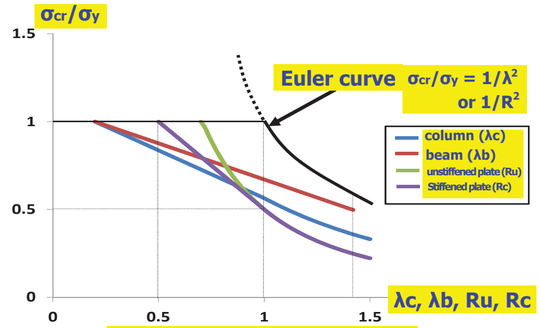
## Strength : $\sigma_{cr}/\sigma_y (\leq 1.0)$ by JHBS



$\lambda c, \lambda b, Ru, Rc = \sqrt{(\sigma y/\sigma E)}$

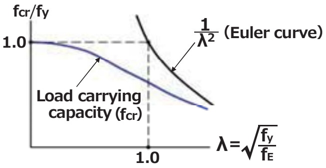
$\sigma_y$ : yield stress, $\sigma_E$ : elastic buckling stress

---

## [1] Safety check

$$\Sigma f_i < h \cdot f_a = h \cdot \min. \{ f_y , f_{cr} (\leq f_y ) \} / \underline{1.7}$$

f : stress ( σ (or) τ )
$f_a$ : allowable stress
$f_y$ : yield stress
$f_{cr}$ : buckling stress
1.7 : safety factor



$f_E$ : Elastic buckling stress

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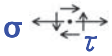
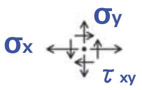
## In case of combination (σ, τ)

$$\sigma_e = \sqrt{\sigma^2 + 3\tau^2} \leqq 1.1 \cdot \sigma_a$$



### In case of combination ($\sigma_x$, $\sigma_y$, $\tau_{xy}$)

$$\sigma_e = \sqrt{\sigma_x^2 - \sigma_x\sigma_y + \sigma_y^2 + 3\tau^2} \leqq 1.1 \cdot \sigma_a$$



$\sigma_e$ : equivalent stress
1.1 : used in Japanese practice

# Material grade and allowable stress

(N/mm²)

| | SS400<br>SM400<br>SMA400W | SM490 | SM490Y<br>SM520<br>SMA490W | SM570<br>SMA570W |
|---|---|---|---|---|
| (mm)<br>t ≦ 40<br>t : thickness | 140<br>80<br>(235) | 185<br>105<br>(315) | 210<br>120<br>(355) | 255<br>145<br>(450) |
| 40 ＜t ≦ 75 | 125<br>75<br>(215) | 175<br>100<br>(295) | 195<br>115<br>(335) | 245<br>140<br>(430) |
| 75 ＜t ≦ 100 | | | 190<br>110<br>(325) | 240<br>135<br>(420) |

| | |
|---|---|
| SSXXX | : Structural steel |
| SMXXX | : Structural steel (for welding) |
| SMAXXXW | : Structural steel (for welding & Weathering) |
| XXX | : Min. tensile strength |

Axial and bending tensile stress
Shear stress
(Yield stress)

# [2] Deflection check

| Allowable deflection | | | Simple (&) continuous girders | Cantilevered part in Gerber girders |
|---|---|---|---|---|
| Steel girder bridges | (with) Concrete deck | L≦10 | L/20,000 | L/12,000 |
| | | 10＜L≦40 | L/ (20,000/ L) | L/ (12,000/ L) |
| | | 40＜L | L/500 | L/300 |
| | (with) Other types of deck | | L/500 | L/300 |
| Suspension bridges | | | L/350 | |
| Cable-stayed bridges | | | L/400 | |
| Other types | | | L/600 | L/400 |

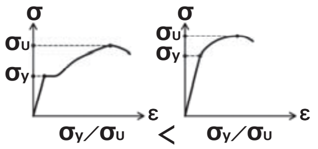
L = span (m)

# Safety factor

Safety factor（ γ ≒ 1.7 ）is used.

| | SS400<br>SM400<br>SMA400W | SM490 | SM490Y<br>SM520<br>SMA490W | SM570<br>SMA570W |
|---|---|---|---|---|
| (mm) t ≦ 40 | 1.68 | 1.70 | 1.69 | 1.76* |
| 40 ＜t ≦ 75 | 1.72 | 1.69 | 1.72 | 1.76* |
| 75 ＜t ≦ 100 | | | 1.71 | 1.75* |

＊ $\sigma_y / \sigma_U$ is relatively high, $\gamma$ = 1.75,1.76 is used.



（$\sigma_y / \sigma_U$）: Yield ratio

$$\Sigma\sigma_i \leqq h \cdot \sigma_{cr}/v \quad (v ≒ 1.7)$$

↓

Ex. (limit state 3)

$$\xi_1 \cdot \xi_2 \cdot (\Sigma\gamma_i\sigma_i) \leqq \Phi_{ut} \cdot \sigma_{cr}$$

h (≧1.0) : coefficient to take into account of probability of simultaneous occurrence of multiple load actions
v : safety factor
$\xi_1$ : analysis factor
$\xi_2$ : member (or) structural factor
$\gamma_i$ : load factor
$\Phi_{ut}$ : resistance factor

# AASHTO LRFD and Euro Code (EC)

## [Required performance]

- Safety
- Serviceability
- Constructability

・・・・・・・・

## [Limit State]

- Ultimate(Strength)Limit
- Serviceability Limit
- Fatigue Limit

・・・・・・・・

## 【Design Method】

- Performance-based Design Method
- Limit State Design Method

Required performance and its level for structures are defined.

whether or not the required performance level is satisfied

## 【Check Method】

- Load Resistance Factor Design Method (LRFD)
- Partial Factor Design Method (PFD)
- Allowable Stress Design Method (ASD)

# Design Level

- Level- I «Standard»

  Partial factor is used

  «$S^* \leqq R^*$» ($S^*$,$R^*$):factored action & resistance

- Level-Ⅱ

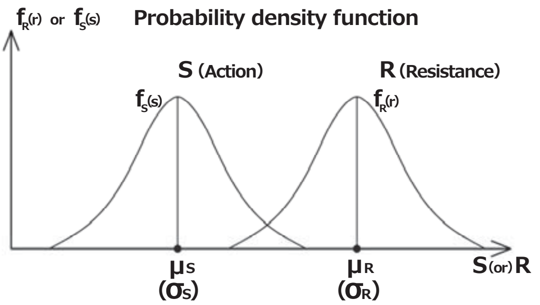
  Safety index(β) is used

  «$\beta \geqq \beta_{target}$»

- Level-Ⅲ

  Failure probability($P_f$) is used

  «$P_f \leqq P_{f,\ target}$»



μ : mean value

σ : variance

## (Performance-based Design Method)

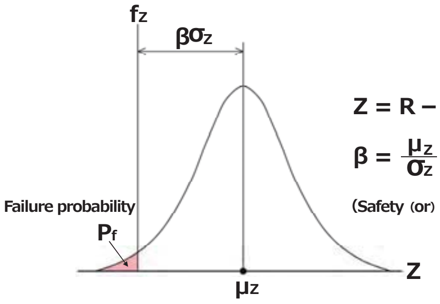
In order to attain the aim of the structures,
require performance is defined.

**Normally,**

**the following check is given**

$$\beta \geqq \beta_{TARGET}$$

[β : safety or reliability index]



$Z = R - S$

$\beta = \dfrac{\mu_Z}{\sigma_Z}$

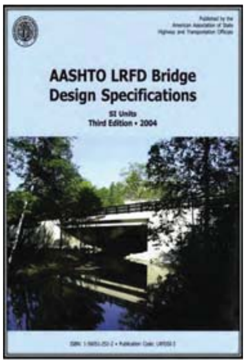
(Safety (or) reliability index)

$\mu_Z = \mu_R - \mu_S$

$\sigma_Z = \sqrt{\sigma_R^2 + \sigma_S^2}$

| | | | | | |
|---|---|---|---|---|---|
| $P_f$ | 0.5 | 0.16 | $10^{-2}$ | $10^{-3}$ | $10^{-4}$ |
| β | 0.0 | 1.00 | 2.33 | 3.10 | 3.72 |

# AASHTO LRFD



**【ASD】**

$$\Sigma Q_i \leq \underline{R_E}/FS$$

Q : Load, $R_E$ : Elastic resistance, FS : safety factor

**【LFD】**

$$\Sigma \gamma_i Q_i \leq \underline{\varphi}R$$

γ : load factor, φ : reduction factor, R : resistance

「Ex.」 1.3(1.0D + 1.67L) = 1.3D + 2.17L ≤ φRn

**【LRFD】**

$$\Sigma \eta_i \gamma_i Q_i \leq \varphi R_n$$

# AASHTO

1931～ ASD(Allowable Stress Design Method)

1971～ LFD(Load Factor Design Method)

1988～ start developing design method based on reliability theory

2007 LRFD(Load Resistance Factor Design Method)

## AASHTO LRFD

for superstructure of bridges,

β = 3.5, $P_f$ = 2.33 x 10-4 (75-year design life)

## EC(target)

β = 3.8, $P_f$ = 7.23 x 10-5 (100-year design life)

(↑ ISO)

# Limit State to be checked

- <u>Strength Limit State Ⅰ～Ⅴ</u>

**Ex.** Ⅰ:vehicle running(no wind)

Ⅱ:allowed special type of vehicle (no wind)

- <u>Extreme Limit State Ⅰ, Ⅱ</u>

Ⅰ:erathquake load

Ⅱ:collisoion (ship, ice)

- <u>Serviceability Limit State Ⅰ～Ⅳ</u>

**Ex.** Ⅱ:Yielding of material

- <u>Fatigue Limit State Ⅰ(forever), Ⅱ(limited duration)</u>

[totally 13 cases are checked]

---

# Strength Limit State - Ⅰ

$$S\,1.25DC + 1.50DW + 1.75[LL + IM] \leqq S_{ult.}$$

S : Stress resultants

Sult. : Ultimate strength( = ΦRn {Rn : 公称強度})

DC : Dead load excluding (DW)

DW : Wearing surface [concrete pavement in USA]

LL + IM : Livre load (LL) including impact (IM)

# Serviceability Limit State - Ⅱ

$$f\,1.00D + 1.30[LL + IM] \leqq 0.95f_y$$

↑overload (heavy vehicle)

f : stress

fy : yield stress

---

# Check Format(LRFD)

$$\Sigma\eta_i\gamma_i Q_i \leqq \Phi R_n = R_r$$

$\gamma_i$ : load factor*

Φ: resistance factor*

$\eta_i$ :modification factor for load (= $\eta_D\eta_R\eta_I$)

[$\eta_D$ :ductility, $\eta_R$ : redundancy, $\eta_I$: importance]

Q : load effect

$R_n$ : (Nominal) resistance

$R_r$ : factored resistance

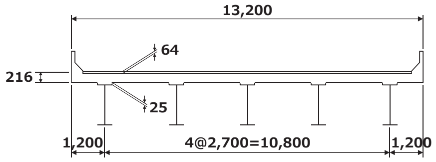
Regardless of the span length,
nearly the same safety index (β) is obtained.

---

# Example of Design by AASHTO LRFD

---

# Design of Simple Composite Girder (span = 35m)



| | | |
|---|---|---|
| Girder section | upper flange | 356 x 19(mm) |
| | web | 1,524 x 16 |
| | lower flange | 508 x 25 |
| Material grade | $f_y$ = 345MPa | |
| Concrete strength | $f_{ck}$ = 28MPa [28 days] | |

---

# Section classification

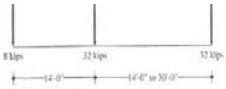
| | |
|---|---|
| compact | $M_P \leqq M_{ult.}$ |
| Non・compact | $M_y \leqq M_{ult.} \leqq M_P$ |
| Slender | $M_{ult.} \leqq M_y$ |

$M_P$ : plastic moment

$M_y$ : yield moment

---

# HL-93 (specified in 1993)



(a) Design truck load

a) 72kips(320kN truck)

(b) Design tandem load

b) [111.2kNx2] Tanadem load

{↑for short span bridge}

(c) Design lane load

c) 9.34kN/m lane loading

{↑excluding impact effect}

[a) + c)] or [b) + c)] Loading

---

# Compact, Noncompact & Slender sections

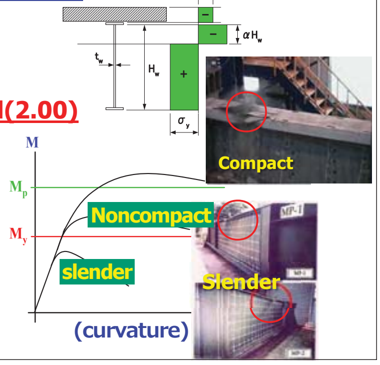
## Criteria for compact Sec.

$$\frac{H_w}{t_w} \leq \frac{2.00}{\alpha}\sqrt{\frac{E}{\sigma_y}} \quad (\alpha < 0.4)$$

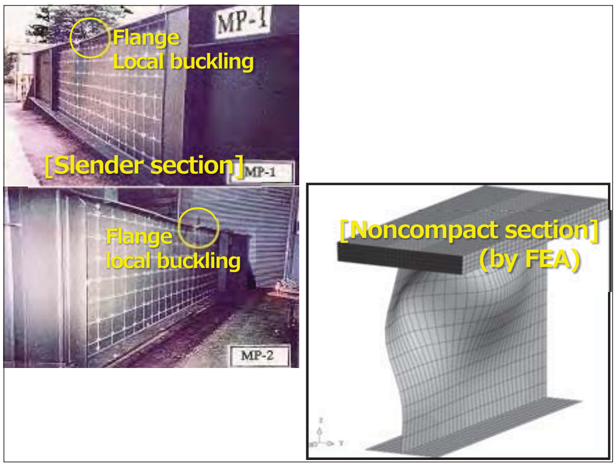
↑our proposal(2.00)

AASHTO LRFD : 1.88

EC ( Class 2 ) : 1.42



## Flexural Failure of Girder with Compact Section (under plus moment)



## Check of outer girder

[Strength Limit State Ⅰ (flexure and shear)]

$M = 1.25(\underline{2,119} + \underline{302.5}) + 1.50(\underline{388.9})$

↑DC ↑parapet ↑DW

$+1.75(\underline{2,961})$

↑live load

$= 8,792\text{kft} \leqq \underline{\Phi_f} \cdot \underline{M_n} = 10,973\text{kft}$

↑=1.0 ↑plastic strength

【action/desistance = 0.80】

Shear【action/resistance = 0.58】

[Serviceability Limit State Ⅱ (Lower flange)]

$f = 1.0(17.73 + 1.91)$

$+ 1.0(2.46) + 1.3(13.3)$

$= 44.3\text{kf/in2} \leqq$

$\Phi_b \cdot F_y = 0.95 \times 50 = 47.5\text{kf/in2}$

SI Unit $\{305\text{N/mm}^2 \leqq 327\text{N/mm}^2\}$

【action/resistance = 0.93】← controlled

## EN (Euro Norm)

**EN1990 :**
Basis of design
Required performance
(safety, serviceability, durability)

**EN1991 : Action (to structures)**

**EN1993～1996,1999 :**
design and structural detail

**EN1997,1998 : soil and earthquake**

## Euro Code [EC]

EC3 (steel bridges)
EC4-2 (steel-concrete composite bridges)

2010: Shifted with NAD*
*NAD : National Application Document

- Limit State Design (LSD)
- Partial factor method

## Limit State

### Ultimate Limit State(ULS)

- normal(including fatigue)
- construction
- accident
- earthquake(seismic)

### Serviceability Limit State LS(SLS)

- comfortability(deflection, vibration)
- appearance(cracking)
- functionality(elastic behavior)

## Design and Verification

| Design situations | Verification |
|---|---|
| Normal use | ULS, SLS |
| Transient such as, execution, maintenance repair | ULS, SLS |
| Accidental (including execution) | ULS |
| Seismic (including execution) | ULS, SLS |

ULS : Ultimate Limit State
SLS : Serviceability Limit State

## Basic action

$$\Sigma G_{k,j} + P + \Sigma Q_{k,i}$$

$G_{k,j}$ : Permanent load*
P : Prestress(←permanent load)
$Q_{k,I}$ : Variable load*

*characteristic value

---

## SLS(Serviceability Limit State)

{functionality, comfortability, appearance check}

1)Characteristic (apply to irreversible limit state)
- no plastic deformation [functionality]

2)Frequent (apply to reversible limit state)
- deflection (including impact) [comfortability]
- decompression and crack width in PC members

3)Quasi-permanent
- concrete crack width (long-term) [appearance]

---

## ULS(Ultimate Limit State)

{safety check}

EQU : Loss of static equilibrium
STR : Strength loss, excessive deformation
GEO : Failure or excessive deformation of ground
FAT : Fatigue failure

### [Action]

$$\Sigma \gamma_{G,j} G_{k,j} + \gamma_p P + \underline{\gamma_{Q,1} Q_{k,1}} + \underline{\Sigma \gamma_{Q,i} \Psi_{0,i} Q_{K,i}}$$

↑leading variable ↑accompanying variable

γ : partial factor

---

Leading action = $Q_{k,i}$ (i = 1)
Accompanying action is reduced using (Ψ)

| | |
|---|---|
| Combination* | $\Psi_0 Q_k$ |
| Frequent** | $\Psi_1 Q_k$ |
| Quasi-permanent*** | $\Psi_2 Q_k$ |

*probability of simultaneous occurrence
**exceed only short period of time
***exceed of considerable period of time

---

## [Action]

1)Characteristic SLS combination
{irreversible limit state}

$$\Sigma G_{k,j} + P + Q_{k,1} + \Sigma \psi_{0,i} Q_{k,i}$$

2)Frequent SLS combination
{reversible limit state}

$$\Sigma G_{k,j} + P + \Psi_{1,1} Q_{k,1} + \Sigma \Psi_{2,i} Q_{k,i}$$

3)Quasi-permanent SLS combination

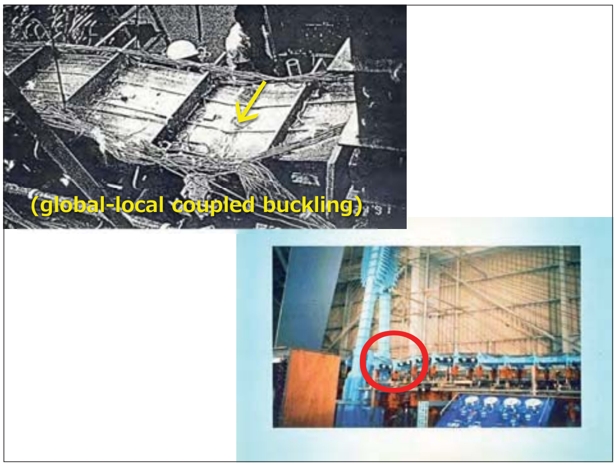
$$\Sigma G_{k,j} + P + \Sigma \Psi_{2,i} Q_{k,i}$$

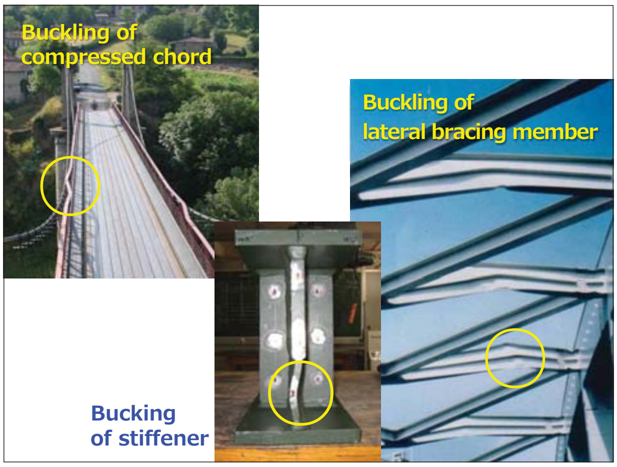
---

## Thank you for your kind attention

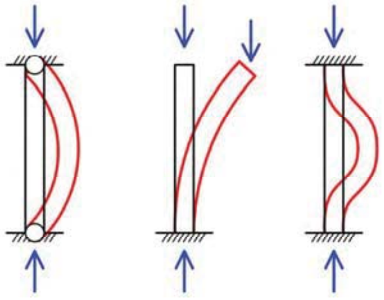
[11-3-1]

# Buckling of Columns & Strength Design

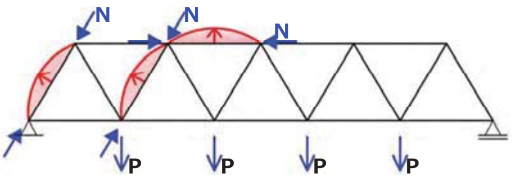




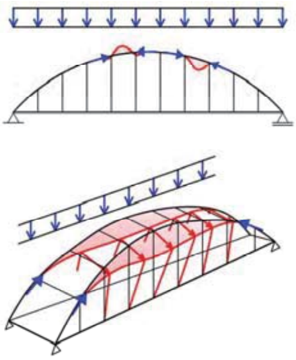
## Column Buckling

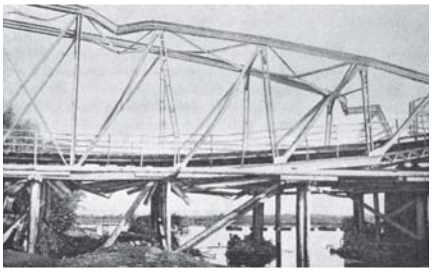


## Buckling of Truss Members

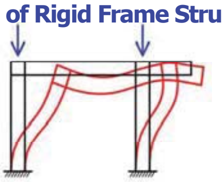


## Buckling of Arch Members

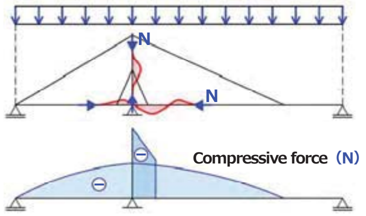


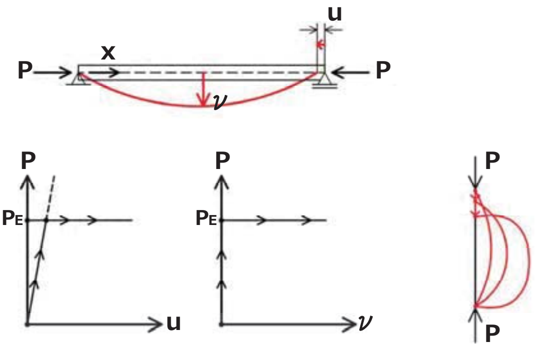


## Buckling of Rigid Frame Structures



## Buckling of Tower and Girder in Cable-stayed Bridge





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# Simple support [PIN-PIN]

at X=0 $\quad \nu = \frac{d^2\nu}{dx^2} = 0$ ——(1)

at X=L $\quad \nu = \frac{d^2\nu}{dx^2} = 0$ ——(2)

From eq. (1) $\quad$ B = D = 0

From eq. (2)

$$\begin{pmatrix} \sin\alpha L & L \\ \alpha^2 \sin\alpha L & 0 \end{pmatrix} \begin{Bmatrix} A \\ C \end{Bmatrix} = \begin{Bmatrix} 0 \\ 0 \end{Bmatrix} \text{——(3)}$$

To obtain no-zero $\{A,C\}^T$

$$\det \begin{vmatrix} \sin\alpha L & L \\ \alpha^2 \sin\alpha L & 0 \end{vmatrix} \text{——(4)}$$

$\alpha^2 L \sin\alpha L = 0 \longrightarrow \sin\alpha L = 0$ (Buckling equation) ——(5)

---

## Fundamental equation

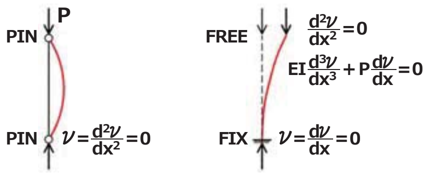
$$EI\frac{d^4\nu}{dx^4} + P\frac{d^2\nu}{dx^2} = 0$$

EI : Flexural rigidity of column

$$\alpha = \sqrt{P/EI}$$

$$\nu = A \sin\alpha x + B \cos\alpha x + Cx + D$$

## Boundary conditions



PIN: $\nu = \frac{d^2\nu}{dx^2} = 0$

FREE: $\frac{d^2\nu}{dx^2} = 0$, $EI\frac{d^3\nu}{dx^3} + P\frac{d\nu}{dx} = 0$

FIX: $\nu = \frac{d\nu}{dx} = 0$

---

$\alpha L = i\pi \longrightarrow \sqrt{\frac{P}{EI}} \cdot L = i\pi$ ——(6)

$$P_E = \left(\frac{i\pi}{L}\right)^2 EI \text{——(7)}$$

↑ Buckling load

i = 1 (min. value is important from practical viewpoint)

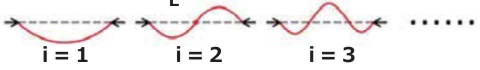
$$\min.P_E = \left(\frac{\pi}{L}\right)^2 EI \text{——(7)}'$$

Substituting eq. (5) to eq. (3)

C = 0

Buckling mode shapes are given

$$\nu = A \sin\frac{\pi}{L}x$$



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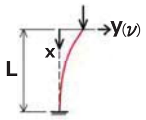
## 【FIX - FREE】



**Buckling equation**

$\cos\alpha L = 0$

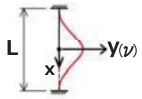
$\alpha L = \frac{2i-1}{2}\pi$ (i=1,2,⋯)

$\sqrt{\frac{P}{EI}} = \frac{2i-1}{2L}\pi$

$\min.P_E = \frac{1}{4}\left(\frac{\pi}{L}\right)^2 EI = \underline{\left(\frac{\pi}{2L}\right)^2 EI}$

$\nu = A\left(1 - \sin\frac{\pi}{2L}x\right)$

## 【 FIX - FIX 】



**Buckling equation**

$\underline{\sin\frac{\alpha L}{2} = 0}$ (or) $\tan\frac{\alpha L}{2} = \frac{\alpha L}{2}$

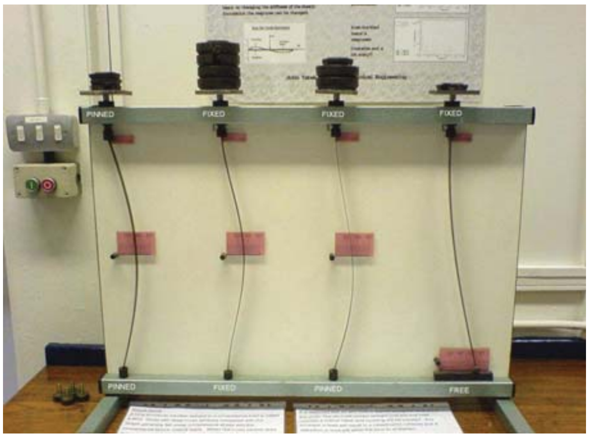
↑ (This eq. gives min. buckling load)

$\frac{\alpha L}{2} = i\pi$ (i=1,2,⋯)

$\min.P_E = 4\left(\frac{\pi}{L}\right)^2 EI = \underline{\left(\frac{\pi}{L/2}\right)^2 EI}$

$\nu = A\left(1 + \cos\frac{2\pi}{L}x\right)$

---



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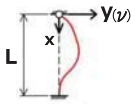
## 【PIN - FIX】



**Buckling equation**

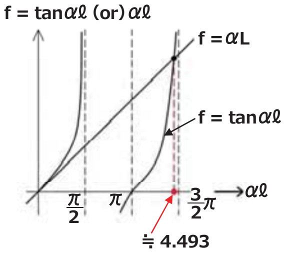
$\tan\alpha L = \alpha L$

$\alpha L \cong 4.493$ (←min. value)

$\alpha = \frac{4.493}{L}$

$\min.P_E = \left(\frac{4.493}{L}\right)^2 EI \cong \left(\frac{\pi}{0.7L}\right)^2 EI$

$\nu = A\left(\sin\alpha X - \frac{\sin\alpha L}{L}x\right)$



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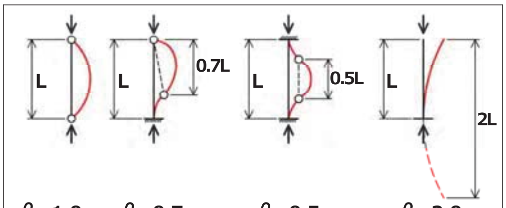
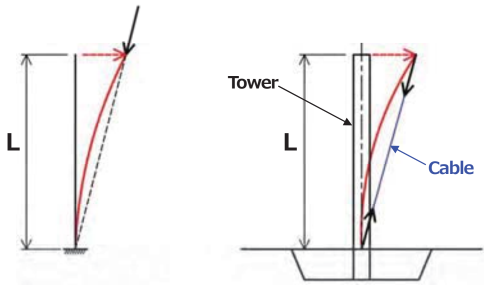
Le : Effective buckling length

$\beta$ : Buckling coefficient (Le = $\beta$L)



| | | | | |
|---|---|---|---|---|
| | $\beta$=1.0 | $\beta$=0.7 | $\beta$=0.5 | $\beta$=2.0 |
| $P_E$ | $\left(\frac{\pi}{L}\right)^2 EI$ | $\left(\frac{\pi}{0.7L}\right)^2 EI$ | $\left(\frac{\pi}{0.5L}\right)^2 EI$ | $\left(\frac{\pi}{2L}\right)^2 EI$ |

$$P_E = \left(\frac{\pi}{Le}\right)^2 EI = \left(\frac{\pi}{\beta L}\right)^2 EI$$

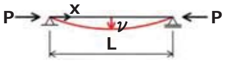


Le = L　　　Le = L

【Follower force】

## Energy method

### (Approximate estimation of buckling load)



- Strain energy of column (due to bending)

$$U = \frac{1}{2}\int_0^L EI\left(\frac{d^2\nu}{dx^2}\right)^2 dx \text{ ——— (1)}$$

- Potential energy from load

$$\nu = -\frac{1}{2}\int_0^L \left(\frac{d\nu}{dx}\right)^2 dx \text{ ——— (2)}$$

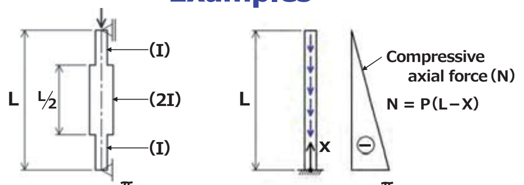
- Total potential energy of the system

$$\pi = U + \nu$$

$$\delta\pi = \delta(U+\nu) = 0 \quad (\leftarrow \text{Stationary condition}) \text{ ——— (3)}$$

Assuming the buckling mode shape satisfying the boundary conditions, and substitute it into eq. (3), we can get approximate buckling load.

## Examples



Assume $\nu = A\sin\frac{\pi}{L}x$ , $\nu = A\left(1-\cos\frac{\pi}{2L}x\right)$

(both satisfy the boundary conditions at ends)
Substituting assumed buckling mode shapes into eq.(3),and

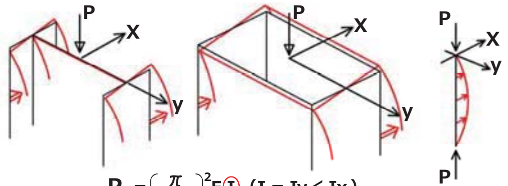
$$\frac{\partial\pi}{\partial A} = 0$$

$$P_E = \left(\frac{\pi}{0.74L}\right)^2 EI \qquad P_E = \left(\frac{\pi}{1.09L}\right)^2 EI$$

0.644 (correct value)　　1.22 (correct value)

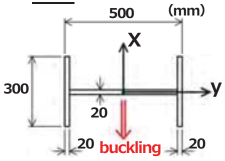
Assume $\nu = A_1\sin\frac{\pi}{L}x + A_2\sin\frac{3\pi}{L}x$, $\nu = A_1(1-\cos\frac{\pi}{2L}x) + A_2(1-\cos\frac{3\pi}{2L}x)$

More accurate buckling load will be obtained.



$$P_E = \left(\frac{\pi}{Le}\right)^2 E\,\text{Ⓘ} \quad (I = I_y < I_x)$$

**ex.**



$$I_x = 2\times30\times2\times24^2 + \frac{2\times46^3}{12} = 9\times10^4\,(cm^4)$$

$$I_y = 2\times\frac{2\times30^3}{12} + \frac{46\times2^3}{12} \cong 9\times10^3\,(cm^4)$$

$$I_y < I_x$$

【$A = 2\times30\times2 + 46\times2 = 2.12\times10^2\,(cm^2)$】

## Calculation of the buckling load

$(I = I_y = 9\times10^7\,mm^4)$

1) L = 5ᵐ (PIN−PIN)

$$P_E = \left(\frac{\pi}{Le}\right)^2 EI = \left(\frac{\pi}{5{,}000}\right)^2 \times \underset{(E)}{\underline{2\times10^5}}\times9\times10^7 = 7.0989\times10^6\,(N)$$

2) L = 10ᵐ (PIN−PIN)

$$P_E = \left(\frac{\pi}{Le}\right)^2 EI = \left(\frac{\pi}{10{,}000}\right)^2 \times 2\times10^5\times9\times10^7 = 1.7747\times10^6\,(N)$$

3) L = 10ᵐ (FIX−FIX)

$$P_E = \left(\frac{\pi}{Le}\right)^2 EI = \left(\frac{\pi}{0.5\times10{,}000}\right)^2 \times 2\times10^5\times9\times10^7 = 7.0989\times10^6\,(N)$$

4) L = 10ᵐ (FIX−FREE)

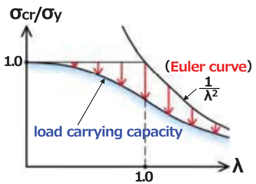
$$P_E = \left(\frac{\pi}{Le}\right)^2 EI = \left(\frac{\pi}{2\times10{,}000}\right)^2 \times 2\times10^5\times9\times10^7 = 4.4368\times10^5\,(N)$$

## Buckling stress (σE) and ultimate strength (σcr)

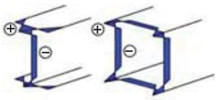
$$\sigma_E = \frac{P_E}{A} = \frac{E\pi^2}{(L_e/r)^2} \qquad r = \sqrt{\frac{I}{A}}$$

$$\frac{\sigma_E}{\sigma_y} = \frac{1}{\lambda^2} \qquad \lambda = \sqrt{\frac{\sigma_y}{\sigma_E}} = \frac{1}{\pi}\sqrt{\frac{\sigma_y}{E}}\cdot\frac{L_e}{r}$$

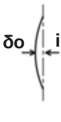
(← slenderness ratio)



$\sigma_{cr}/\sigma_y \sim \lambda$ curve



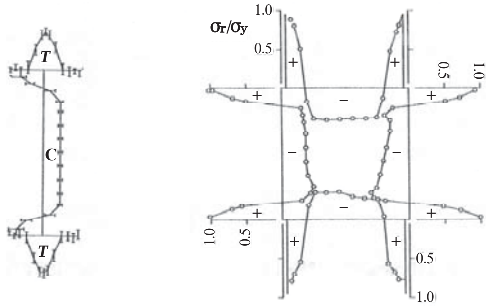
residual stress (σr)
due to welding



initial
deflection (δo)

(↓) reduction due to initial imperfection.
such as residual stress (σr) and
initial deflection (δo)

## Distribution of residual stress due to welding



I-section　　　Box-section

## Ultimate strength of columns (σcr) by JHBS

$$\frac{\sigma_{cr}}{\sigma_y} = 1.0 \qquad \lambda \leqq 0.2$$

$$\frac{\sigma_{cr}}{\sigma_y} = 1.109 - 0.547\lambda \qquad 0.2 < \lambda \leqq 1.0$$

$$\frac{\sigma_{cr}}{\sigma_y} = \frac{1.0}{0.773 + \lambda^2} \qquad 1.0 < \lambda$$

**ex.**

$L = 5^m$ [PIN-PIN] (SM400, $\sigma_y = 235\ {}^N/{}_{mm^2}$)

$P_E = \left(\frac{\pi}{Le}\right)^2 EI = 7.0989\times10^6\ {}^{(N)}$

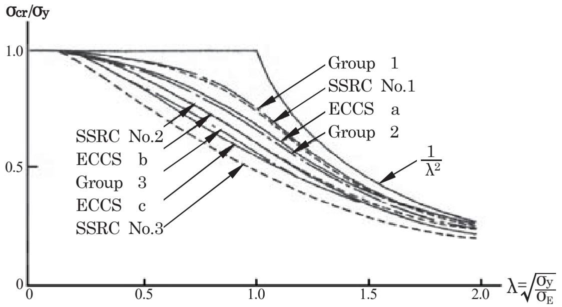
$\sigma_E = P_E/A = 7.0989\times10^6/2.12\times10^4 = 355.9\ {}^N/{}_{mm^2}$

$\lambda = \sqrt{\sigma_y/\sigma_E} = \sqrt{235/355.9} = 0.813\ (0.2 < \lambda \leqq 1.0)$

$\sigma_{cr}/\sigma_y = 1.109 - 0.547\lambda = 0.664 \rightarrow \underline{\sigma_{cr}} = 0.664\sigma_y = \underline{156.0\ {}^N/{}_{mm^2}}$

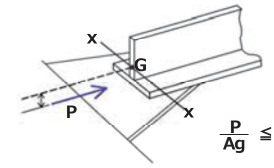
$\underline{\sigma_a\text{(allowable stress)}} \cong \sigma_{cr}/1.7 = \underline{91.8\ {}^N/{}_{mm^2}}$

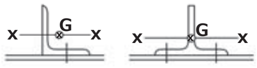
## Load carrying capacity curve for columns

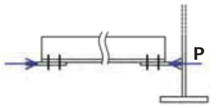


| | | |
|---|---|---|
| SSRC | No.1～3 | (USA) |
| ECCS | a～c | (Europa) |
| Group | 1～3 | (Fukumoto,Itoh) |

## Eccentricity of loading







$$\frac{P}{Ag} \leqq \sigma ca\left(0.5+\frac{Le/rx}{1{,}000}\right)$$

Ag : cross-sectional area (gross)

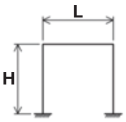
Le : effective buckling length

rx : radius of gyration of area (x-axis)

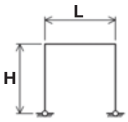
σca : allowable compressive stress

$$\sigma ca = \sigma cr(Le/rx)/1.7$$

## Rigid frame (Rahmen) structures



$He = 1.5H$
$=\{1.5-0.04(k\text{-}5)\}H$

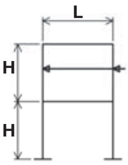


$He = 3.5H$
$=\{3.5-0.2(k\text{-}5)\}H$

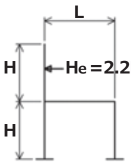
He : effective buckling length
(He = upper value (k ≦ 5)
Lower value (5 < k ≦ 10))

$$k=\frac{I_C/H}{I_B/L}$$

$I_C$ : geometrical moment of inertia of column
$I_B$ : geometrical moment of inertia of beam

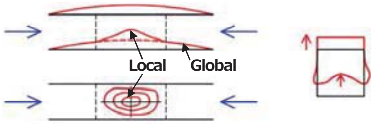


$He = 1.9H$
$=\{1.9-0.14(k\text{-}5)\}H$



$He = 2.2H$

## Global (column) and local (plate) coupled buckling strength



【JHBS , Product formula】

$$\sigma cr = \sigma cr(column)\cdot\frac{\sigma cr(plate)}{\sigma y}$$

σcr (column): column buckling strength excluding the effect of plate buckling
σcr ( plate ): plate buckling strength

【American practice* , Q-factor method】

$$\sigma cr = f(\overline{\lambda}_i)\cdot(Q\cdot\sigma y)$$

Q·σy : plate buckling strength (= σcr (plate))

$$\overline{\lambda}_i=\sqrt{\frac{Q\cdot\sigma y}{\sigma_E}}$$

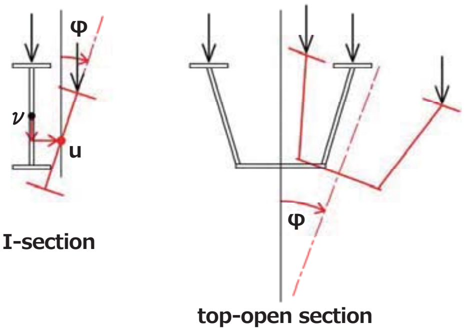
$f(\overline{\lambda}_i)$: column buckling strength

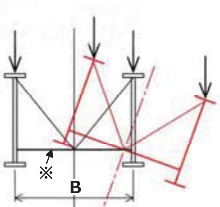
* American Institute of Steel Construction

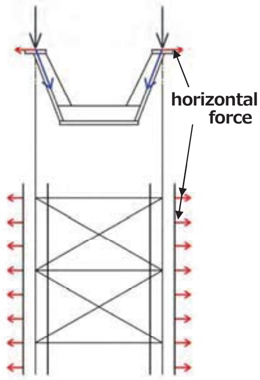
[11-3-2]

# Buckling of Beam & Beam-column & Strength Design

## Lateral Torsional Buckling



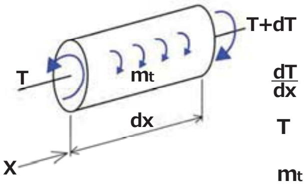




※ lower lateral bracing

When, L/B > 18
check should be made
(L : span)

## Member under torsion



$$\frac{dT}{dx} + m_t = 0$$

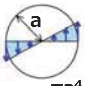
T : torsional moment (torque)

$m_t$ : torsional load

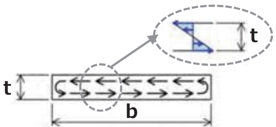
$T = T_w + T_s$

$T_w$ : Warping torsion

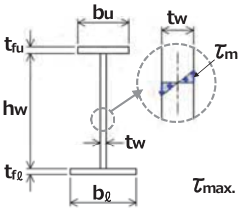
$T_s$ : St.Venant (or Pure) torsion



$$J = \frac{\pi a^4}{2}$$



$$J = \frac{1}{3} bt^3$$

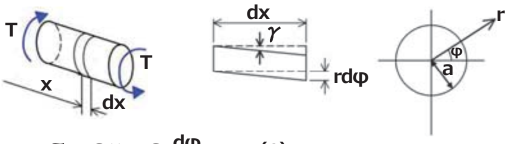


$$J = \Sigma \frac{1}{3}(bt^3)$$

$$= \frac{1}{3}(b_u t^3{}_{fu} + b_\ell t^3{}_{f\ell} + h_w t^3{}_w)$$

$$\tau_{max.} = \frac{T}{J}\left(\frac{t}{2}\right)$$

## Pure torsion of round bar



$$\tau = G\gamma = Gr\frac{d\varphi}{dx} \text{ ——— (1)}$$

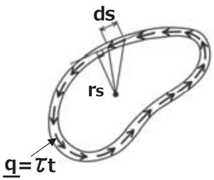
$$T = \int_A (\tau dA)\cdot r = G \overset{I_p}{\boxed{\int_A r^2 dA}} \cdot \frac{d\varphi}{dx} = GJ\frac{d\varphi}{dx} \text{ ——— (2)}$$

J : St.Venant torsional constant

GJ : St.Venant torsional rigidity

$$\tau = \frac{T}{J}\cdot r \text{ ——— (3)}$$

## Pure torsion of tubular structures



$\underline{q} = \tau t$
shear flow (constant)

$$T_s = \oint q r_s ds \left(\oint r_s ds = 2\oint dA = 2A\right)$$

$$= 2qA$$

$$q = \frac{T}{2A}\left(\rightarrow \tau = \frac{T}{2At}\right)$$

Displacement in member axis

$$du = \underbrace{-r_s\frac{d\varphi}{dx}ds}_{\text{(rotation)}} + \underbrace{\frac{\tau}{G}ds}_{\text{(shear strain)}} = -\left(r_s\frac{d\varphi}{dx} - \frac{\tau}{G}\right)ds$$

Since, $\oint du = 0$

$$-\oint r_s \frac{d\varphi}{dx} ds + \oint \frac{\tau}{G} ds = 0$$

$$2A \frac{d\varphi}{dx} = q \oint \frac{ds}{Gt}$$

$$\frac{d\varphi}{dx} = \frac{q}{2GA} \oint \frac{ds}{t}$$

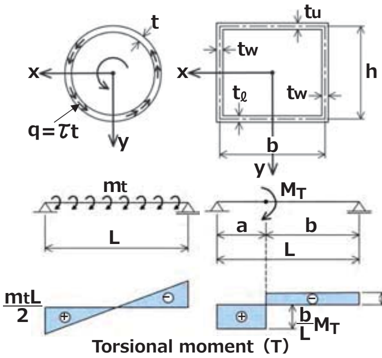
$$\Big\downarrow \quad \left(q = \frac{T_s}{2A}\right)$$

$$\frac{d\varphi}{dx} = \frac{T_s}{G(2A)^2} \oint \frac{ds}{t} \quad \leftarrow \text{(Bredt Batho formula)}$$

$$\Big\downarrow \quad \left(T_s = GJ\frac{d\varphi}{dx}\right)$$

$$J = \frac{(2A)^2}{\oint \frac{ds}{t}}$$

## Pure torsion（T=Ts）

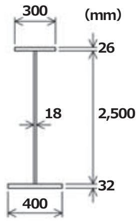


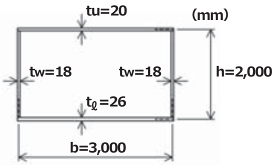
$A = bh$

$$J = \frac{(2A)^2}{\oint \frac{ds}{t}} = \frac{4b^2h^2}{\frac{b}{t_u} + \frac{b}{t_\ell} + 2\frac{h}{t_w}}$$

$$\tau_s = \frac{q}{t} = \frac{T}{2At} = \frac{T}{2bht}$$

## ex.





$$J = \frac{(2A)^2}{\oint \frac{ds}{t}} \quad \frac{(2bh)^2}{\frac{b}{t_u} + \frac{b}{t_\ell} + 2\frac{h}{t_w}}$$

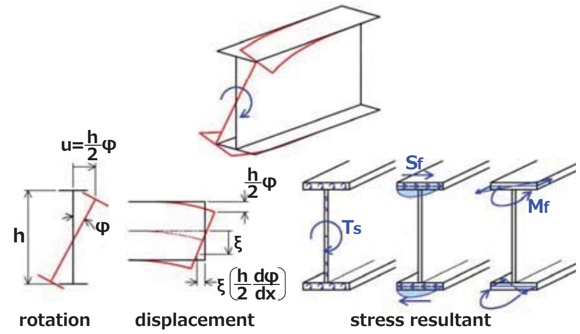
$$J = \frac{1}{3}(300\times26^3 + 400\times32^3 + 2{,}500\times18^3)$$

$$= \underline{3{,}296\times10^7\,(\mathrm{mm}^4)}$$

$$J = \frac{(2\times3{,}000\times2{,}000A)^2}{\frac{3{,}000}{20} + \frac{3{,}000}{26} + 2\frac{2{,}000}{18}}$$

$$= \underline{2{,}953\times10^{11}\,(\mathrm{mm}^4)}$$

## Torsion of open section



rotation displacement stress resultant

$$T_w = S_f \cdot h$$

$$\Big\downarrow \quad \left\{ S_f = -EI_f \frac{d^3u}{dx^3} = -EI_f \left(\frac{h}{2}\right) \frac{d^3\varphi}{dx^3} \right\}$$

$$T_w = -EI_f \left(\frac{h^2}{2}\right) \frac{d^3\varphi}{dx^3} = -EI_w \frac{d^3\varphi}{dx^3}$$

$I_w \left(= I_f \left(\frac{h^2}{2}\right)\right)$ : warping constant

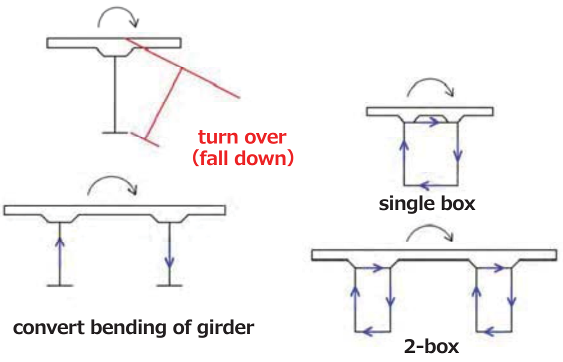
$$T_s = GJ\frac{d\varphi}{dx}$$

$$T = T_w + T_s \quad (\&) \quad \frac{dT}{dx} + m_t = 0$$

$$\underline{EI_w \frac{d^4\varphi}{dx^4} - GJ\frac{d^2\varphi}{dx^2} = m_t} \quad \text{(for open section)}$$

when EIw is small compared to GJ

$$\underline{GJ\frac{d^2\varphi}{dx^2} = -m_t} \quad \text{(for closed section)}$$



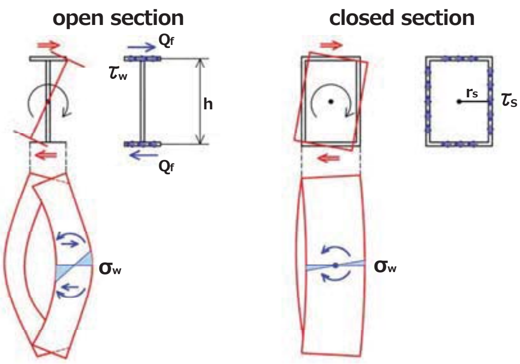
turn over
(fall down)

single box

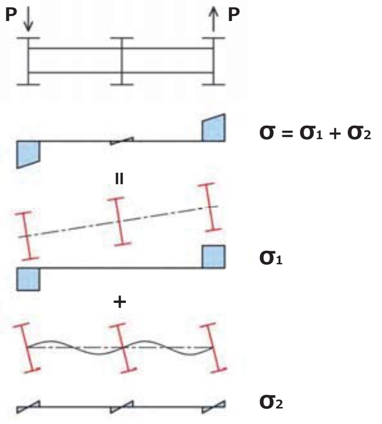
convert bending of girder

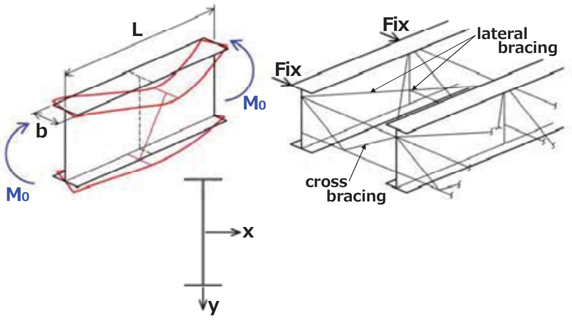
2-box

## Behavior under torsion



open section closed section





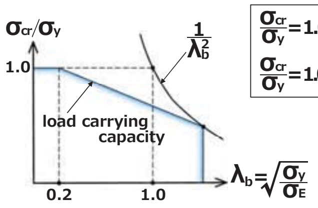
$$(M_0)_E = \left(\frac{\pi}{L}\right)\sqrt{EI_yGJ\left(1+\frac{\pi^2EI_w}{L^2GJ}\right)}$$

---

$$\sigma_E = \frac{\pi^2E}{4}\,\frac{1}{\left\{K\left(\frac{L}{b}\right)\right\}^2}$$

$$K = \sqrt{3+\frac{A_W}{2A_F}}$$

$$\underline{\lambda_b} = \sqrt{\frac{\sigma_y}{\sigma_E}} = \underline{\frac{2}{\pi}K\sqrt{\frac{\sigma_y}{E}}\cdot\left(\frac{L}{b}\right)}$$



$$\frac{\sigma_{cr}}{\sigma_y} = 1.0 \qquad \lambda \leqq 0.2$$

$$\frac{\sigma_{cr}}{\sigma_y} = 1.0-0.412(\lambda_b-0.2) \quad 0.2<\lambda<\sqrt{2}$$

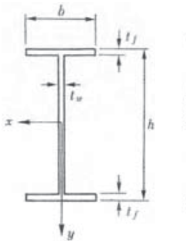
$$\left(\begin{array}{ll} A_W/A_F \leqq 2 & K=2 \\ A_W/A_F > 2 & \\ & K=\sqrt{3+\frac{A_W}{2A_F}} \end{array}\right)$$

---

$$\sigma_E = \frac{(M_0)_E}{W} = \sqrt{\frac{\pi^2EI_yGJ}{L^2W^2}+\frac{\pi^4E^2I_yI_w}{L^4W^2}} = \sqrt{\sigma_v{}^2+\sigma_w{}^2}$$

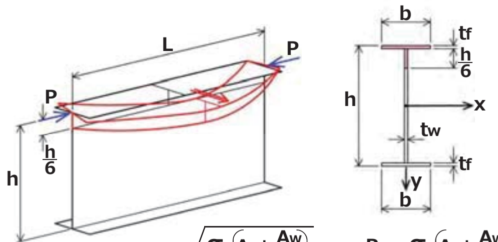
from $\sigma_w \gg \sigma_v$ , $\sigma_E \cong \sigma_w$

$$\sigma_E = \sqrt{\frac{\pi^4E^2I_yI_w}{L^4W^2}} = \frac{\pi^2E\sqrt{I_yI_w}}{L^2W}$$



$$A = 2bt_f + (h-2t_f)\,t_w$$
$$= 2A_F + A_W$$
$$I_y \fallingdotseq A_Fb^2/6$$
$$I_w \fallingdotseq I_yh^2/4$$
$$J = \frac{A_F}{3}(2t_f{}^2 + A_Wt_w{}^2/A_F)$$
$$W = A_Fh(1+A_W/6A_F)$$

---



$$\underline{\lambda} = \sqrt{\frac{P_y}{P_E}} = \sqrt{\frac{\sigma_y\left(A_F+\frac{A_W}{6}\right)}{\left(\frac{\pi}{L}\right)^2E\,\frac{b^2A_F}{12}}}$$

$$= \underline{\frac{2}{\pi}K\sqrt{\frac{\sigma_y}{E}}\cdot\left(\frac{L}{b}\right)}$$

$$K = \sqrt{3+\frac{A_W}{2A_F}}$$

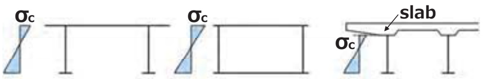
the same as $\lambda_b$

$$P_y = \sigma_y\left(A_F+\frac{A_W}{6}\right)$$

$$P_E = \left(\frac{\pi}{L}\right)^2E\,\frac{b^2A_F}{12}$$

---

## No lateral torsional buckling



π-section box-section I-girder with slab

Allowable compressive stress in flexure ($\sigma_{ba}$)

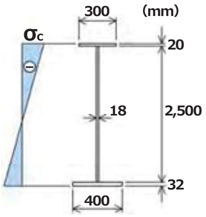
(N/mm²)

| | SS400<br>SM400<br>SMA400W | SM490 | SM490Y<br>SM520<br>SMA490W | SM570<br>SMA570W |
|---|---|---|---|---|
| t≦40 | 140 | 185 | 210 | 255 |
| 40 <t≦75 | 125 | 175 | 195 | 245 |
| 75 <t≦100 | | | 190 | 240 |

$$\sigma_{ba} = \left\{\underline{\sigma_{cr}(=\sigma_y)}/(\gamma \cong 1.7)\right\}$$

---

## ex.



$L = 5{,}000$ mm

Material grad SM400 ($\sigma_y = 235$ N/mm²)

$$K = \sqrt{3+\frac{A_W}{2A_C}} = \sqrt{3+\frac{2{,}500\times18}{2\times300\times20}} = 2.60$$

$$\lambda_b = \frac{2}{\pi}\times2.60\times\sqrt{\frac{235}{2\times10^5}}\times\left(\frac{5{,}000}{300}\right) = 0.946$$

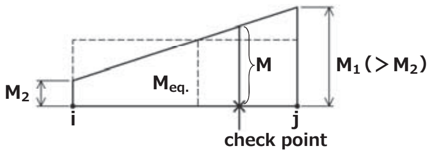
$$\frac{\sigma_{cr}}{\sigma_y} = 1.0-0.412(\lambda_b-0.2) = 0.693$$

$\sigma_{cr} = 0.693\times235 = 162.9$ (N/mm²)

$\sigma_{ba} = \sigma_{cr}/1.7 = 95.8$ (N/mm²)

---

## In case of ununiform bending moment



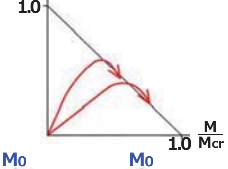
$M_{eq.} = \min.\{(0.6M_1 + 0.4M_2)$ or $0.4M_1\}$

allowable stress can be increased
by multiplying ($M/M_{eq.}$),

however not exceeding ($\sigma_y/\gamma$)

---

## Bending moment (M) and compressive force (P)



$$\frac{P}{P_{cr}} + \frac{M^*}{M_{cr}} < 1.0$$

$$\left(M^* = M\frac{1}{1-P/P_E}\right)$$

$$\underline{\frac{P}{P_{cr}} + \frac{M}{M_{cr}(1-P/P_E)} < 1.0}$$

$P_E$ : elastic buckling load

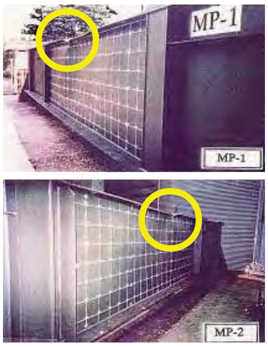
$$M_{max.} = M_0\sec\frac{\alpha\ell}{2}, \quad \alpha = \sqrt{\frac{P}{EI}}$$

$$\cos\frac{\alpha\ell}{2} = 1-P/P_E \quad \Rightarrow \quad M_{max.} = M_0\frac{1}{(1-P/P_E)}$$

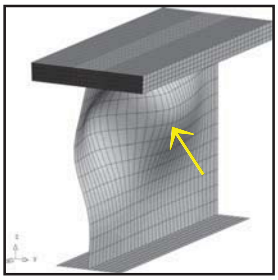
[11-3-3]

# Buckling of Plate & Strength Design

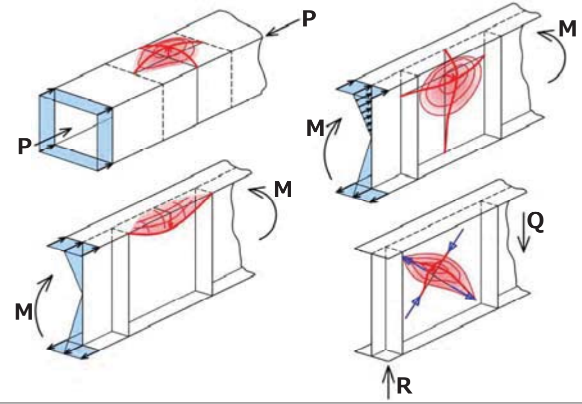
## Local buckling of compressed plate



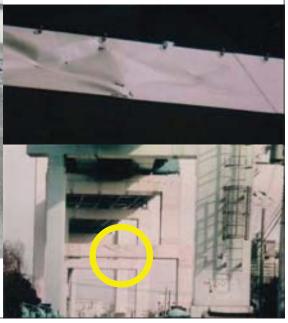
## Local buckling of web

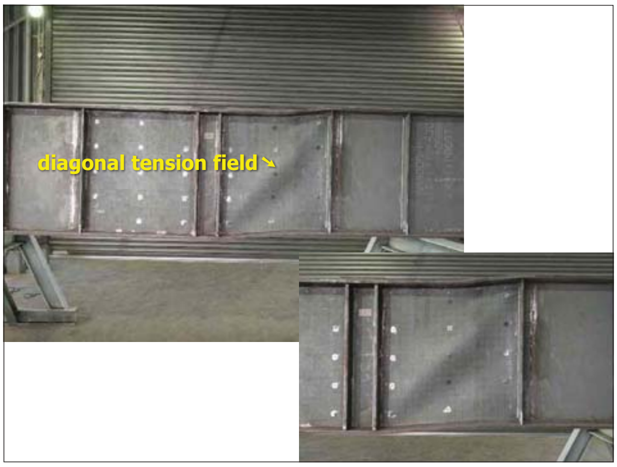


## Buckling of Plates

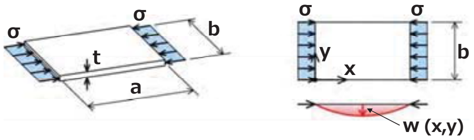


## Shear buckling





## Fundamental equation



$$B\left(\frac{\partial^4 w}{\partial x^4}+2\frac{\partial^4 w}{\partial x^2\partial y^2}+\frac{\partial^4 w}{\partial y^4}\right)+(\sigma t)\frac{\partial^2 w}{\partial x^2}=0 \quad\text{——(1)}$$

$$B=\frac{Et^3}{12(1-\nu^3)}$$

$$\text{Assuming, } w(x,y)=\underbrace{A_{m,n}\sin\frac{m\pi}{a}x\cdot\sin\frac{n\pi}{b}y}_{(m,\,n=1,2,3\cdots\cdots)} \quad\text{——(2)}$$

↑ satisfying boundary condition

Substituting eq.（2）into eq.（1）,

# Buckling of unstiffened plates

$$\sigma_E=\frac{\pi^2 B}{b^2 t}\left(m\frac{b}{a}+\frac{n^2}{m}\frac{a}{b}\right)^2$$

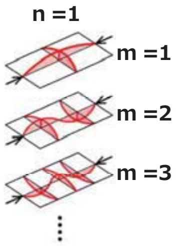
min. value is obtained when $\underline{n=1}$ , and setting $\underline{\alpha=a/b}$ ($\alpha$: aspect ratio)

$$\sigma_E=\frac{\pi^2 B}{b^2 t}\left(\frac{m}{\alpha}+\frac{\alpha}{m}\right)^2$$

We rewrite

$$\sigma_E=k\underbrace{\boxed{\frac{\pi^2 E}{12(1-\nu^2)}\left(\frac{t}{b}\right)^2}}_{=\sigma_{E0}}=k\cdot\sigma_{E0}$$

$$\boxed{k=\left(\frac{m}{\alpha}+\frac{\alpha}{m}\right)^2}\quad(m=1,2,3\cdots\cdots)$$



k : buckling coefficient of plate

（a/b= 2）

---

# Examples

**ex. 1**

$k_{min.}=4.0$

【t=10mm】

$$\sigma_{E0}=\frac{\pi^2E}{12(1-\nu^2)}\left(\frac{t}{b}\right)^2$$
$$=1.8058\times10^5\times\left(\frac{t}{b}\right)^2$$
$$=1.8058\times10^5\times\left(\frac{10}{400}\right)^2$$
$$=112.9\,(\mathrm{N/mm^2})$$
$$\sigma_E=4\times\sigma_{E0}=451.4\,(\mathrm{N/mm^2})$$

【t=6mm】

$$\sigma_{E0}=40.6\,(\mathrm{N/mm^2})$$
$$\sigma_E=4\times\sigma_{E0}=162.6\,(\mathrm{N/mm^2})$$

**ex. 2**

$k_{min.}=0.426$

【t=25mm】

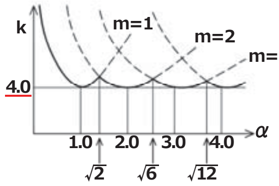
$$\sigma_{E0}=1.8058\times10^5\times\left(\frac{25}{300}\right)^2$$
$$=1254.0\,(\mathrm{N/mm^2})$$
$$\sigma_E=0.426\times\sigma_{E0}=534.2\,(\mathrm{N/mm^2})$$

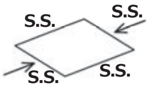
【t=15mm】

$$\sigma_{E0}=1.8058\times10^5\times\left(\frac{15}{300}\right)^2=451.5\,(\mathrm{N/mm^2})$$
$$\sigma_E=0.426\times\sigma_{E0}=192.3\,(\mathrm{N/mm^2})$$

$$\left(\begin{array}{l}E=2\times10^5\,(\mathrm{N/mm^2})\\ \nu=0.3\end{array}\right)$$

---

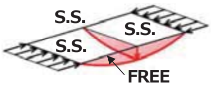




S.S. : Simply Supported

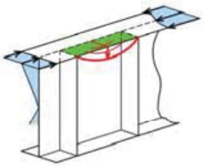
$$\left(w=0,\ \frac{\partial^2w}{\partial x^2}=\frac{\partial^2w}{\partial y^2}=0\right)$$

$k_{min.}=4.0$



$$k=\frac{1}{\alpha^2}+0.426$$

$\alpha\rightarrow\infty$ $k_{min.}=0.426$



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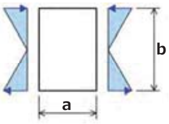
# K value depending on stress condition

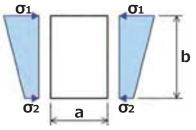
$(\sigma_E=k\sigma_{E0})$



$\alpha=\dfrac{a}{b}$

$2/3<\alpha$ $K\cong23.9$

$\alpha<2/3$ $K\cong15.87+1.87\alpha^2+8.6/\alpha^2$

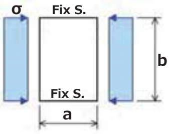


$\phi=\dfrac{\sigma_1-\sigma_2}{\sigma_1}$

$K=\dfrac{8.4}{2.1-\phi}$ $0\leqq\phi\leqq1$

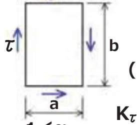
$K=10\phi-13.73\phi-11.36$ $1<\phi\leqq2$

$\uparrow\alpha>1.0$



$2/3<\alpha$ $K\cong7.0$

$\alpha<2/3$ $K\cong2.366+5.3\alpha^2+\dfrac{1}{\alpha^2}$



$(\tau_E=k_\tau\sigma_{E0})$

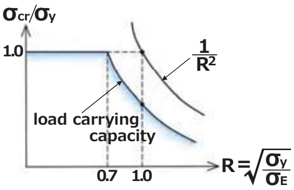
$1<\alpha$ $K_\tau=5.34+\dfrac{4}{\alpha^2}$

$\alpha<1$ $K_\tau=4.00+\dfrac{5.34}{\alpha^2}$

---

# Ultimate strength of unstiffened plate by JHBS

$$R=\sqrt{\frac{\sigma_y}{\sigma_E}}=\sqrt{\frac{\sigma_y}{E}\cdot\frac{12(1-\nu^2)}{\pi^2k}}\cdot\left(\frac{b}{t}\right)$$

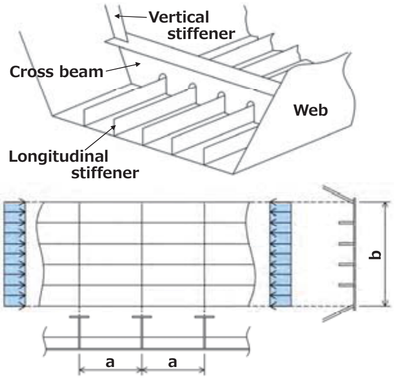


$\dfrac{\sigma_{cr}}{\sigma_y}=1.0$ $R\leqq0.7$

$\dfrac{\sigma_{cr}}{\sigma_y}=0.5/R^2$ $0.7<R$

---

# Stiffened Plate



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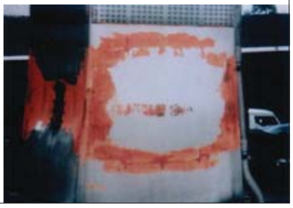
# Buckling of stiffened plate

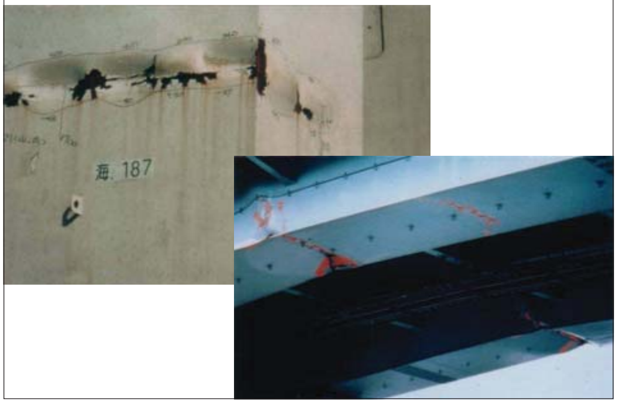
---

# Global buckling of stiffened plates



## Local buckling of stiffened plates



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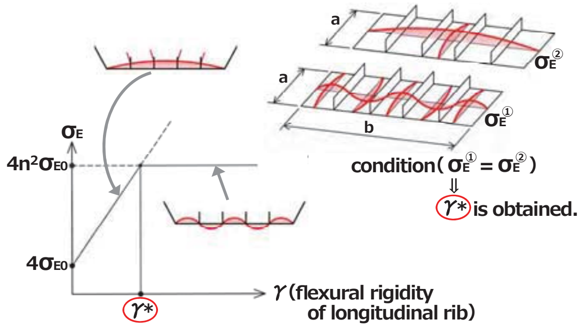
## {Local elastic buckling stress} = {Global elastic buckling stress}



condition( $\sigma_E^{①} = \sigma_E^{②}$ )

⇓

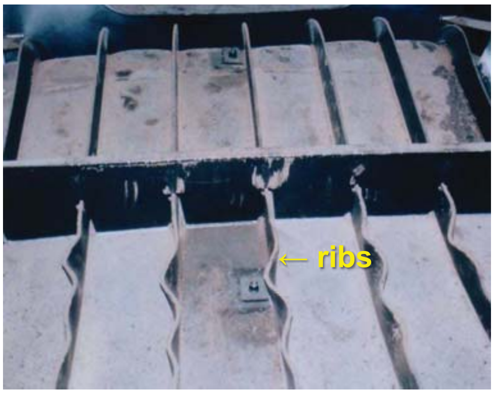
$\gamma^*$ is obtained.

n : number of panel

n-1 : number of longitudinal ribs

---

## Buckling of longitudinal ribs

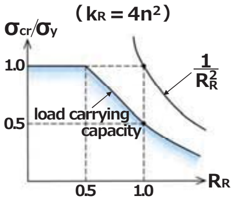


---

## Ultimate strength of stiffened plate by JHBS

$$R_R = \sqrt{\frac{\sigma_y}{\sigma_E}} = \sqrt{\frac{\sigma_y}{E} \cdot \frac{12(1-\nu^2)}{\pi^2 k_R}} \cdot \left(\frac{b}{t}\right)$$

( $k_R = 4n^2$ )



$\frac{\sigma_{cr}}{\sigma_y} = 1.0$ $R_R \leqq 0.5$

$\frac{\sigma_{cr}}{\sigma_y} = 1.5 - R_R$ $0.5 < R_R \leqq 1.0$

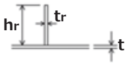
$\frac{\sigma_{cr}}{\sigma_y} = \frac{0.5}{R_R^2}$ $1.0 < R_R$

---

## Design of longitudinal ribs ( $I_\ell$ )

$$\gamma_\ell = \frac{I_\ell}{\frac{bt^3}{12(1-\nu^2)}} \cong \frac{I_\ell}{\frac{bt^3}{11}} \longrightarrow I_\ell \geqq \frac{bt^3}{11} \cdot \gamma_{\ell,\text{req.}}$$

$$A_\ell = \frac{bt}{10n}$$



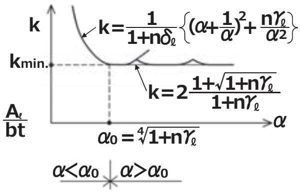
$A_\ell = h_r t_r$

$I_\ell = \frac{h_r^3 t_r}{3}$

From condition,

$\sigma_E^{①}$ (buckling stress of plate between ribs) $=$ $\sigma_E^{②}$ (buckling stress of stiffened plate)

$k = 4n^2$



$$k = \frac{1}{1+n\delta_\ell}\left\{\left(\alpha+\frac{1}{\alpha}\right)^2 + \frac{n\gamma_\ell}{\alpha^2}\right\}$$

$$k = 2\frac{1+\sqrt{1+n\gamma_\ell}}{1+n\gamma_\ell}$$

$$\alpha_0 = \sqrt[4]{1+n\gamma_\ell}$$

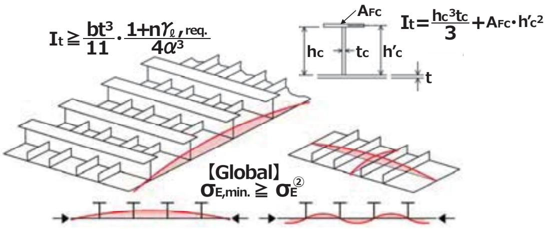
$$\delta_\ell = \frac{A_\ell}{bt}$$

---

## Design of cross beam ( $I_t$ )

$$I_t \geqq \frac{bt^3}{11} \cdot \frac{1+n\gamma_{\ell,\text{req.}}}{4\alpha^3}$$

$$I_t = \frac{h_c^3 t_c}{3} + A_{Fc} \cdot h_c'^2$$



$$\alpha \leqq \alpha_0 \quad 2\frac{1+\sqrt{(1+n\gamma_\ell)(1+\gamma_t/\alpha)}}{1+n\gamma_\ell} \geqq \frac{1}{1+n\gamma_\ell}\left\{\left(\alpha+\frac{1}{\alpha}\right)^2 + \frac{n\gamma_\ell}{\alpha^2}\right\}$$

$$\gamma_{t,\text{req.}} = \frac{1+n\gamma_{\ell,\text{req.}}}{4\alpha^3} \underbrace{- \frac{\alpha}{2} + \frac{5}{4(1+n\delta_\ell)}}_{\text{neglected}}$$

$$\alpha > \alpha_0 \quad 2\frac{1+\sqrt{(1+n\gamma_\ell)(1+\gamma_t/\alpha)}}{1+n\gamma_\ell} \geqq 2\frac{1+\sqrt{(1+n\delta_\ell)}}{1+n\delta_\ell} \Rightarrow \gamma_t = 0$$

---

1) $\alpha \leqq \alpha_0$ (&) $I_t \geqq \frac{bt^3}{11} \cdot \frac{1+n\gamma_{\ell,\text{req.}}}{4\alpha^3}$

$\gamma_{\ell,\text{req.}} = 4\alpha^2 n\left(\frac{t_0}{t}\right)^2(1+n\delta_\ell) - \frac{(\alpha^2+1)^2}{n}$ ( $t \geqq t_0$ ) ( $R_R < 0.5$ )

$= 4\alpha^2 n(1+n\delta_\ell) - \frac{(\alpha^2+1)}{n}$ ( $t < t_0$ ) ( $R_R > 0.5$ )

( $t_0$ is the thickness when $R_R = 0.5$ )

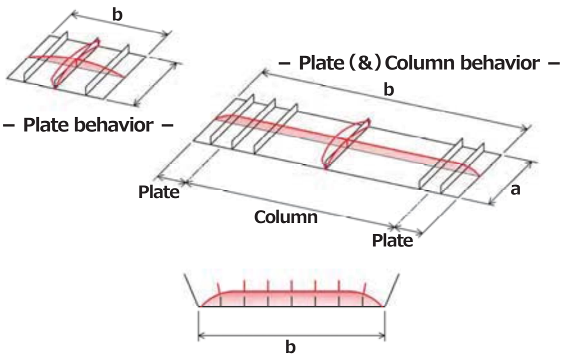
2) the others [ $(\alpha > \alpha_0), (\alpha \leqq \alpha_0$ & $I_t < \frac{bt^3}{11} \cdot \frac{1+n\gamma_{\ell,\text{reg.}}}{4\alpha^3})$ ]

$\gamma_{\ell,\text{reg.}} = \frac{1}{n}\left[\left\{2n^2\left(\frac{t_0}{t}\right)(1+n\delta_\ell)-1\right\}^2-1\right]$ ( $t \geqq t_0$ ) ( $R_R < 0.5$ )

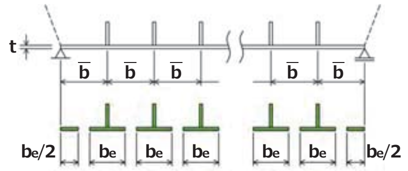
$= \frac{1}{n}\left[\left\{2n^2(1+n\delta_\ell)-1\right\}^2-1\right]$ ( $t > t_0$ ) ( $R_R > 0.5$ )

---

## Buckling mode shape of plate with large α



## Column approach

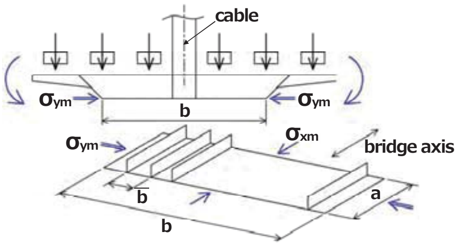


$$N_{cr} = \left\{\left(\frac{\sigma_{cr}}{\sigma_y}\right)_c \cdot n \cdot A_T + \left(\frac{\sigma_{cr}}{\sigma_y}\right)_p \cdot b_e \cdot t\right\} \quad \text{——(3)}$$

$N_{cr}$ : load carrying capacity of stiffened plate
$(\sigma_{cr})_c$ : load carrying capacity of column
n : number of rib
$A_T$ : cross-sectional area of column with T-section
$(\sigma_{cr})_p$ : load carrying capacity of plate

---

## Biaxial compression



$$\left(\frac{\sigma_{xm}}{\sigma_{xmo}}\right)^2 + \left(\frac{\sigma_{ym}}{\sigma_{ymo}}\right)^2 < 1.0$$

↑proposed by Kitada（1988）

$\sigma_{xmo}$ (= strength under $\sigma_{xm}$ only）is estimated by eq.（3）

---

## Evaluation of strength

$\underline{(\sigma_{cr})_c}$

$$\left(\frac{\sigma_{cr}}{\sigma_y}\right)_c = 1.0 \qquad (\qquad \bar{\lambda} \leq 0.2)$$
$$= 1.109 - 0.545\bar{\lambda} \qquad (0.2 < \bar{\lambda} \leq 1.0)$$
$$= 1.0/(0.773 + \bar{\lambda}^2) \qquad (1.0 < \bar{\lambda} \qquad )$$

$$\bar{\lambda} = \frac{1}{\pi}\sqrt{\frac{\sigma_y}{E}}\left(\frac{a}{r}\right) \qquad r = \sqrt{\frac{I_T}{A_T}}$$

$I_T$ : geometrical moment of inertia of T-section
a : distance of cross beams

$$\frac{b_e}{b} = 0.702R_e^3 - 1.640R_e^2 + 0.654R_e + 0.926$$

$$R_e = 0.526\frac{\bar{b}}{t}\sqrt{\frac{\sigma_{cr}}{E}}$$

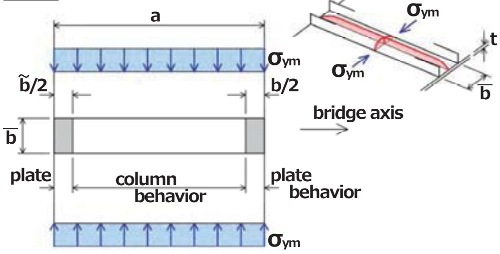
First, $\sigma_{cr}$ is assumed and repeat calculation until converged σcr is obtained

$\underline{(\sigma_{cr})_P}$ : load carrying capacity of plate with width（$b_e$）, and simply supported at 4-side.

---

## Strength of plate under $\sigma_{ym}$ only ($\sigma_{ymo}$)

$\underline{\sigma_{ymo}}$ $[\gamma\{= I_\ell/(bt^3/11)\} > \gamma^*]$



$$\sigma_{ymo} = \frac{\sigma_{ym} + 0.9\sigma_{ym}(\alpha - 1)}{\alpha}$$

$\sigma_{ymo}$ : strength under $\sigma_{ym}$ only

---

$$\frac{\sigma_{ymc}}{\sigma_y} = 1.0 \qquad (\qquad \bar{\lambda} \leq 0.2)$$
$$= 1.109 - 0.545\bar{\lambda} \qquad (0.2 < \bar{\lambda} \leq 1.0)$$
$$= 1.0/(0.773 + \bar{\lambda}^2) \qquad (1.0 < \bar{\lambda} \qquad )$$

$$\bar{\lambda} = \frac{\sqrt{12}}{\pi}\frac{\bar{b}}{t}\sqrt{\frac{\sigma_y}{E}}$$

$$\frac{\sigma_{ym}}{\sigma_y} = 0.542R^3 - 1.249R^2 + 0.412R + 0.968 \quad (0.3 \leq R \leq 1.3)$$

↑proposed by Komatsu（1978）

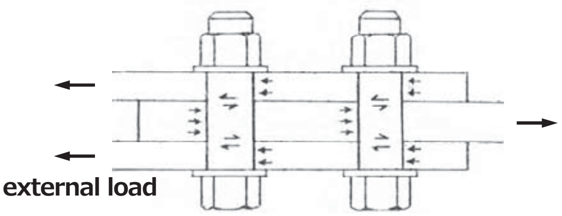
$$R = \frac{1}{\pi}\sqrt{\frac{\sigma_y}{E} \cdot \frac{12(1-\nu^2)}{\pi^2 k}} \cdot \left(\frac{b}{t}\right) \quad (k = 4.0)$$

[11-4-1]

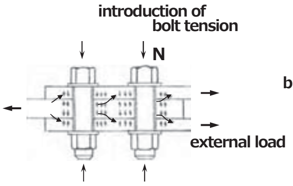
# Design of Connection (Bolt)

1) Friction grip connection
2) Bearing connection
3) Tension-type bolt connection

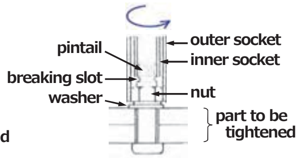
## Bearing connection



## Friction grip connection

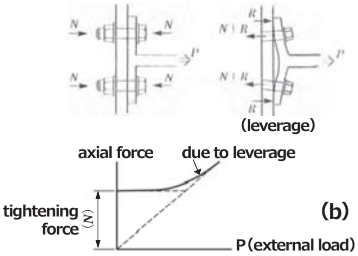


【 hexagon headed bolt 】

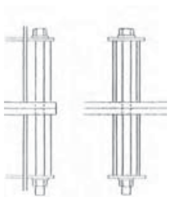


【 torque-shear bolt 】
tension introduction

## Tensile-type bolted connection



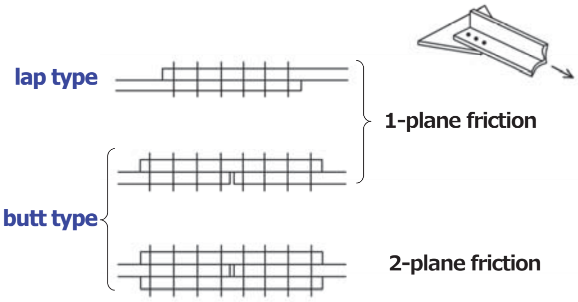
(a) short (tightening) type



(b) long (tightening) type

↑ used for tower of suspension bridges

## Bolt Connection

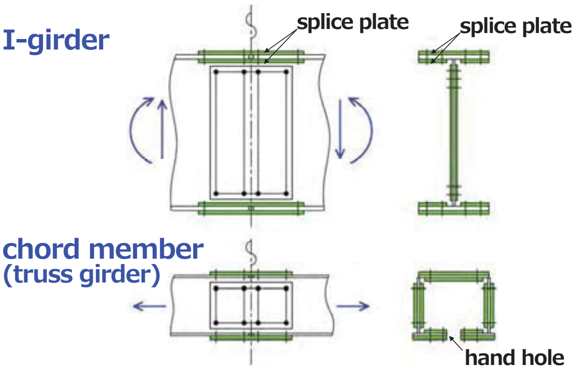


lap type

butt type

1-plane friction

2-plane friction



## Tensile test

## Allowable bolt force ($\rho_a$) per one friction plane

$$\rho_a = \frac{\mu N}{\gamma}$$

$$N = \alpha \sigma_y A_e$$

$\mu$ : friction (slip) coefficient ( $\mu = 0.4$ )

$\gamma$ : Safety factor ( $\gamma = 1.7$ )

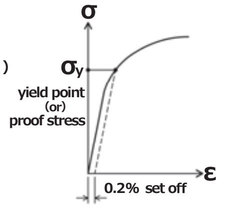
$\alpha$ : ratio to yield point

$\alpha = 0.85$ ( F8T )

$= 0.75$ ( F10T, S10T )

$\sigma_y$ : yield point

$A_e$ : effective area of bolt thread



S-S relation
(high strength steel)

## Arrangement of bolts

### max. and min. bolt distance and min. edge length

(mm)

|  | min. | max. |  |  | min. edge* |
|---|---|---|---|---|---|
|  |  | p |  | g |  |
| M20 | 65 | 130 | 12t<br>(staggered type)<br>$15t - \frac{3}{8}g$<br>not exceeding<br>12t | 24t<br>not exceeding<br>300㎜ | 28 |
| M22 | 75 | 150 |  |  | 32 |
| M24 | 85 | 170 |  |  | 37 |

t : outer side plate thickness (or) thickness of rolled steel

(*) applied to「press cut edge」「automatic gas cutting edge」,「finisher edge」

## Allowable bolt force ($\rho_a$)

| bolt grade | nominal designation of thread | $\gamma$ | $\mu$ | $\alpha$ | $\sigma_y$ (N/㎟) | $A_e$ (㎟) | N (kN) | $\rho_a$ (kN) |
|---|---|---|---|---|---|---|---|---|
| F8T* | M20**<br>M22<br>M24 | 1.7 | 0.4 | 0.85 | 640 | 245<br>303<br>353 | 133<br>165<br>192 | 31.3(31)<br>38.8(39)<br>45.2(45) |
| F10T<br>***<br>S10T | M20<br>M22<br>M24 | 1.7 | 0.4 | 0.75 | 900 | 245<br>303<br>353 | 165<br>205<br>238 | 38.8(39)<br>48.2(48)<br>56.0(56) |

(design value)

* F 8 T

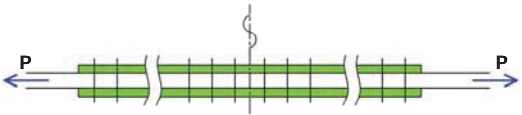
Tensile strength

80 kgf/㎟ (strength)

Friction grip joint

** Diameter of bolt

*** Torque-shear type bolt (S : for Structural joint)



$n_k$ : number of row

( $n_k \le 8$ ) is recommended

## Transfer force (P) at the design

$$P = \max.\{\sigma_1 A,\ 0.75\sigma_a A\}$$

$\sigma_1 A$ : working stress (strength)

$\sigma_a A$ : full strength ( $\sigma_a$ : allowable stress )

### [In tension]

$$\sigma_a \times A = \sigma_{ta} \times A_n$$

$\sigma_{ta}$ : allowable tensile stress

$A_n$ : net cross-sectional area*

*hole width (d + 3 : mm) for bolt with a diameter (d : mm) is subtracted

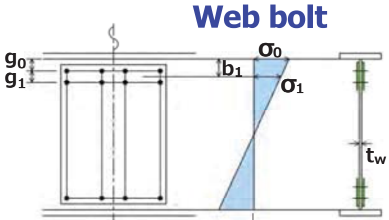
### [In compression]

$$\sigma_a \times A = \sigma_{ca} \times A_g$$

$\sigma_{ca}$ : allowable compressive stress

$A_g$ : gross cross-sectional area

## Web bolt



$b_1 = g_0 + \frac{g_1}{2}$

$p = b_1 t_w \left(\frac{\sigma_0 + \sigma_1}{2}\right)$

$\rho = \frac{p}{n} < \rho_a$

$n_{req.} > \frac{p}{\rho_a}$

n : number of bolt

$g_0 = 100$ mm

$g_1 = 80$ mm

$\sigma_0 = 150$ N/mm²

$\sigma_1 = 130$ N/mm²

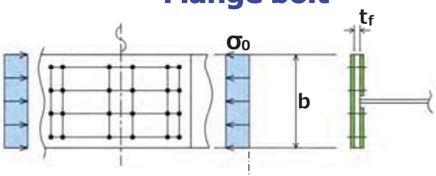
$t_w = 12$ mm

$p = (100 + \frac{80}{2}) \times 12 \times \frac{150+130}{2} = 235.2$kN

【 2 – plane friction, M22, F10T 】

$n_{req.} > \frac{235.2}{96} = 2.45 \Rightarrow \underline{3}$

## Flange bolt



$P = \sigma_0 b t_f$

$\rho = \frac{p}{n} < \rho_a$

$n_{req.} > \frac{p}{\rho_a}$

n : total number of bolt

$b = 300$ mm

$\sigma = 150$ N/mm²

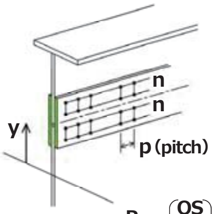
$t_f = 25$ mm

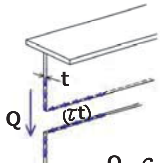
$P = 300 \times 25 \times 150 = 1{,}125$kN

【 2 – plane friction, M22, F10T 】

$n_{req.} > \frac{1{,}125}{96} = 11.7$ ( $\Rightarrow \underline{n = 12}$ )

## Web shear connection





$$\tau t = \frac{Q}{I}\int y t\,ds = \frac{QS}{I}$$

$$P_h = \left(\frac{QS}{I}\right)\frac{p}{n} < \rho_a$$

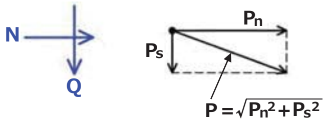
$$n_{req.} > \frac{\left(\frac{QS}{I}\right)p}{\rho_a}$$

## When subjected to shear force（Q）

$$P_s = \frac{Q}{n} < \rho_a \quad \Rightarrow \quad n_{req.} > \frac{Q}{\rho_a}$$

## When subjected to axial force（N）and shear

$$P \ = \sqrt{P_n^2 + P_s^2} < \rho_a$$



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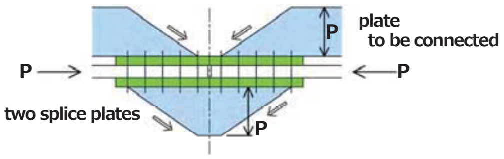
# Design of splice plates

Force transfer



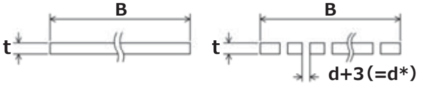
## （1）in compression

$$\sigma_{[\text{ splice plate }]} < \sigma_{ta} \ (= \sigma_y / 1.7\ )$$

---

## （2）in tension

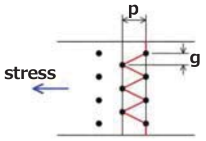
net sectional area（$A_n$）has to be used
plate to be connected also has to be checked



$A_g = Bt$
(gross area)

$A_n = (B - \ n \times d^*)\ t$
n ： number of bolt
d ： diameter of bolt

【 in case of staggered arrangement 】



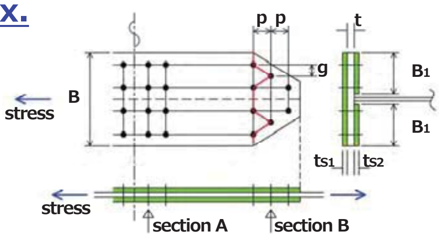
$$A_n = A_g - \{4d^* - 3w\}\ t$$

$$w = d^* - \frac{p^2}{4g}$$

when　$w < 0 \ \longrightarrow \ A_n = A_g - 4d^* t$

---

## ex.



Net area of splice plate（Section A）

$$A_n = (\,B - 4d^*\,)\,ts_1 + 2\,(\,B_1 - 2d^*\,)\,ts_2$$

Net area of plate to be connected（Section B）

$$A_n = A_g - \{2d^* + 4\,(\,d^* - \frac{p^2}{4g})\}\ t$$

$$A_g = Bt$$

---

## （3）in flexure（bending）

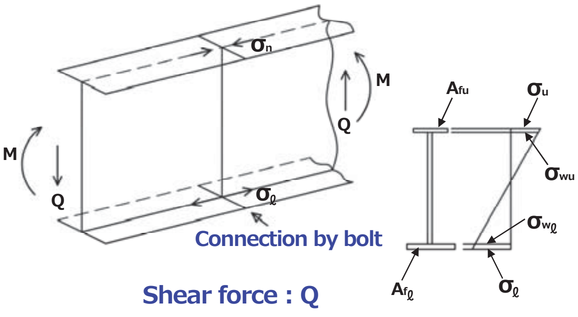
$$\sigma \ = \frac{M}{I}y < \sigma_a$$

$\sigma$ ： stress at tip（fiber）of splice plate
M ： bending moment（carried by splice plate）
I ： geometrical moment of inertia of splice plate

ex.　will be given later

---

# Design of bolt connection



$$(\,\sigma = \frac{M}{I}y\,)$$

---

## [ upper flange ]

$$\sigma_o \ = \max.\ \{\sigma_u\,,\,0.75\sigma_{ca}\}$$

$\sigma_{ca}$ ：allowable compressive stress

### （1）Number of bolt（$n_{req.}$）

$$n_{req.} \ > \ \frac{\sigma_o A_{fu}}{\rho_a}$$

### （2）Splice plate

$$\sigma_{SpL} \ = \sigma_o \frac{A_{fu}}{A_{SpL}} < \sigma_{ta}$$

$\sigma_{ta}$ ：allowable tensile stress

(or)　$A_{SpL} \ > \ A_{fu}$

---

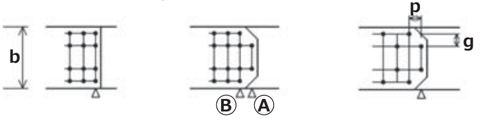
## [ lower flange ]

$$\sigma_o \ = \max.\ \{\sigma_\ell\,,\,0.75\sigma_{ta}\}$$

### （1）Number of bolt（$n_{req.}$）

$$n_{req.} \ > \ \frac{\sigma_o A_{f\ell}}{\rho_a}$$

### （2）Check the plate（thickness = t）to be connected



$A_{net} = \{b\text{-}4(d+3)\}\ t$

Ⓐ
$A_{net} = \{b\text{-}2(d+3)\}\ t$

Ⓑ
$A_{net} = \{b\text{-}4(d+3)\}\ t$

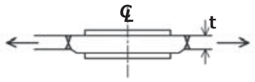
if　$w = d\text{-}p^2/4g > 0$
$A_{net} = \{b\text{-}2(d+3)\text{-}2w\}t$

if　$w < 0$
$A_{net} = \{b\text{-}2(d+3)\}\ t$

$$\sigma_1 = \frac{\sigma_o A_{f\ell}}{A_{net}} < \sigma_{ta}$$

if the above is not satisfied,

### (a) plate thickness is increased



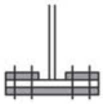
### (b) change bolt arrangement

### (3) Splice plate

$$\sigma_{SpL} = \sigma_o \frac{A_{f\ell}}{A_{SpL}} < \sigma_{ta}$$

$A_{SpL}$ : net cross sectional area

(or) $A_{SpL} > A_{f\ell}$



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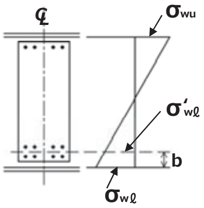
## [ web ]



### (1) Number of bolt

$$P_w = \frac{\sigma_{w\ell} + \sigma'_{w\ell}}{2} bt \longrightarrow n_{req.} > \frac{P_w}{\rho_a} \text{ (}n_1\text{ is selected)}$$

### (2) Safety check of bolt

$$\rho_S = \frac{P_w}{n_t}, \quad \rho_n = \frac{P_w}{n_t}$$

$$\sqrt{\rho_S^2 + \rho_n^2} < \rho_a$$

$n_t$ : total number of bolt in web

---

### (3) Splice plate

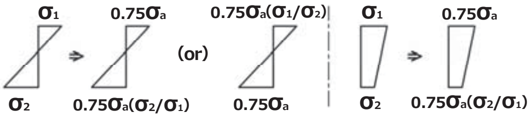
$$\sigma_{SpL} = \frac{M_S}{W_S} < \sigma_{ta}$$

$M_S$ : moment on splice plate

$W_S$ : section modulus of splice plate

(or) $I_{SpL} > I_w$

**[ NOTE ]** when produced stress is less than 75% of full strength



---

## Bearing connection

allowable shear stress (N/㎟)

| B8T | B10T |
|---|---|
| 150 | 190 |

## Tension-type bolt connection

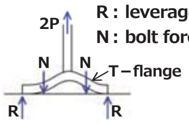
【 short bolt 】

$$P_p = \frac{P(1+p_y)}{n} < \rho_{a\ell}$$

P : tension force

$p_y$ : leverage action

n : number bolt



R : leverage reaction

N : bolt force

【 long bolt 】

$$P_p = \frac{P}{n} < \rho_{a\ell}$$

$\rho_{a\ell}$ (allowable force) (kN)

| | F10T | S10T |
|---|---|---|
| M20 | 130 | 130 |
| M22 | 160 | 160 |
| M24 | 185 | 185 |

# Design of Connection (Welding)

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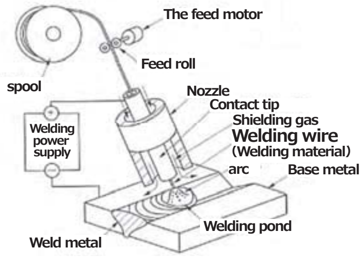
## Gas shield arc welding method



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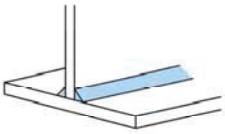
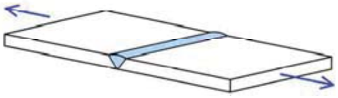
## Welding



**Fillet Welding**

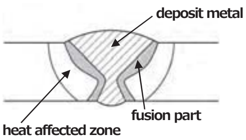


**Groove Welding**

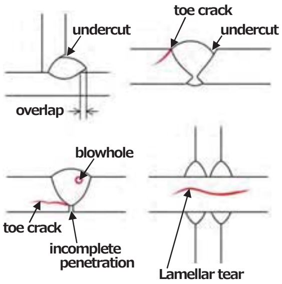
---

## Welded section and defects

### [Section]



### [Defects]



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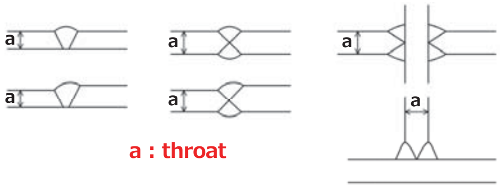
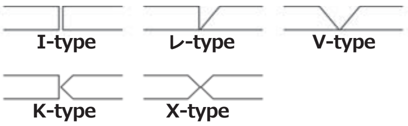
## Welding type（groove welding）



a : throat

groove shape



---

## Design method

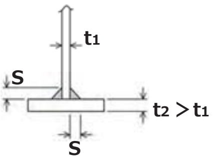
1）Size of fillet welding

$t_1 > S > \sqrt{2t_2}$

$t_1$ : thinner plate thickness

$t_2$ : thicker plate thickness

( $S_{min.} > 6mm$ )

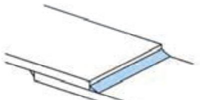


$t_1 = 18mm$ , $t_2 = 36mm$

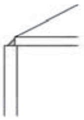
$18 > S > \sqrt{2\times36} = \sqrt{72} = 8.5 \rightarrow \underline{S = 9mm}$

---

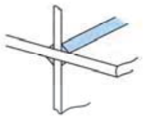
## Welding type（fillet welding）



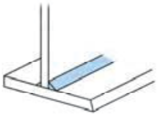
Lap-type



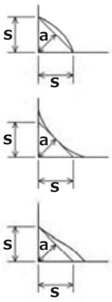
Corner-type



Cross-type



T-type



s : size
a : throat

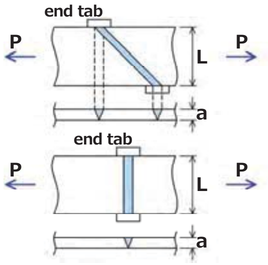
---

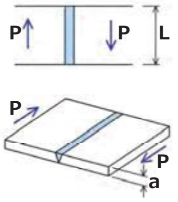
2）Axial force and shear

a）groove welding

$$\sigma = \frac{P}{\Sigma aL}$$

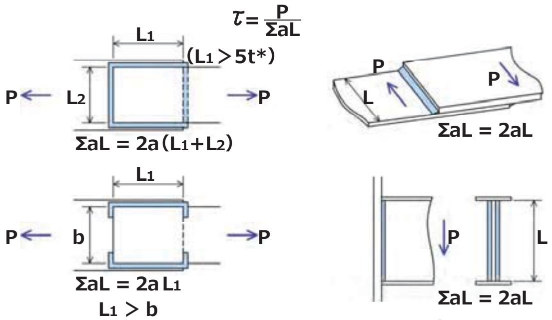
$$\tau = \frac{P}{\Sigma aL}$$





### b) fillet welding

$$\tau = \frac{P}{\Sigma aL}$$



$L_1 > b$

$b < 10t^*$ (in compression)

$b < 20t^*$ (in tension)

$t^*$ : thinner plate thickness

a : throat

---

## 3) bending moment

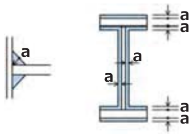
【 groove welding 】

$$\sigma = \frac{M}{I} y < \sigma_a$$

【 fillet welding 】

$$\tau = \frac{M}{I} y < \tau_a$$

– fillet welding –



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## 4) Axial force, bending and shear combination

【 groove welding 】

$$\left(\frac{\sigma}{\sigma_a}\right)^2 + \left(\frac{\tau_s}{\tau_a}\right)^2 < 1.2$$

【 fillet welding 】

$$\left(\frac{\tau}{\tau_a}\right)^2 + \left(\frac{\tau_s}{\tau_a}\right)^2 < 1.0$$

$\sigma$ : normal stress due to axial force and/or bending

$\tau$ : shear stress due to axial force and/or bending

$\tau_s$ : shear stress due to shear force

$\sigma_a$ : allowable tensile stress

$\tau_a$ : allowable shear stress

---

## Allowable stress

( N/mm$^2$ )

| | | SM400<br>SMA400W | SM490 | SM490Y<br>SM520<br>SMA490YW | SM570<br>SMA570W |
|---|---|---|---|---|---|
| groove welding | $\sigma_a$ | 140 | 185 | 210 | 255 |
| | $\tau_a$ | 80 | 105 | 120 | 140 |
| fillet welding | $\tau_a$ | 80 | 105 | 120 | 140 |

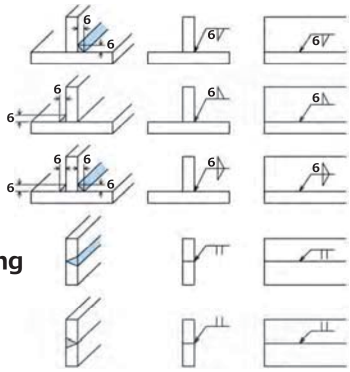
( $t \leq 40$mm )

---

## Welding symbol (1)

**Fillet welding**

**Groove welding**
**(I-type)**



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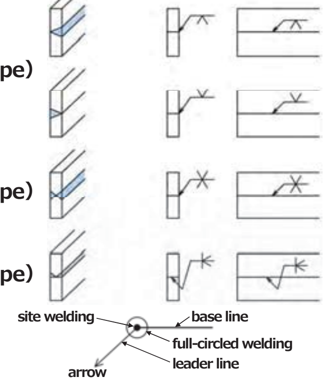
## Welding symbol (2)

(V-type)

(X-type)

(K-type)



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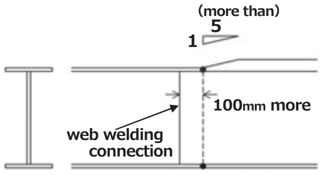
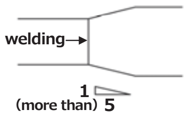
## Remarks



change of plate thickness



change of plate width

---

# Quality control is important

## - check of defect of welding -

[11-4-3]

# Fatigue
# and its Design

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## Example of fatigue cracks

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## Collapse, Failure of Steel Structures

**Buckling strength and material break**

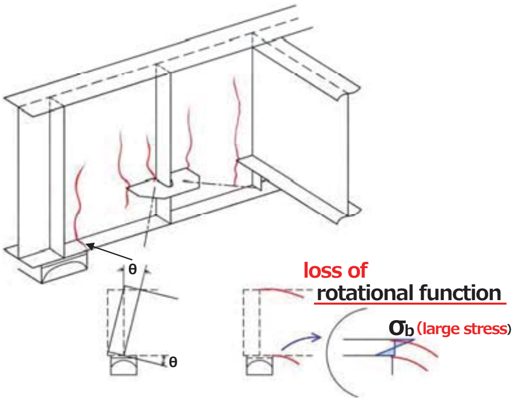
**depend on material strength**

**However, fatigue strength**

**does not depend on material strength**

---

## Example of cracking（１）



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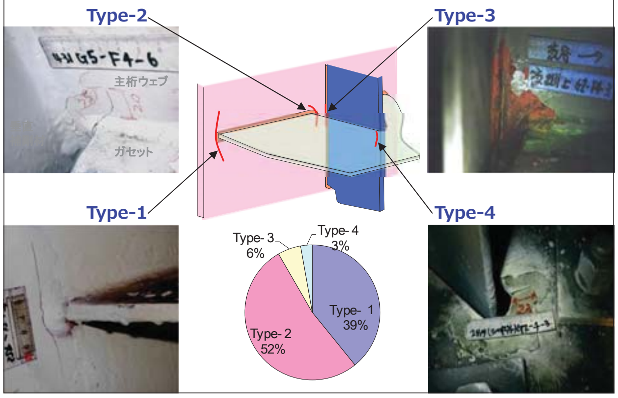
## Fatigue cracking at gusset



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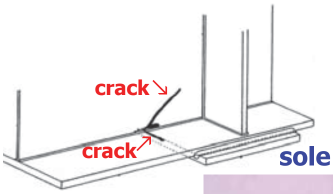
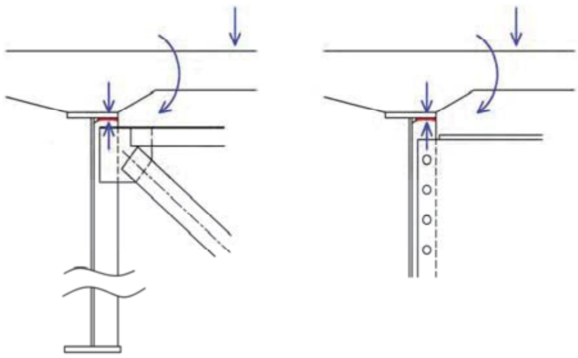


**sole plate**

**sole plate**

## Example of cracking（2）



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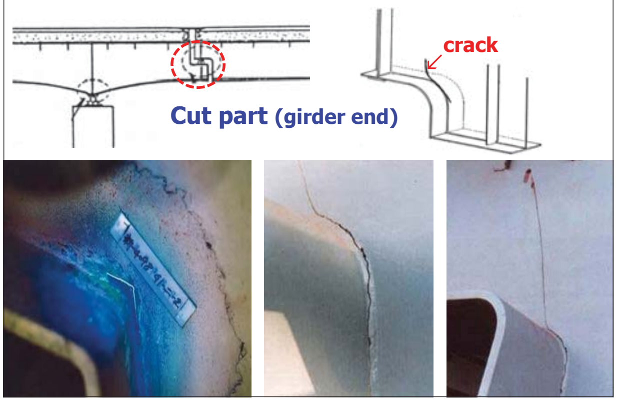
## Example of cracking（3）



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# Example (4)
# Steel deck fatigue

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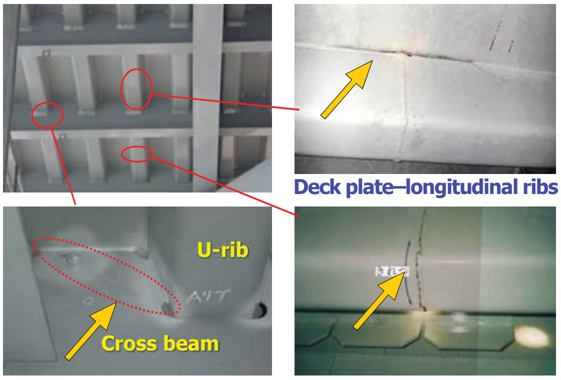
## Steel deck fatigue（1）



Deck plate–longitudinal ribs

Cross beam–longitudinal rib

Welding at longitudinal ribs

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## [Recommendation]

**Structural details with stress concentration should be avoided.**

**Evaluation of fatigue strength of such part is difficult.**

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## Steel deck fatigue（2）

Penetrating to deck surface

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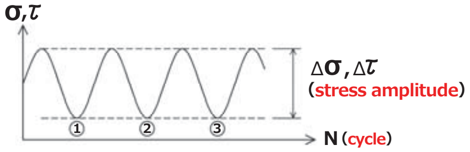
## Fatigue limit state

Cracking progresses, and the structural function（or）required performance is impaired.

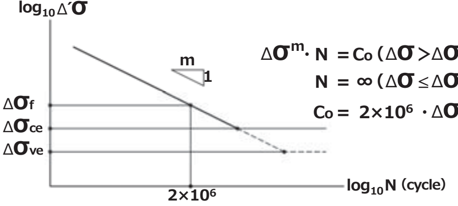
## Dominate parameter controlling fatigue strength



{(Δσ,Δτ) and N}

important parameter

# S-N curve



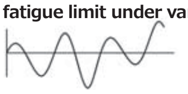
$\Delta\sigma^m \cdot N = C_o \; (\Delta\sigma > \Delta\sigma_{ce}, \Delta\sigma_{ve})$

$N = \infty \; (\Delta\sigma \leq \Delta\sigma_{ce}, \Delta\sigma_{ve})$

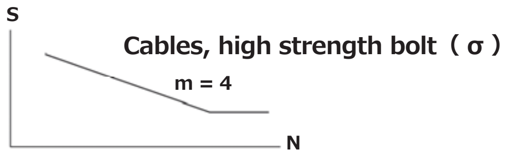
$C_o = 2\times10^6 \cdot \Delta\sigma_f{}^m$

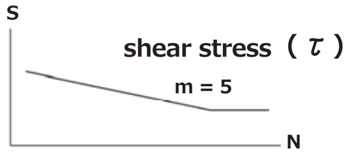
$\Delta\sigma_{ce}$ : fatigue limit under constant stress amplitude

$\Delta\sigma_{ve}$ : fatigue limit under variable stress amplitude

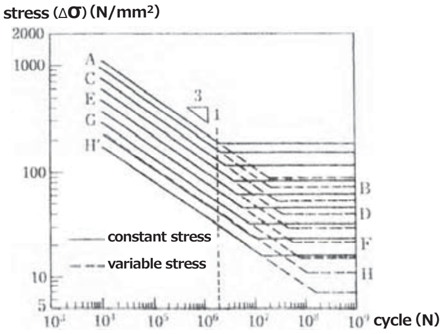


# S-N curves

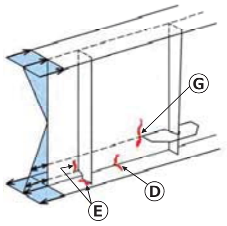


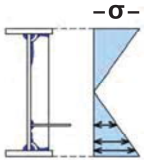


# Fatigue design curve（normal stress σ）



# Check of fatigue strength（normal stress）





Fatigue strength (N/㎟)

| Grade | $\Delta\sigma_f$ | $\Delta\sigma_{ce}$ | $\Delta\sigma_{ve}$ |
|---|---|---|---|
| D | 100 | 84 | 39 |
| E | 80 | 62 | 29 |
| G | 50 | 32 | 15 |

Corresponding to cycle when cracking penetrate the thickness

## Step－1 （Simplified method）

$\Delta\sigma_{max.} < \Delta\sigma_{ce} \cdot C_R \cdot C_T$

$\Delta\tau_{max.} < \Delta\tau_{ce}$

$\Delta\sigma_{max.}$ , $\Delta\tau_{max.}$ : maximum stress range

$C_R = 1.00$ $\quad -1.0 < R < 1.0$

$C_R = 1.30(1.00-R)/(1.60-R)$ $\quad R < -1.0$

$C_R = 1.30$ $\quad (\sigma_{max.}, \sigma_{min.} < 0)$

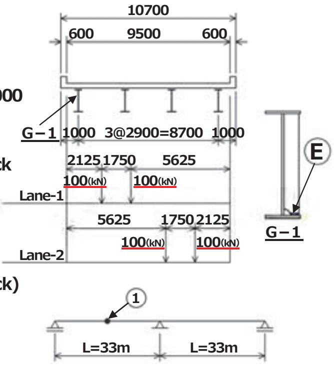
$R = (\sigma_{max.}/\sigma_{min.}) \leftarrow \sigma_{max.}, \sigma_{min.}$ including dead load effect

$C_T = \sqrt[4]{25/t}$ $\quad$ ( if $t \geq 25$㎜)

in case of cables $\quad C_R = (1-R)/(1-0.9R)$

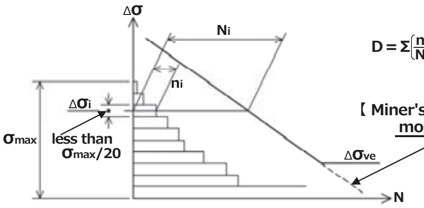
## ex.

- Design period 100-year (life)
- ADTT (per one way, lane)＝3,000
  Average Daily Truck Traffic
- Design load for fatigue check
  (T-Load) × (1+ if )
  T-Load : 100kN (2-point)
- if ＝ 10 / (50+L)
  ↑half value (at fatigue check)



## Step－2

### Linear cumulative damage rule

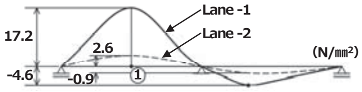


$D = \Sigma\left(\frac{n_i}{N_i}\right) = \underline{1.0}$
↑
fatigue damage

【 Miner's law (or) modified Miner's law 】

$n_i$ : (cycle of $\Delta\sigma_i$ ) in obtained by rainflow method

## Step－1



$\gamma_T = \underline{3.00}$ (for max. stress evaluation)

$\Delta\sigma_{1,1} = \{17.2-(-4.6)\} \times \underline{3.00} = 65.4$ (N/㎟)

$\Delta\sigma_{2,1} = \{2.6-(-0.9)\} \times \underline{3.00} = 10.5$ (N/㎟)

$\sigma_{i,j}$ i : lane number , j : number of stress range

$C_R = C_t = 1.0$ (assumption of this case)

$\Delta\sigma_{ce}$(grade Ⓔ) = 62.0 (N/㎟)

$\Delta\sigma_{max.} = \underline{65.4\ (N/㎟) >} \; \Delta\sigma_{ce} \times C_R \times C_T = \underline{62.0\ (N/㎟)}$

**OUT!!**

Go to Step－2

## Step – 2

$D_{i,j}$ = $nt_i$ / $N_{i,j}$

$nt_i$ : number of stress range $\Delta\sigma_{i,j}$

(= $NT_i \times 365$(day) $\times 100$(year))

$NT_i$ = $ADTT_i \times \gamma_n$ ($\gamma_n$ =0.03*)

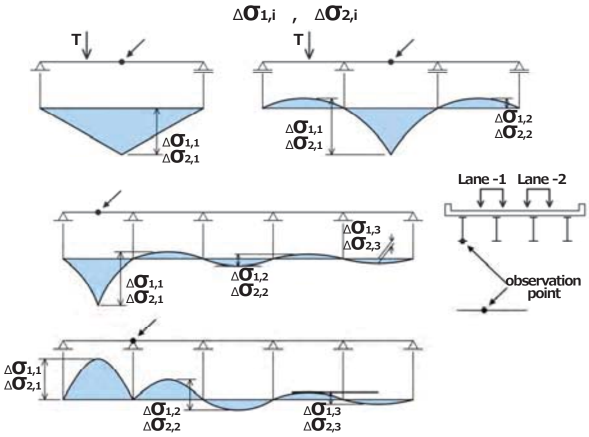
$N_{i,j}$ = fatigue life for stress range of $\Delta\sigma_{i,j}$

$(= 2 \times 10^6 (\Delta\sigma_f \times C_R \times C_T)^m / \Delta\sigma_{i,j}{}^m)$

if $\Delta\sigma_{i,j} < \sigma_{ve} \times C_R \times C_T \rightarrow N_{i,j} = \infty$

* reducing factor

(adjusting factor to take into account of passing truck with weight exceeding 20tf. )



$NT_i$ = $ADTT \times \gamma_n = 3000 \times 0.03 = 90$

$nt_i$ = $NT_i \times 365$(day) $\times 100$(year) $= 3.29 \times 10^6$ (cycle)

$\Delta\sigma_f = 80$(N/㎟) → $\Delta\sigma_f C_R C_T = 80$(N/㎟)

$\Delta\sigma_{ve} = 29$(N/㎟) → $\Delta\sigma_{ve} C_R C_T = 29$(N/㎟)

$m = 3$

$N_{i,j} = 2 \times 10^6 (\Delta\sigma_f C_R C_T)^m / \Delta\sigma_{i,j}$ $(C_R = C_t = 1.0)$

$N_{1,1} = 2 \times 10^6 \times (80 / 65.4)^3 = 3.66 \times 10^6$

$N_{2,1} = \infty$ ( $\sigma_{2,1} = 10.3$(N/㎟) $<$ $\Delta\sigma_{ve} C_R C_T = 29$(N/㎟) )

$$\frac{3.29 \times 10^6}{3.66 \times 10^6} = 0.90 < 1.0$$ ( OK )

## Preventive method for enhancing fatigue strength

【1】Grinder

- welding（head）shape is made smooth to reduce stress concentration

【2】TiG（Tungsten）welding

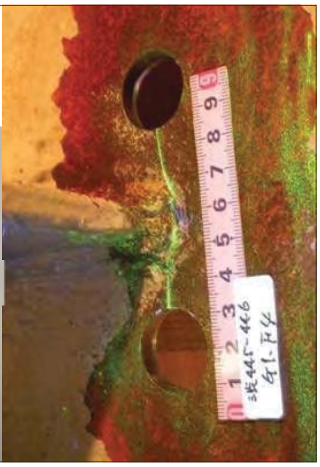
- good appearance（smooth shape）is obtained
- less possibility of occurrence of welding defects
- take long time for welding work

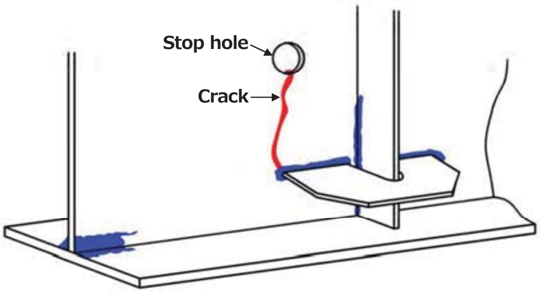
【3】Hammer peening

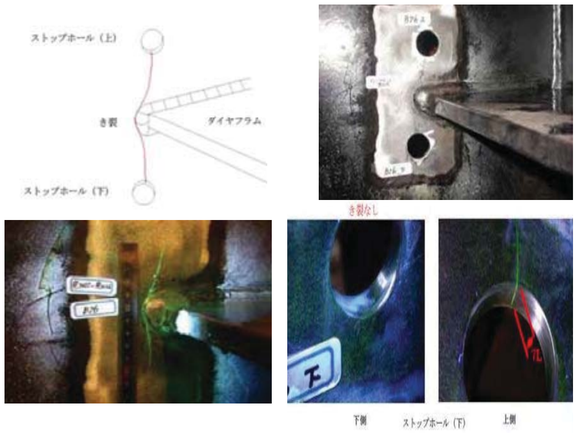
- At weld toe, residual stress and stress concentration are reduced by hitting.

# Repair methods

## Stop hole







## Plate attachment

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# Appendix

## Japanese Spec. Ⅱ Steel Bridge

## Fatigue design

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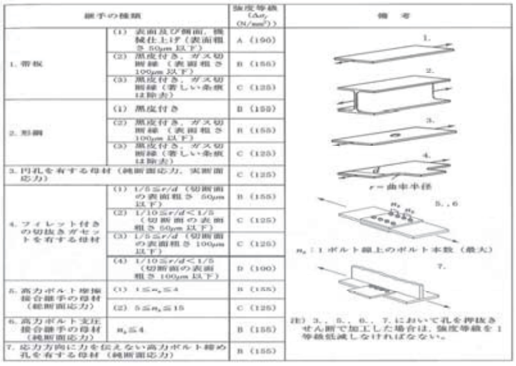
## Reference 「Japanese Spec. Ⅱ Steel Bridge」P-201

Table-6.3.7 Joint type and Strength class under the vertical stress

(a) Non-weld



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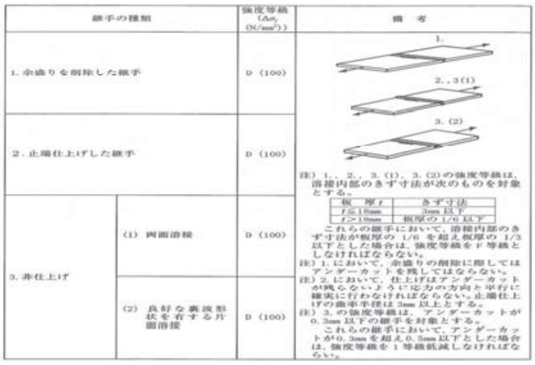
## Reference 「Japanese Spec. Ⅱ Steel Bridge」P-202

Table-6.3.7 Joint type and strength class under the vertical stress

(b) Butt weld



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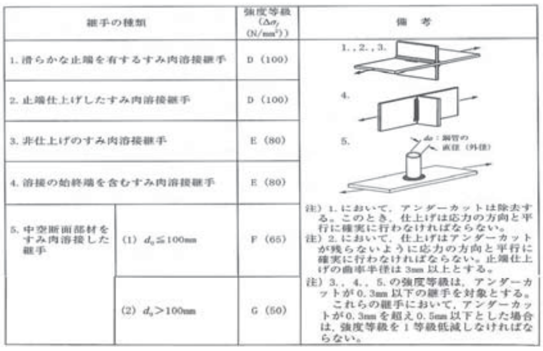
## Reference 「Japanese Spec. Ⅱ Steel Bridge」P-203

Table-6.3.7 Joint type and strength class under the vertical stress

(d) Non-load transfer type cross weld



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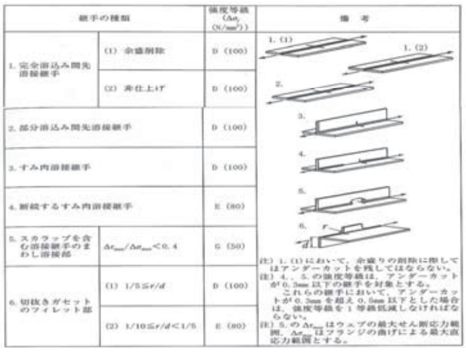
## Reference 「Japanese Spec. Ⅱ Steel Bridge」P-202

Table-6.3.7 Joint type and strength class under the vertical stress

(c) Longitudinal weld



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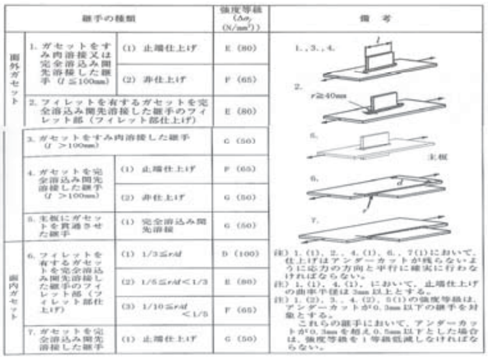
## Reference 「Japanese Spec. Ⅱ Steel Bridge」P-204

Table-6.3.7 Joint type and strength class under the Vertical stress

(e) Load transfer type cross weld

# Reference「Japanese Spec. Ⅱ Steel Bridge」P-204

Table-6.3.7 Joint type and strength class under the vertical stress
(f) Gusset PL weld



| 継手の種類 | | | 強度等級（$\Delta\sigma_f$（N/mm²）） | 備考 |
|---|---|---|---|---|
| 面外ガセット | 1. ガセットをすみ肉溶接又は完全溶込み開先溶接した継手（l ≦100mm） | (1) 止端仕上げ | E (80) | 注）1.(1)、2.、4.(1)、6.、7(1)において、仕上げはアンダーカットが残らないように応力の方向と平行に確実に行わなければならない。<br>注）1.(1)、4.(1)、において、止端仕上げの曲率半径は3mm以上とする。<br>注）1.(2)、3.、4.(2)、5(1)の強度等級は、アンダーカットが0.3mm以下の継手を対象とする。<br>これらの継手において、アンダーカットが0.3mmを超え0.5mm以下とした場合は、強度等級を1等級低減しなければならない。 |
| | | (2) 非仕上げ | F (65) | |
| | 2. フィレットを有するガセットを完全溶込み開先溶接した継手のフィレット部（フィレット部仕上げ） | | E (80) | |
| | 3. ガセットをすみ肉溶接した継手（l >100mm） | | G (50) | |
| | 4. ガセットを完全溶込み開先溶接した継手（l >100mm） | (1) 止端仕上げ | F (65) | |
| | | (2) 非仕上げ | G (50) | |
| | 5. 主板にガセットを貫通させた継手 | (1) 完全溶込み開先溶接 | G (50) | |
| 面内ガセット | 6. フィレットを有するガセットを完全溶込み開先溶接した継手のフィレット部（フィレット部仕上げ） | (1) 1/3≦r/d | D (100) | |
| | | (2) 1/5≦r/d<1/3 | E (80) | |
| | | (3) 1/10≦r/d<1/5 | F (65) | |
| | 7. ガセットを完全溶込み開先溶接した継手 | (1) 止端仕上げ | G (50) | |

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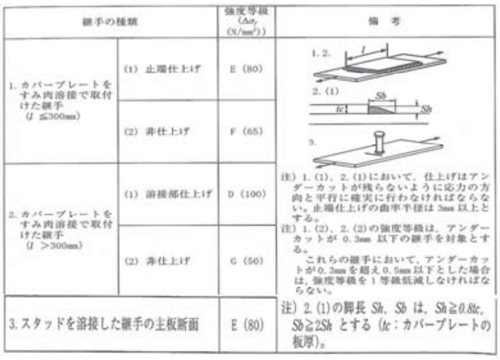
# Reference「Japanese Spec. Ⅱ Steel Bridge」P-205

Table-6.3.7 Joint type and strength class under the vertical stress
(g) Other types weld



| 継手の種類 | | 強度等級（$\Delta\sigma_f$（N/mm²）） | 備考 |
|---|---|---|---|
| 1. カバープレートをすみ肉溶接で取付けた継手（l ≦300mm） | (1) 止端仕上げ | E (80) | 注）1.(1)、2.(1)において、仕上げはアンダーカットが残らないように応力の方向と平行に確実に行わなければならない。止端仕上げの曲率半径は3mm以上とする。<br>注）1.(2)、2.(2)の強度等級は、アンダーカットが0.3mm以下の継手を対象とする。<br>これらの継手において、アンダーカットが0.3mmを超え0.5mm以下とした場合は、強度等級を1等級低減しなければならない。 |
| | (2) 非仕上げ | F (65) | |
| 2. カバープレートをすみ肉溶接で取付けた継手（l >300mm） | (1) 溶接部仕上げ | D (100) | |
| | (2) 非仕上げ | G (50) | |
| 3. スタッドを溶接した継手の主板断面 | | E (80) | 注）2.(1)の脚長 Sh、Sb は、Sh≧0.8tc、Sb≧2Sh とする（tc：カバープレートの板厚）。 |

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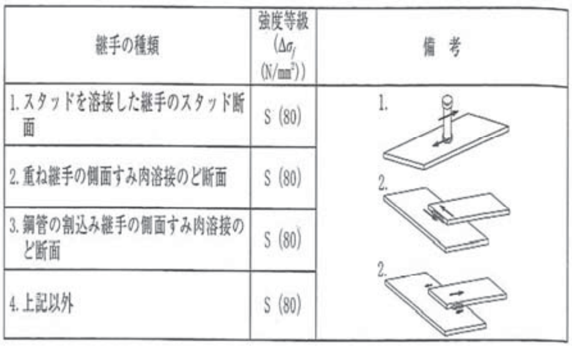
# Reference「Japanese Spec. Ⅱ Steel Bridge」P-206

Table-6.3.8 Joint type and strength class under the share stress



| 継手の種類 | 強度等級（$\Delta\sigma_f$（N/mm²）） | 備考 |
|---|---|---|
| 1. スタッドを溶接した継手のスタッド断面 | S (80) | |
| 2. 重ね継手の側面すみ肉溶接のど断面 | S (80) | |
| 3. 鋼管の割込み継手の側面すみ肉溶接のど断面 | S (80) | |
| 4. 上記以外 | S (80) | |

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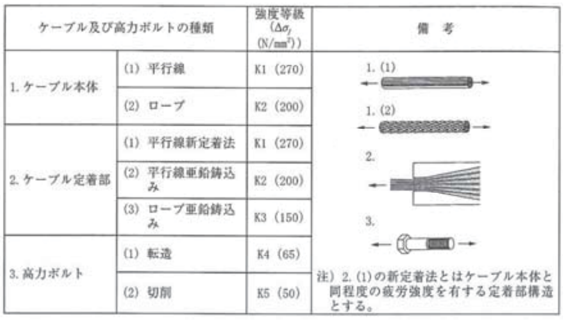
# Reference「Japanese Spec. Ⅱ Steel Bridge」P-206

Table-6.3.9 Cable, H.T. Bolt
and strength class under the vertical stress



| ケーブル及び高力ボルトの種類 | | 強度等級（$\Delta\sigma_f$（N/mm²）） | 備考 |
|---|---|---|---|
| 1. ケーブル本体 | (1) 平行線 | K1 (270) | |
| | (2) ロープ | K2 (200) | |
| 2. ケーブル定着部 | (1) 平行線新定着法 | K1 (270) | |
| | (2) 平行線亜鉛鋳込み | K2 (200) | |
| | (3) ロープ亜鉛鋳込み | K3 (150) | |
| 3. 高力ボルト | (1) 転造 | K4 (65) | |
| | (2) 切削 | K5 (50) | 注）2.(1)の新定着法とはケーブル本体と同程度の疲労強度を有する定着部構造とする。 |

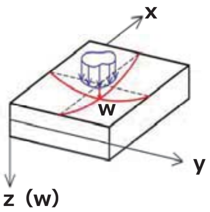
[11-5-1]

# Design of Slabs

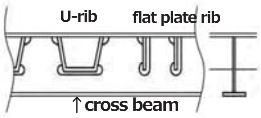
## Concrete and steel decks

RC deck

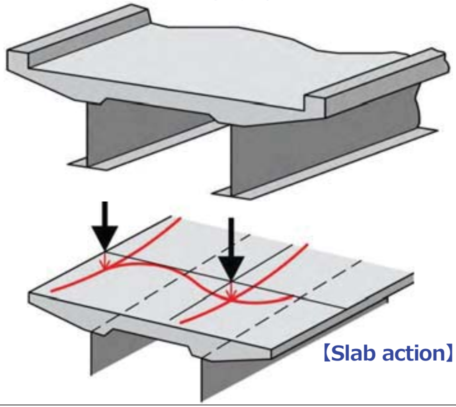
Steel deck

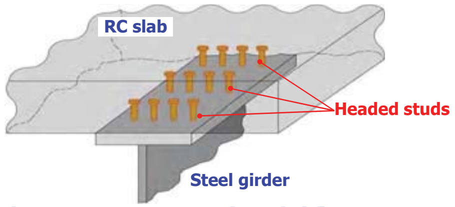


W = w (x,y)

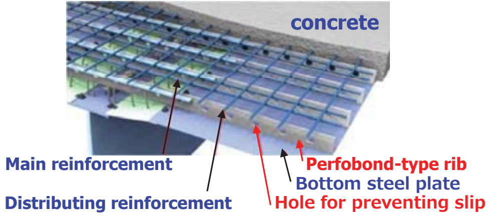


## RC Slab

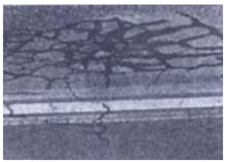




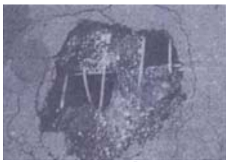
[Steel-concrete composite slab]



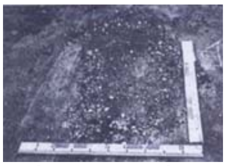
## Damage of slabs



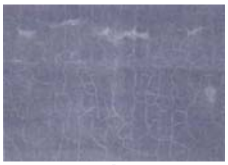
Crack in pavement surface



Cave in



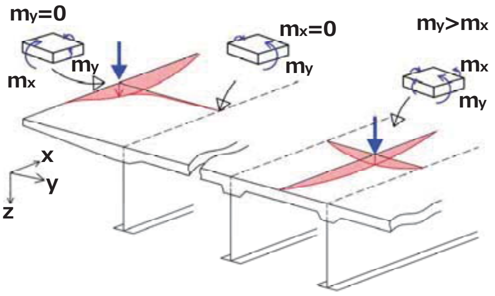
Separation of mortar and aggregate
[upperpart of slab]



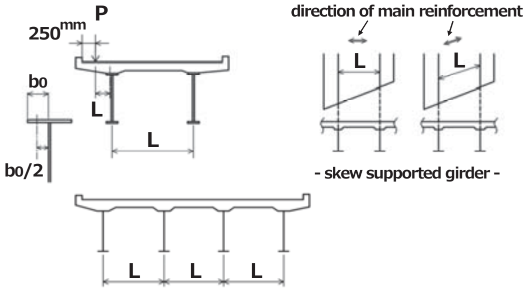
Crack net
[underneath of slab]

# Design of RC deck

## Deformation of RC slab

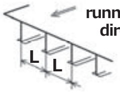
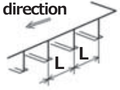


# Definition of slab span (L)



- skew supported girder -

# Minimum slab thickness (d) required

| | $d_0$（mm） | | L : span（m） |
|---|---|---|---|
| |  running direction | | running direction  |
| simple slab | 40 L + 110 | | 65 L + 130 |
| continuous slab | 30 L + 110 | | 40 L + 130 |
| cantilever slab | 0 < L < 0.25 | 280L + 160 | 240 L + 130 |
| | 0.25 < L | 80 L + 210 | |

$d\ (> d_0) = k_1\, k_2\, d_0$

# Coefficient $K_1$ and $K_2$

$k_1$ : effect of large-size truck volume

| N : Number of truck / day | $k_1$ |
|---|---|
| N < 500 | 1.10 |
| 500 ≦ N < 1,000 | 1.15 |
| 1,000 ≦ N < 2,000 | 1.20 |
| 2,000 ≦ N | 1.25 |

$k_2$（$=0.9\sqrt{M/M_0} > 1.0$）:

: effect of differential settlement

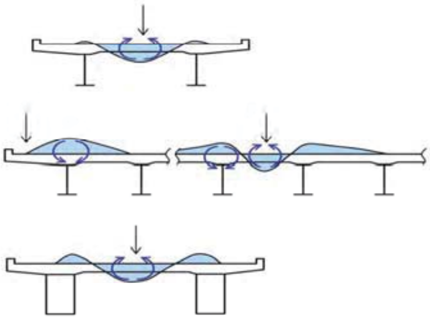
$M_0$ : design moment

M : $M_0 + \varDelta M\ (1 + i)$

⊿M : additional moment

i : impact coefficient

# Slab moment



# Design moment per unit length (1m)
## by T-load for RC slab

| | | simple slab (0 < L ≦ 4m) | continuous slab (0 < L ≦ 4m) | | | cantilevered slab (0 < L ≦ 1.5m ) | |
|---|---|---|---|---|---|---|---|
| | | at span center | at span center | at span center (end span) | at intermediate support | at support | at tip |
| dead load$^{(*)}$ (w) | | $\frac{wL^2}{8}$ | $\frac{wL^2}{14}$ | $\frac{wL^2}{10}$ | 2-span $-\frac{wL^2}{8}$<br>3-span more $-\frac{wL^2}{10}$ | $-\frac{wL^2}{2}$ | — |
| T-load | main reinforcement | (0.12L +0.07) p Ⓐ | 0.8×Ⓐ | 0.8×Ⓐ | −0.8×Ⓐ | $-\frac{PL}{1.30L+0.25}$ | — |
| | distributing reinforcement | (0.10L +0.04) p Ⓑ | 0.8×Ⓑ | 0.8×Ⓑ | — | — | (0.15L +0.13)p |

L : slab span

$p = 100^{kN}$

（$*$）: distributing direction（M=0）

## Additional（increase）rate for simple and continuous slab

| L（m） | L ≦ 2.5 | 2.5 < L ≦ 4.0 |
|---|---|---|
| coefficient | 1.0 | 1.0 +（L-2.5）/ 12 |

(direction of main reinforcement)

## Allowable stress of reinforcement

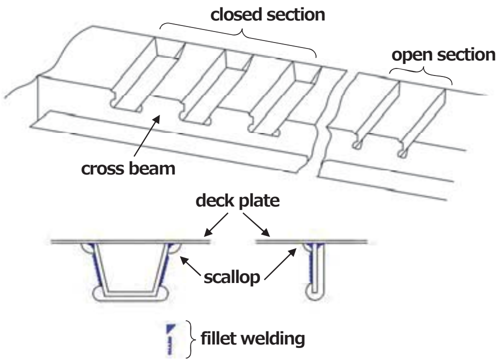
（$N/mm^2$）

| | SD345 |
|---|---|
| tension | 140 |
| compression | 200 |

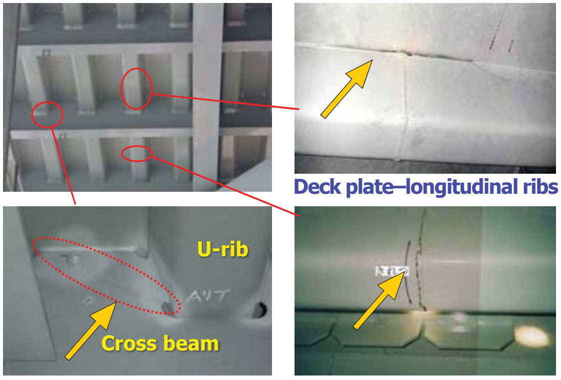
# Design of Steel deck plates

# Steel deck

# Steel deck plate



# Steel deck fatigue



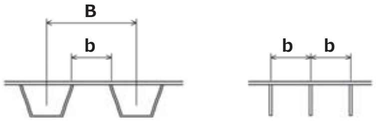
Deck plate–longitudinal ribs

Cross beam–longitudinal rib

Welding at longitudinal ribs

# Deck plate thickness and rib arrangement

$t = 0.035\,b \quad (\geqq 12^{mm})$



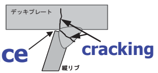
$B = 620 \sim 660^{mm}$

$b = 300 \sim 340^{mm}$

$b = 300 \sim 340^{mm}$

# Steel deck fatigue

## Penetrating to deck surface



# Recent topics (due to fatigue problem)

Mostly , $12^{mm}$ thickness has been used so far.

Due to severe fatigue damage,

**$16^{mm}$ thickness is recommended**

## Impact ( i ) for the design

longitudinal ribs $i = 0.4$

cross beams $i = \dfrac{20}{50 + L}$

L : span of cross beams

## Additional increase rate ( k ) for cross beams

$k = k_0 \qquad (L \leqq 4)$

$k = k_0 - (k_0 - 1) \times (L - 4) / 6 \qquad (4 < L \leqq 10)$

$k = 1.0 \qquad (10 < L)$

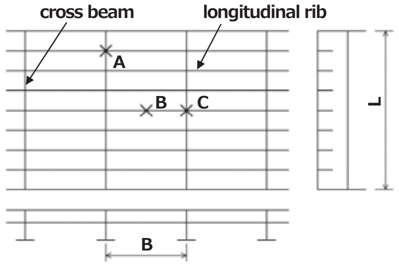
$k_0 = 1.0 \qquad (B \leqq 2)$

$k_0 = 1.0 + 0.2 \times (B - 2) \qquad (2 < B \leqq 3)$

$k_0 = 1.2 \qquad (3 < )$

B : distance of cross beams

# Calculation of stress resultants (grid model)

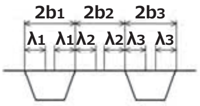


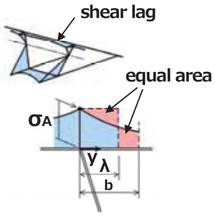
Pts. A,B,C : Observation points

Pts. A,B : for designing longitudinal ribs

Pt. C : for designing cross beams

# Effective Width





$\lambda = b \qquad (b/L_e \leqq 0.02)$

$\lambda = \left\{1.06 - 3.2\left(\dfrac{b}{L_e}\right) + 4.5\left(\dfrac{b}{L_e}\right)^2\right\} b \qquad (0.02 < b/L_e < 0.30)$

$\lambda = 0.15 L_e \qquad (0.30 \leqq b/L_e)$

$L_e$ : equivalent span length

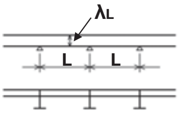
effective width ( λ ) is introduced to catch ( $\sigma_A$ )

$\lambda \sigma_A = \int_0^b \sigma(y)\,dy$

# Equivalent span length（Le）

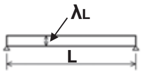
**longitudinal ribs**

$\lambda_L$（Le = 0.6 L）



**cross beams**

$\lambda_L$（Le = L）



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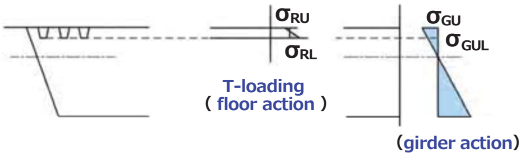
# Stress evaluation

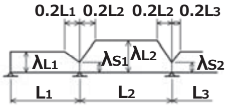


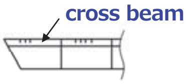
**allowable stress** （N/mm²）

| | SM400<br>SMA400W | SM490 | SM490Y<br>SM520<br>SMA490W | SM570<br>SMA570W |
|---|---|---|---|---|
| t ≦ 40 | 195 | 260 | 295 | 355 |
| 40 < t ≦ 75 | 175 | 245 | 275 | 345 |
| 75 < t ≦ 100 | | | 265 | 335 |

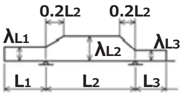
(↑ in case of combined stress check )

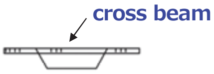
---





$\lambda_{L1}$（Le = 0.8 $L_1$）
$\lambda_{S1}$（Le = 0.2（$L_1+L_2$））
$\lambda_{L2}$（Le = 0.6 $L_2$）
$\lambda_{S2}$（Le = 0.2（$L_2+L_3$））





$\lambda_{L1}$（Le = 2 $L_1$）
$\lambda_{L2}$（Le = 0.2（$L_1+L_2$））
$\lambda_{L3}$（Le = 2 $L_3$）

---

# Plate theory

---

# Basic assumption

[1] slab thickness (t) is constant.

[2] Hook's law* is applied.
*stress-strain relation is proportional ($\sigma = E\varepsilon$)

[3]displacement (w) is enough small
compared to slab thickness (t)*
*$w = w(x,y) << t$

---

# From equilibrium condition ,

$$\frac{\partial q_x}{\partial x} + \frac{\partial q_y}{\partial y} + p_z = 0 \quad \text{——— (1)}$$

$$\frac{\partial m_x}{\partial x} + \frac{\partial m_{xy}}{\partial y} - q_x = 0 \quad \text{——— (2)}$$

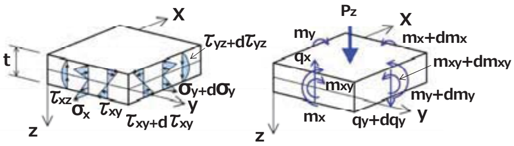
$$\frac{\partial m_{xy}}{\partial x} + \frac{\partial m_y}{\partial y} - q_y = 0 \quad \text{——— (3)}$$

$$\frac{\partial}{\partial y}\text{(eq.（2））} \ \& \ \frac{\partial}{\partial x}\text{(eq.（3））} \xrightarrow{\text{substitute}} \text{eq.（1）}$$

$$\frac{\partial^2 m_x}{\partial x^2} + 2\frac{\partial^2 m_{xy}}{\partial x \partial y} + \frac{\partial^2 m_y}{\partial y^2} = -p_z \quad \text{——— (4)}$$

---

# Stress and stress resultants



$$m_x = \int \sigma_x z dA, \qquad m_y = \int \sigma_y z dA$$

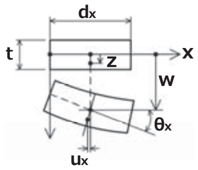
$$m_{xy} = \int \tau_{xy} z dA$$

$$q_x = \int \tau_{xz} dA, \qquad q_y = \int \tau_{yz} dA$$

$$\left( \int dA \rightarrow \int_{-t/2}^{t/2} dA \right)$$

---

# Strain-displacement relation



$$u_x = -z\sin\theta_x \fallingdotseq -z\frac{\partial w}{\partial x}$$

$$u_y = -z\sin\theta_y \fallingdotseq -z\frac{\partial w}{\partial y}$$

$$\varepsilon_x = \frac{\partial u_x}{\partial x} = -z\frac{\partial^2 w}{\partial x^2} \quad (= -zw,_{xx})$$

$$\varepsilon_y = \frac{\partial u_y}{\partial y} = -z\frac{\partial^2 w}{\partial y^2} \quad (= -zw,_{yy})$$

$$\gamma_{xy} = \frac{\partial u_x}{\partial y} + \frac{\partial u_y}{\partial x} = -2z\frac{\partial^2 w}{\partial x \partial y} \quad (= -2zw,_{xy})$$

# Stress-displacement relation

$$\begin{Bmatrix} \sigma x \\ \sigma y \\ \tau xy \end{Bmatrix} = \frac{E}{1-\nu^2} \begin{bmatrix} 1 & -\nu & 0 \\ -\nu & 1 & 0 \\ 0 & 0 & \frac{1-\nu}{2} \end{bmatrix} \begin{Bmatrix} \varepsilon x \\ \varepsilon y \\ \gamma xy \end{Bmatrix}$$

E : Young's modulus of elasticity
$\nu$ : Poisson's ratio

$$\sigma x = -\frac{E_z}{1-\nu^2}(w,_{xx} + \nu w,_{yy})$$

$$\sigma y = -\frac{E_z}{1-\nu^2}(\nu w,_{xx} + w,_{yy})$$

$$\tau xy = -2G_z w,_{xy}$$

---

Substituting eq. (5) into eq. (4),

$$\frac{\partial^4 w}{\partial x^4} + 2\frac{\partial^4 w}{\partial x^2 \partial x^2} + \frac{\partial^4 w}{\partial x^4} = \frac{P_z}{B}$$

↑ **fundamental equation of plate**

**Stress** $\sigma_x = \frac{M_x}{I} z$

$$\sigma_y = \frac{M_y}{I} z$$

$$\tau_{xy} = \frac{M_{xy}}{I} z$$

$$I = \frac{t^3}{12}$$

---

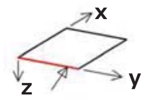
$$\left.\begin{aligned} m_x &= \int \sigma_x z dA = -B(w,_{xx} + \nu w,_{yy}) \\ m_y &= \int \sigma_y z dA = -B(\nu w,_{xx} + w,_{yy}) \\ m_{xy} &= \int \tau_{xy} z dA = -\frac{Gt^3}{6} w,_{xy} = -(1-\nu) B w,_{xy} \\ q_x &= \int \tau_{xz} dA = -B(w,_{xxx} + w,_{xyy}) \end{aligned}\right\} (5)$$

$$q_y = \int \tau_{yz} dA = -B(w,_{yyy} + w,_{xxy})$$

$$B = \frac{Et^3}{12(1-\nu^2)}$$

---

# Boundary conditions



1) Simple support

$w = w,_{y} = w,_{x} = 0$

$m_x = 0 \longrightarrow \Delta w = w,_{xx} + w,_{yy} = 0$

2) fix support

$w = w,_{x} = w,_{y} = w,_{yy} = 0$

$m_{xy} = 0$

3) free

$m_x = \overline{q}_x = 0$

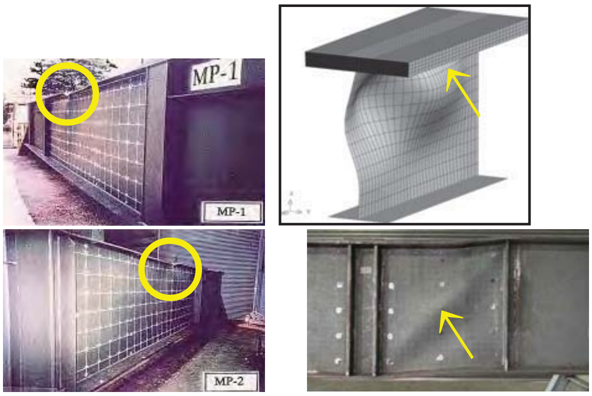
$(\overline{q}_x = q_x + m_{xy},_{y} \;,\; \overline{q}_y = q_y + m_{xy},_{x})$

[11-5-2]

# Design of Girder Bridges

---

## Local buckling of compressed plate and web



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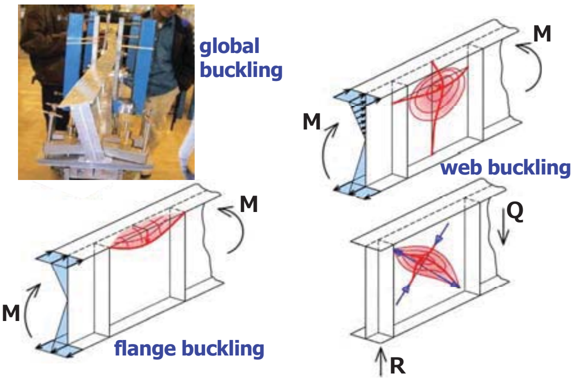
## Buckling of girder and plates



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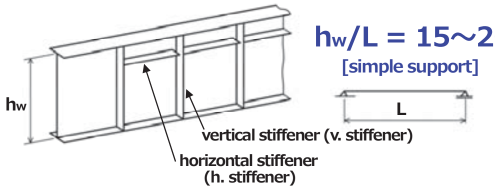
## Minimum web thickness



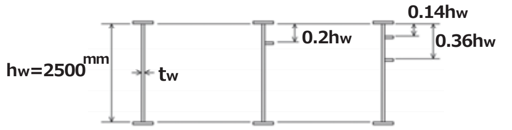
$h_w/L = 15\sim20$

[simple support]

| | SS400<br>SM400<br>SMA400W | SM490 | SM490Y<br>SM520<br>SMA490YW | SM570<br>SMA570W |
|---|---|---|---|---|
| no h. stiffener | hw/152 | hw/130 | hw/123 | hw/110 |
| one h. stiffeners | hw/256 | hw/220 | hw/209 | hw/188 |
| two h. stiffeners | hw/310 | hw/310 | hw/294 | hw/262 |

---

## Ex. (SM490Y)



$t_{w,\,min.} = \frac{h_w}{123} = \frac{2{,}500}{123} = 20.3^{mm} \longrightarrow \underline{21^{mm}}$

$t_{w,\,min.} = \frac{h_w}{209} = 11.97^{mm} \longrightarrow \underline{12^{mm}}$

$t_{w,\,min.} = \frac{h_w}{294} = 8.5^{mm} \longrightarrow \underline{9^{mm}}$

---

When span (L) becomes longer,
web depth ($H_w$) becomes higher.
($H_w/L$ : 15~20 {simple span})

Thickness of web without stiffeners
becomes considerably large.

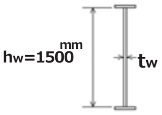
→ To avoid thick web,
stiffeners (H & V) are employed
to prevent buckling.

---

## $h_w$ without vertical stiffener

| | SS400<br>SM400<br>SMA400W | SM490 | SM490Y<br>SM520<br>SMA490W | SM570<br>SMA570W |
|---|---|---|---|---|
| min.hw | 70tw | 60tw | 57tw | 50tw |

ex. (SM400)



$t_w > \frac{h_w}{70} = 21.4^{mm}$

$\longrightarrow \underline{22^{mm}}$

more than 22mm ,
no v. stiffener allowed

(SM490Y)

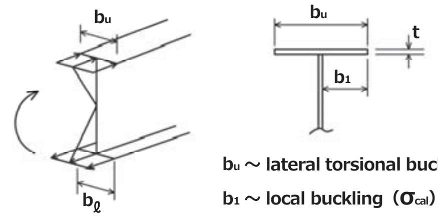
$t_w > \frac{h_w}{57} = 26.3^{mm}$

$\longrightarrow \underline{27^{mm}}$

more than 27mm ,
no v. stiffener allowed

---

## Flange in compression



$b_u \sim$ lateral torsional buckling ($\sigma_{ba}$)

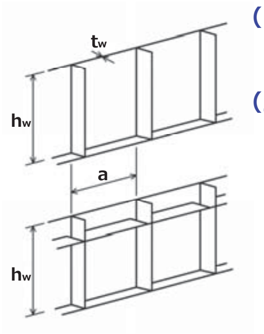
$b_1 \sim$ local buckling ($\sigma_{cal}$)

$\underline{\sigma_{ult.}} = \min.\{\sigma_{ba}, \sigma_{cal}\}$
strength of beams

In tension

$t > \frac{b_1}{16}$

# Design of web



## (1) Horizontal stiffeners

See PPT No. 5

## (2) Vertical stiffeners

$$\left(\frac{\sigma}{\sigma_E}\right)^2 + \left(\frac{\tau}{\tau_E}\right)^2 \leqq \frac{1}{\gamma^2}$$

$\sigma_E = k\sigma(=23.9) \cdot \sigma_{E0}$

$\tau_E = k\tau \cdot \sigma_{E0}$

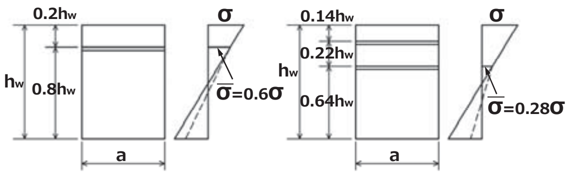
$\gamma = 1.25$

↓

verification formula

---

# In case of the web with stiffener



one h. stiffener

$B = 0.8h_w$

$\bar{\sigma} = 0.6\sigma$

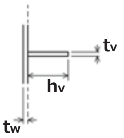
two h. stiffener

$B = 0.64h_w$

$\bar{\sigma} = 0.28\sigma$

$\left.\begin{array}{l} B \rightarrow h_w \\ \bar{\sigma} \rightarrow \sigma \end{array}\right\} \rightarrow$ formula without stiffener

---



$I_v = \frac{t_v h_v^3}{3}$

$I_v > \frac{h_w t_w^3}{11} \cdot \gamma_{v,req.}$

$\gamma_{v,req.} = 8.0 \left(\frac{h_w}{a}\right)^2$

## Check of shear strength of web

[ex. In case of no horizontal stiffener]

$$\left(\frac{h_w}{100t_w}\right)^4 \left[\left(\frac{\sigma}{345}\right)^2 + \left\{\frac{\tau}{77+58(h_w/a)^2}\right\}^2\right] \leqq 1.0 \quad (a/h_w > 1.0)$$

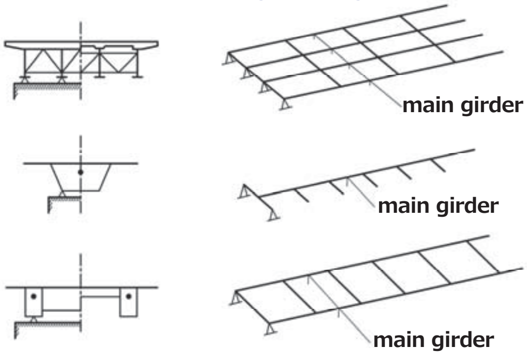
$$\left(\frac{h_w}{100t_w}\right)^4 \left[\left(\frac{\sigma}{345}\right)^2 + \left\{\frac{\tau}{58+77(h_w/a)^2}\right\}^2\right] \leqq 1.0 \quad (a/h_w \leqq 1.0)$$

must be satisfied

---

# Stress resultants (M, Q)

are calculated using following model



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# Safety check

## (1a) normal stress ($\sigma_b$) of I-girder

$\sigma_b = (M/I) \cdot y \leqq \sigma_a$

$\sigma_a = \min. \{\sigma_{ba}, \sigma_{cal}\}$

M : bending moment
I : geometrical moment of inertia
$\sigma_{ba}$ : allowable flexural compressive stress
$\sigma_{cal}$ : allowable local buckling stress

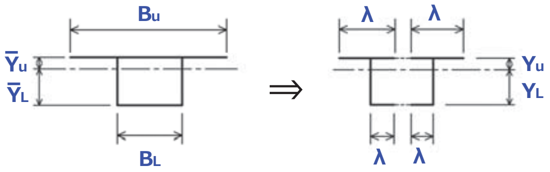
---

# (1b) normal stress ($\sigma_b$) of box girder

$\sigma_b = (M/I) \cdot y \leqq \sigma_{ba}$

$\sigma_{ba} = \underline{\sigma_y/1.7}$
↑ no lateral torsional buckling

I : geometrical moment of inertia
(calculate using effective width λ)



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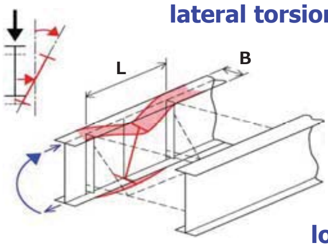
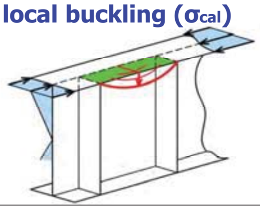
## lateral torsional buckling ($\sigma_{ba}$)

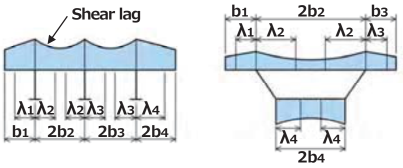


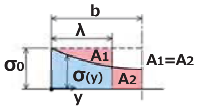
## local buckling ($\sigma_{cal}$)



---

# Effective width (λ)





$\sigma_0 \lambda = \int_0^b \sigma(y)\,dy$

$\lambda = \int_0^b \sigma(y)\,dy \,/\, \sigma_0$

λ : for evaluating peak stress ($\sigma_0$)

$$\left.\begin{array}{ll}\lambda = b & \left(\frac{b}{L_e} \leqq 0.05\right)\\ \lambda = \left\{1.1 - 2\left(\frac{b}{L_e}\right)\right\} b & \left(0.05 < \frac{b}{L_e} < 0.30\right)\\ \lambda = 0.15L_e & \left(0.30 \leqq \frac{b}{L_e}\right)\end{array}\right\} - (1)$$

- parabolic -

$$\left.\begin{array}{ll}\lambda = b & \left(\frac{b}{L_e} \leqq 0.02\right)\\ \lambda = \left\{1.06 - 3.2\left(\frac{b}{L_e}\right) + 4.5\left(\frac{b}{L_e}\right)^2\right\} b & \left(0.02 < \frac{b}{L_e} < 0.30\right)\\ \lambda = 0.15L_e & \left(0.30 \leqq \frac{b}{L_e}\right)\end{array}\right\} - (2)$$

- straight -

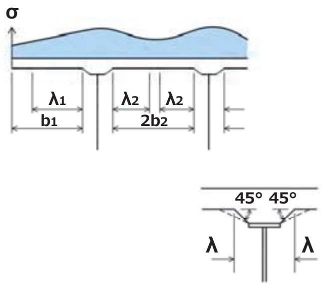
⇑

moment distribution

$L_e$ : equivalent span length

---

# Effective width of concrete slab



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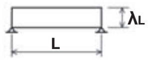
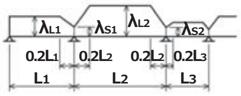
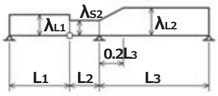


$\lambda_L$ ($L_e = L$) by eq. (1)



$\lambda_{L1}$ ($L_e = 0.8L$) by eq. (1)
$\lambda_{L2}$ ($L_e = 0.6L$) 〃
$\lambda_{S1}$ ($L_e = 0.2\,(L_1+L_2)$) by eq. (2)
$\lambda_{S2}$ ($L_e = 0.2\,(L_2+L_3)$) 〃



$\lambda_{L1}$ ($L_e = L_1$) by eq. (1)
$\lambda_{L2}$ ($L_e = 0.8L_3$) by eq. (1)
$\lambda_{S2}$ ($L_e = 2L_2$) by eq. (2)

---

## ( 2 ) shear stress ($\tau_b$) in flexure

$$\tau_b = \frac{Q}{A_w} < \tau_a\ (= \tau_y / 1.7)$$

$Q$ : shear force
$A_w$ : cross sectional area of webs
$\tau_a$ : allowable shear stress
$\tau_y$ : shear yield stress ($= \sigma_y / \sqrt{3}$ )

※ in case of checking flange, shear stress based on shear flow theory is recommended

---

## ( 3 ) normal and shear stresses ($\sigma_w, \tau_s, \tau_w$) in torsion

in case of I-section, ($\sigma_w, \tau_s, \tau_w$) can be neglected.

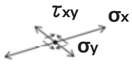
in case of box-section, ($\sigma_w, \tau_w$) can be neglected.

$\sigma_w$ : warping stress
$\tau_s$ : St.Venant shear stress (pure torsion)
$\tau_w$ : shear stress due to warping torsion

---

## ( 6 ) bi-axial stress ($\sigma_x, \sigma_y, \tau_{xy}$) check

$$\left(\frac{\sigma_x}{\sigma_a}\right)^2 - \left(\frac{\sigma_x}{\sigma_a}\right)\left(\frac{\sigma_y}{\sigma_a}\right) + \left(\frac{\sigma_y}{\sigma_a}\right)^2 + \left(\frac{\tau_{xy}}{\tau_a}\right)^2 < 1.2$$



## Mises stress ($\sigma_e$)

10%up

$$\sigma_e = \sqrt{\sigma_x^2 - \sigma_x\sigma_y + \sigma_y^2 + 3\tau_{xy}^2} < \underline{1.1\sigma_a}$$

$$\left(\frac{\sigma_x}{\sigma_a}\right)^2 - \left(\frac{\sigma_x}{\sigma_a}\right)\left(\frac{\sigma_y}{\sigma_a}\right) + \left(\frac{\sigma_y}{\sigma_a}\right)^2 + 3\left(\frac{\tau_{xy}}{\sigma_a}\right)^2 < 1.2$$

$$\tau_y = \tilde{\sigma}_y / \sqrt{3} \longrightarrow \sigma_a = \sqrt{3}\tau_a \ (\tilde{\sigma}_y : \text{yield stress})$$

$$\left(\frac{\sigma_x}{\sigma_a}\right)^2 - \left(\frac{\sigma_x}{\sigma_a}\right)\left(\frac{\sigma_y}{\sigma_a}\right) + \left(\frac{\sigma_y}{\sigma_a}\right)^2 + \left(\frac{\tau}{\tau_a}\right)^2 < 1.2$$

---

## ( 4 ) combined stress ($\sigma_b, \tau_b$) check

$$\left(\frac{\sigma_b}{\sigma_a}\right)^2 + \left(\frac{\tau_b}{\tau_a}\right)^2 < 1.2^*$$

$\sigma_b < \sigma_a$
$\tau_b < \tau_a$

## ( 5 ) with torsional moment

$$\left(\frac{\sigma}{\sigma_a}\right)^2 + \left(\frac{\tau}{\tau_a}\right)^2 < 1.2^*$$

$\sigma < \sigma_a$
$\tau < \tau_a$

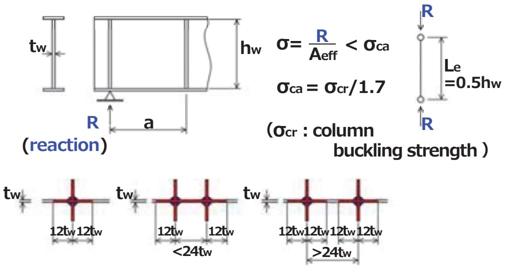
$\sigma = \sigma_b + \sigma_w$
$\tau = \tau_b + \tau_s + \tau_w$

* take into account that loading conditions for $\sigma_{max.}$ , $\tau_{max}$ are different

---

# Design of stiffeners

# Vertical stiffener at support



$\sigma = \dfrac{R}{A_{eff}} < \sigma_{ca}$

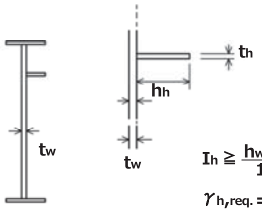
$\sigma_{ca} = \sigma_{cr}/1.7$

($\sigma_{cr}$ : column buckling strength )

Effective area（$A_{eff} < 1.7A_{stiffener}$）

$A_{eff} = A_{web} + A_{stiffener}$

# Horizontal stiffeners



$I_h = \dfrac{t_h h_h^3}{3}$

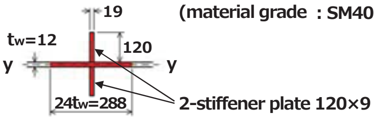
$I_h \geqq \dfrac{h_w t_w^3}{11} \cdot \gamma_{h,req.}$

$\gamma_{h,req.} = 30\ (a/h_w)$

a : distance of vertical stiffeners

hw : depth of the web

# Example



(material grade ：SM400)

$A_{web} = 288\times12 \quad = 34.^{56}$

$A_{rib} = 2\times120\times19 = 45.^{60}$

$80.16^{(cm^2)} < 1.7A_{rib} = 82.08^{(cm^2)}$

$I_y = 2{,}534^{cm^4},\ r_y = 5.62^{cm}$

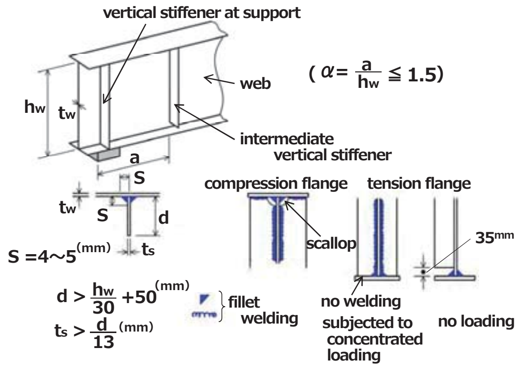
$L_e/r_y = 0.5\times160^{(cm)}/\ 5.62 = 14.2 \quad (h_w : 1600^{mm})$

$R = 879.4\ (\text{kN})$

$\sigma_C = \dfrac{879.4\times10^3}{80.16\times10^2} = 109.0^{(N/mm^2)} < \sigma_{ca}\ (\text{ok})$

$(L_e/r_y < 18 \rightarrow \sigma_{ca} = 140\ ^{N}/_{mm^2})$

# Intermediate vertical stiffeners

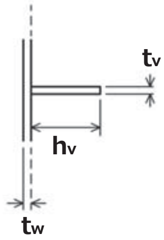


$(\alpha = \dfrac{a}{h_w} \leqq 1.5)$

$S = 4\sim5^{(mm)}$

$d > \dfrac{h_w}{30} + 50^{(mm)}$

$t_s > \dfrac{d}{13}\ ^{(mm)}$



$I_v = \dfrac{t_v h_v^3}{3}$

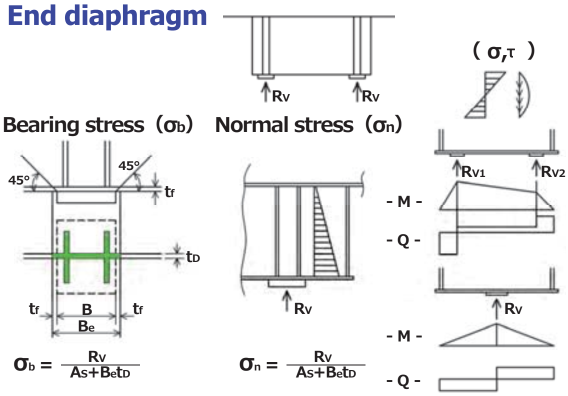
$I_v > \dfrac{h_w t_w^3}{11} \cdot \gamma_{v,req.}$

$\gamma_{v,req.} = 8.0\ (\dfrac{h_w}{a})^2$

a : distance of vertical stiffeners

hw : depth of the web

# End diaphragm



$\sigma_b = \dfrac{R_V}{A_S + B_e t_D}$

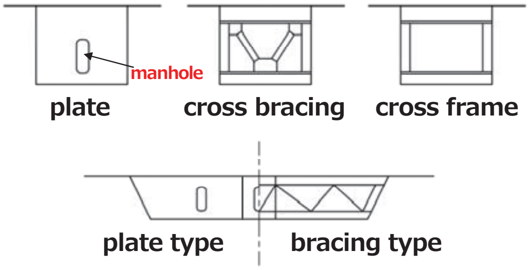
$\sigma_n = \dfrac{R_V}{A_S + B_e t_D}$

As : cross-sectional area of the stiffeners

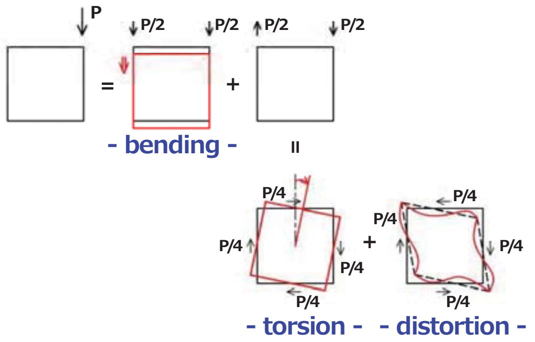
# Design of support diaphragms & intermediate diaphragms

# Intermediate diaphragms

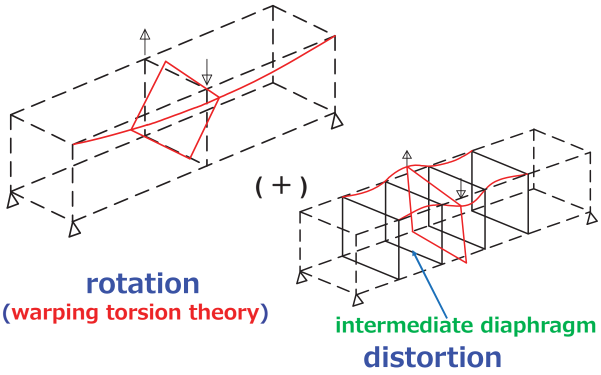
## (to prevent distortion)

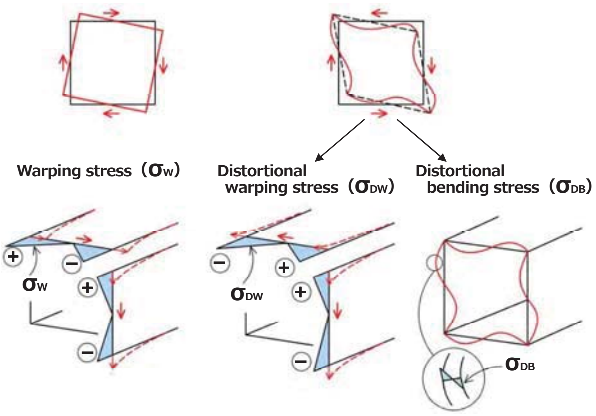


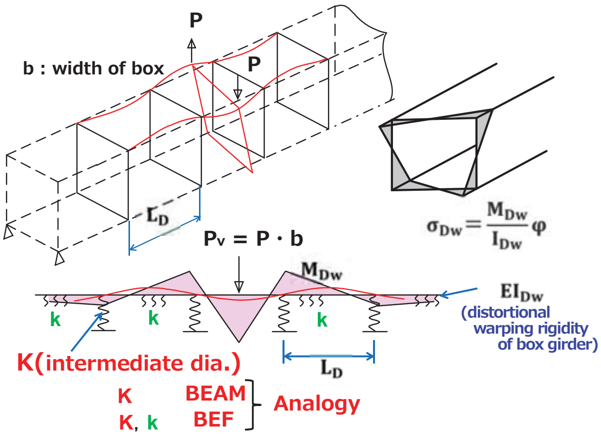
## Torsion and distortion under eccentric loading



## Torsion and distortion





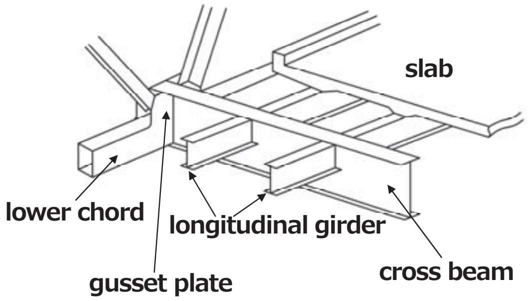


$$\sigma_{Dw} = \frac{M_{Dw}}{I_{Dw}} \varphi$$

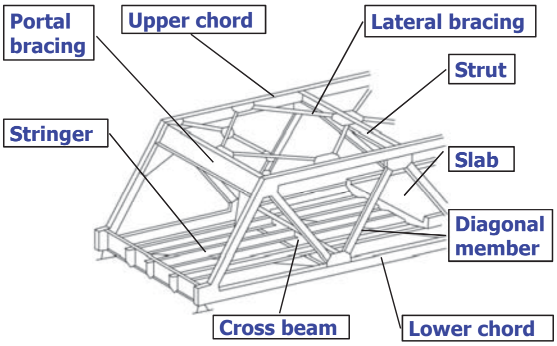
[11-5-3]

# Design of Truss Bridges

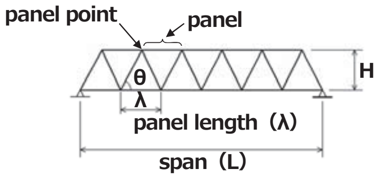
## Members



## Name of Truss Members



## Preliminary design



$H/L = 1/6 \sim 1/8$

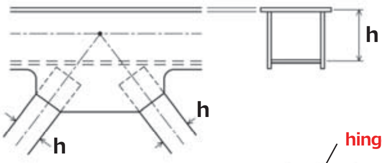
$\theta = 40 \sim 60°$

$\lambda = 6 \sim 10^{m}$

$\delta^{*} < L / 600$

＊deflection by live load（excluding impact）

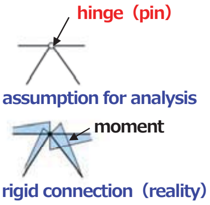
## h/λ for pin structure



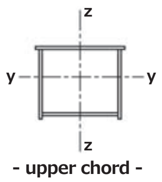
$$\frac{h}{\lambda} = \frac{1}{15} \sim \frac{1}{20}$$

λ : panel length

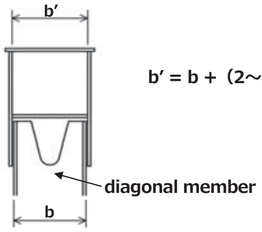
to reduce moment



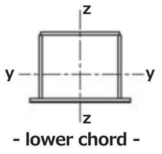
If (h/λ) is large, rigid frame model is used.



- upper chord -

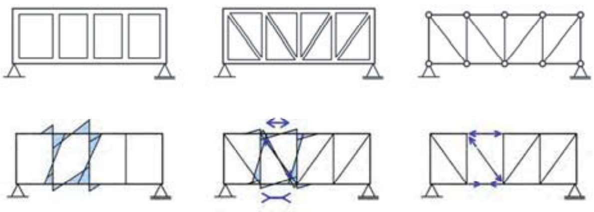


$b' = b + (2\sim3)^{mm}$



- lower chord -

- For the compressive member, box section is used and the condition
  $\lambda_z\ (=L_e/r_z) < \lambda_y\ (=L_e/r_y)$
  must be satisfied.
  λ : slenderness ratio
- $(A_w)$ must be greater than $(0.4A_g)$.
  $A_w$ : cross-sectional area of the web
  $A_g$ : cross-sectional area of the chord



Bending moment (M)
Shear force（Q）

- Vierendeel Br. -

Bending moment(M)
Shear force(Q)
Axial force (N)

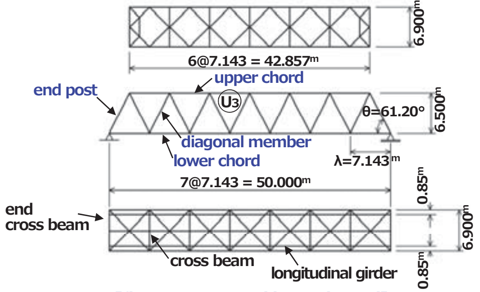
- Rigid frame structure -

Axial force (N)

- Truss -

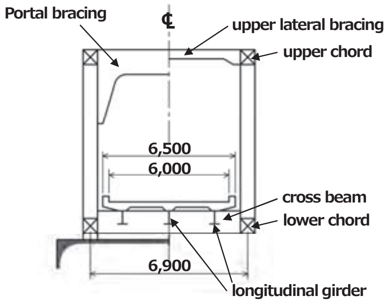
## Frame model

[ upper lateral ]

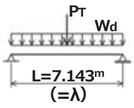


[floor system and lower lateral]

## Cross section



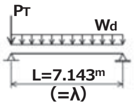
## Maximum bending moment and shear force



$$M_d = \frac{W_d L^2}{8}$$

$$M_{L+i} = \frac{P_T L}{4}(1+i)$$

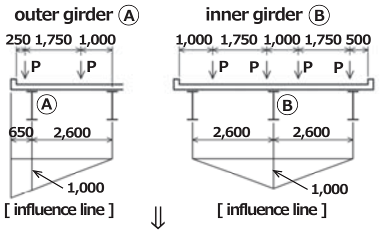
$$i = \frac{20}{50+L} = \frac{20}{50+7.143} = 0.35$$



$$Q_d = \frac{W_d L}{2}$$

$$Q_{L+i} = P_T(1+i)$$

## Design of longitudinal girder



⇓

Load intensity

$W_d$, Ⓐ
$W_d$, Ⓑ } dead load
$P_T$, Ⓐ
$P_T$, Ⓑ } live load

## Stress check

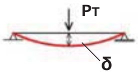
$$\sigma = \frac{(M_d + M_{L+i})}{I} y < \sigma_a (= \sigma_y / 1.7)$$

since no possibility of buckling

$$\tau = \frac{Q_d + Q_{L+i}}{A_w} < \tau_a (= \tau_y / 1.7)$$

(Plus design of vertical stiffener)

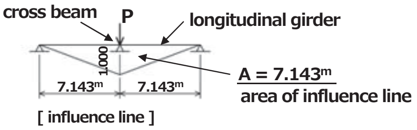
## Deflection check



$$\delta < \frac{L}{2{,}000} \; (L < 10^m)$$

## Design of cross beam

Span of cross beam is assumed to be distance between chord member

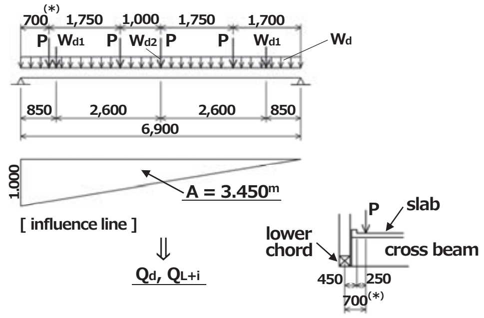


$W_{d,1}$ (outer girder) = $W_d$, Ⓐ × A

$W_{d,2}$ (inner girder) = $W_d$, Ⓑ × A

$W_d$ : Self weight of cross beam

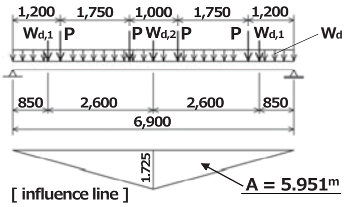
## Shear force



⇓

$Q_d$, $Q_{L+i}$

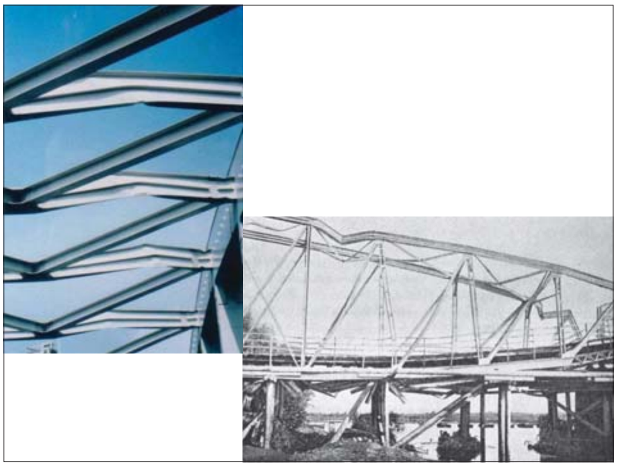
## Maximum bending moment



⇓

$M_d$, $M_{L+i}$

## Design of chord and web members



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# Maximum allowable slenderness ratio*

| | | $L^{**}/r$ |
|---|---|---|
| compression | main member | 120 |
| | secondary member*** | 150 |
| tension | main member | 200 |
| | secondary member | 240 |

* to ensure bridge global rigidity

** effective buckling length (in compression)
panel length (in tension)

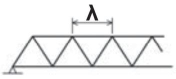
*** members in cross or lateral bracing

---

# Effective buckling length (Le)

**[ Chord member ]**

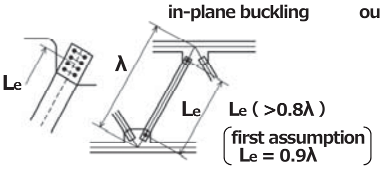
in-plane & out-of-plane buckling  $L_e = \lambda$ (panel length)



**[ Web member ]**

in-plane buckling  out-of-plane buckling

$L_e = \lambda$



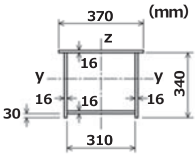
$L_e$ ( $>0.8\lambda$ )

first assumption
$L_e = 0.9\lambda$

---

# Ex. Design of upper chord

(ex.) Upper chord $U_3$ ( Axial force = $-2370.1^{kN}$ )



$A = 217.6^{cm^2}$
$I_y = 37{,}151^{cm^4}$
$I_z = 39{,}633^{cm^4} > I_y$
( SM400 )

local buckling of plate  $b/t = 31/1.6 = 19.4 < 38.7$ (ok)

global buckling of member

$$\underline{L_e}/r = \underline{714.3} / \sqrt{37{,}151/217.6} = 54.6$$

$$\sigma_{ca} = 140 - 0.82\,(54.6 - 18) = 110^{N/mm^2}$$

$$\sigma = \frac{2370.1\times10^3}{217.6\times10^2} = 108.9^{\,N/mm^2} < \sigma_{ca} \ \text{(ok)}$$

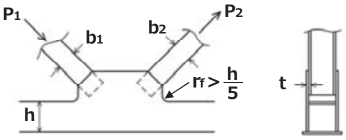
---

# Design of gusset plate

$t$ (plate thickness, mm) $> 2 \times \frac{P}{b}$

P : maximum force of end post or web member (kN)

b : width of end post or web member (mm)



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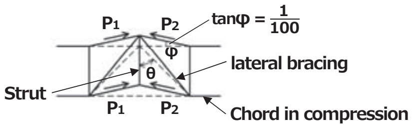
Strut and lateral bracing members attached to chord in compression have to be designed to resist the following loads

Strut : $\frac{P_1 + P_2}{100}$

lateral bracing : $\frac{P_1 + P_2}{100}\sec\theta$



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# Design of lateral bracing members

## buckling length

$L_e = 0.9\lambda$ ( λ : panel length)

( *from conservative viewpoint, $L_e = \lambda$ )

## max. allowable slenderness ratio

in compression  $L_e/r < 150$
in tension  $\lambda/r < 240$

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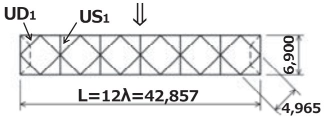
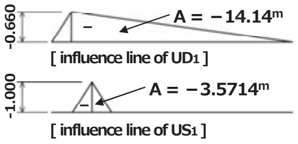
# Design of upper lateral bracing

wind load ( > earthquake load )



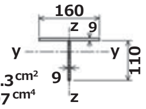


$N = \pm\,\underline{3.80}\times14.14$
↑wind load ( kN/m )
$= \pm\,53.73^{(kN)}$

$N = -3.80\times3.5714$
$= -13.57^{(kN)}$

# UD1

**– in compression –**



$A = 24.3^{cm^2}$
$I_y = 307^{cm^4}$
$I_z = 307^{cm^4}$

check of slenderness ratio

$$L_e/r_y = 496.5/\sqrt{307/24.3} = \underline{140 < 150}$$

$$\sigma_{cay} = \frac{1,200,000}{6,700+140^2} = 45.6^{(N/mm^2)}$$

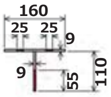
$$\sigma'_{cay} = 45.6\ (0.5+\frac{140}{1,000}) = 29.1^{(N/mm^2)}$$

$$\sigma_c = \frac{53.73\times10^3}{24.3\times10^2} = 22.1^{(N/mm^2)} < \underline{1.2}\sigma'_{cay} = 34.9^{(N/mm^2)}$$

↑ extra coefficient

(ok!!)

**– in tension –**



$$\sigma_t = \frac{53.73\times10^3}{1,485} = 36.2^{(N/mm^2)} < 1.2\sigma_{ta}$$
$$= 1.2\times140 = 168^{(N/mm^2)}$$

$$A_n = (160-2\times25)\times9+\frac{110}{2}\times9 = 1,485^{cm^2}$$

$A_n$ : net cross sectional area

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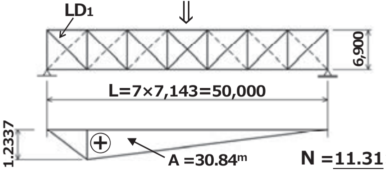
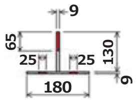
# Design of lower lateral bracing

earthquake load （ > wind load ）



$$N = \underline{11.31}\times30.84 = 348.8^{(kN)}$$

↑ earthquake load (kN/m)



$$\sigma_t = \frac{348.8\times10^3}{1,755} = 198.8^{(N/mm^2)} < 1.5\sigma_{ta}$$
$$= \underline{1.5}\times140 = 210^{(N/mm^2)}$$

↑ extra coefficient

$$A_n = (180-2\times25)\times9+65\times9 = 1,755^{mm^2}$$

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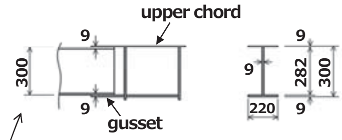
# US1



$\sigma_c < \sigma_a$ （ $L_e = 6,900^{mm}$ ）

＊height of strut in lower deck type bridge shall have the same height of the chord

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# Pony Truss Bridges

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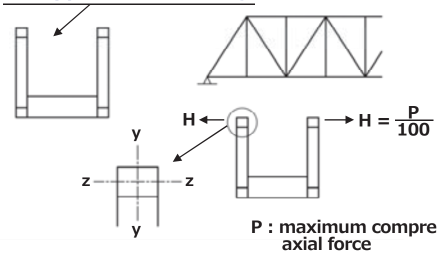
# Design of Pony truss

no upper lateral bracing



$$H = \frac{P}{100}$$

$r_y \geq 1.5r_x$

P : maximum compressive axial force

Safety shall be checked under H loading