Deteriorations of Road Infrastructures

Institute of Industrial Science
The University of Tokyo
Kohei NAGAI

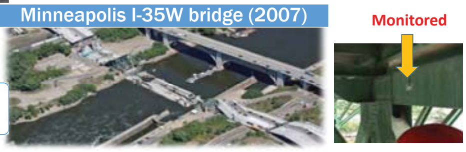
Minneapolis I-35W bridge

"America in Ruins"

Silver Bridge (1967)

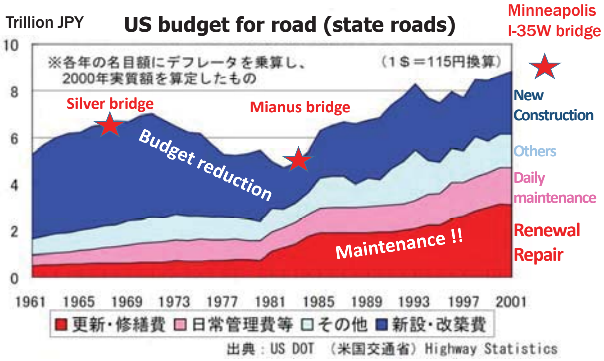
Mianus River Bridge (1983)

Minneapolis I-35W bridge (2007)

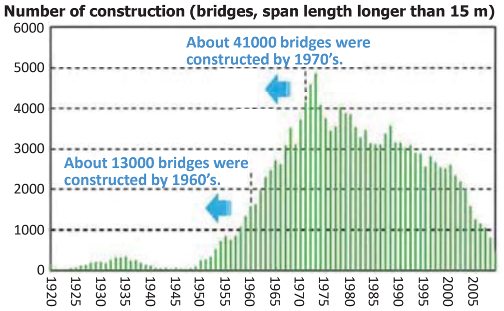


Many bridges in the US built in New Deal in 1930s were damaged for inappropriate maintenance.

Background - America in Ruins



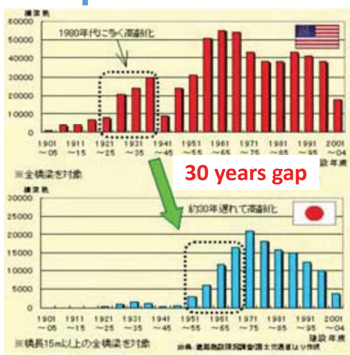
Aging of Infrastructures in Japan



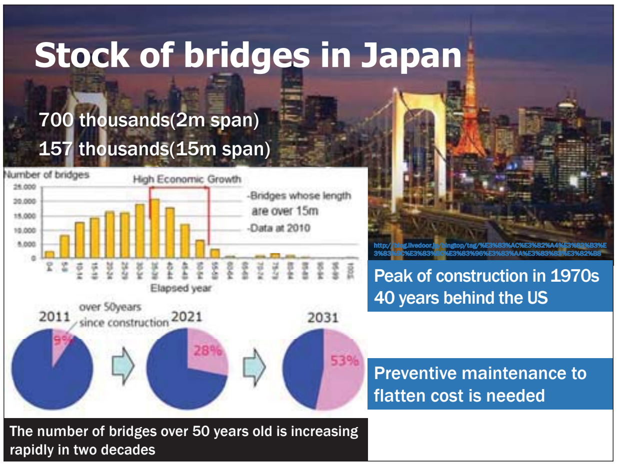
By Ministry of Land, Infrastructure, Transport and Tourism of Japan

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

Japan in ruins?



Will Japanese bridges also collapse?
Maintenance becomes important.



Examples of damage and repair

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

Corrosion

-damage-

-repair-

At bridge end

-bridge Full view-

- State of the outer surface of the girder -

-Corrosion state of the girder inner surface-

Corrosion

-damage-

-repair-

Fillet welding between web and lower flange

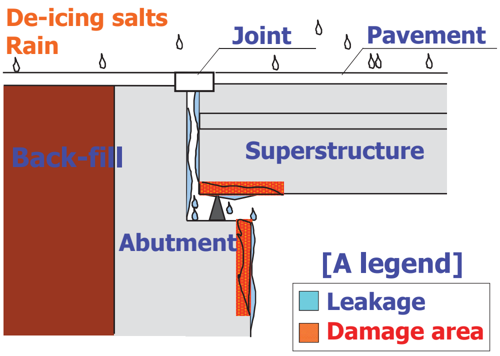
Corrosion

Breakage of diagonal Member in truss bridge

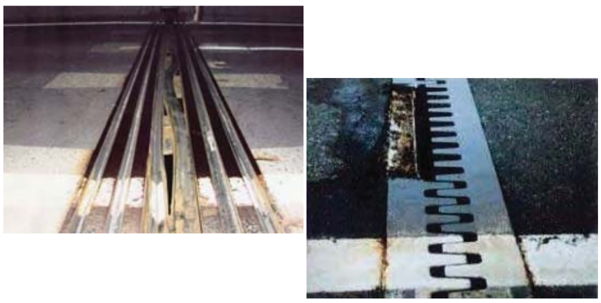
-damage-

-repair-

Water Leakage From Expansion Joint



Damage of Expansion Joint



fatigue

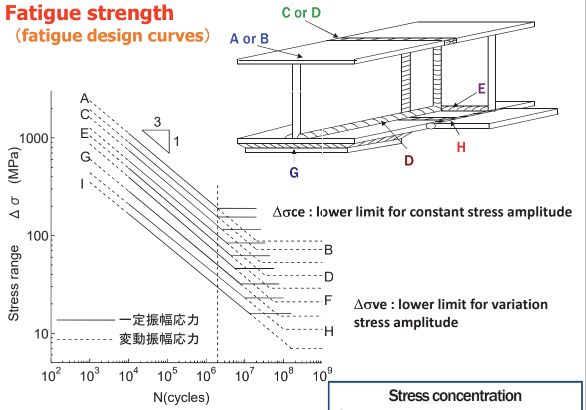
-fatigue crack-

-repair-

2006年10月2日発見、名阪国道「山添橋」

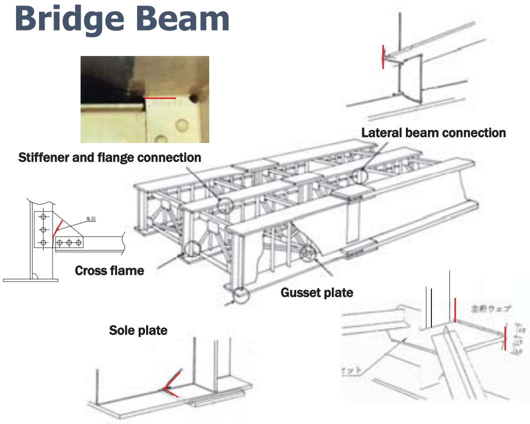
中央線第2平等川橋梁（山梨県石和町）2004年

Fatigue strength
（fatigue design curves）

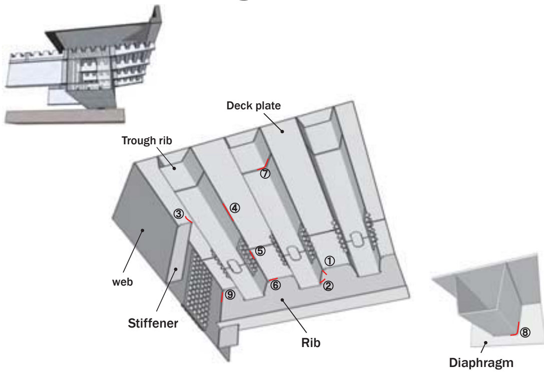


Stress concentration
Stress at the crack propagating point

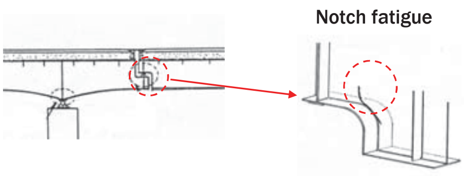
Bridge Beam



Steel slab bridge



Bridge Beam



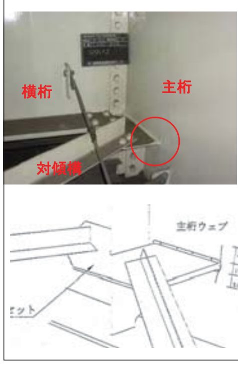
Main cause of the fatigue crack

stress range ＞ fatigue strength

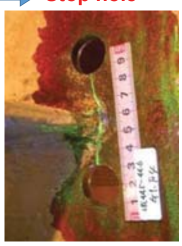
High stress range：lack of knowledge, inappropriate structure

Low fatigue strength：not well welding

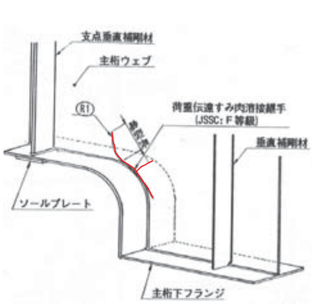
補修事例　①主桁横桁取合部



Fatigue Crack → Stop hole

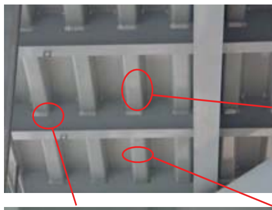


②主桁の桁端切欠き部

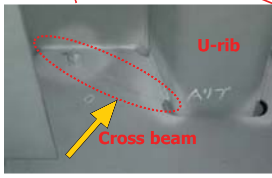


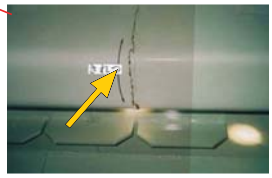
Cause：stress concentration, welding condition

fatigue Steel plate deck



Deck plate and U-rib





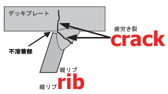
Cross of cross beam and U-rib

Welding of U-rib

fatigue

-fatigue crack-

-repair-

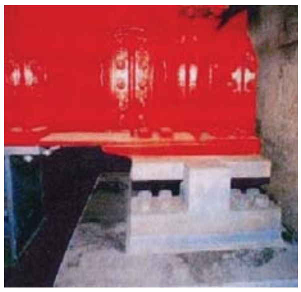


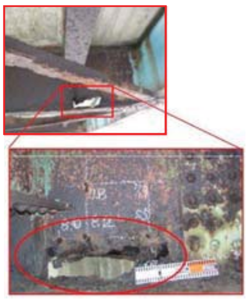
strengthening by（SFRC pavement）

fatigue

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

Replacement of shoe (metal to rubber)





著しく腐食し支承としての機能を失っている

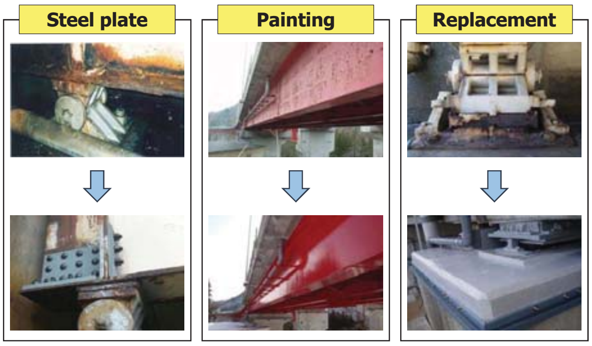
著しく腐食し支承としての機能が損なわれている

橋梁の劣化
（主に国交省HP）

CFRP bonding

Asari river bridge
(Central-Nippon Expressway)

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

Corrosion mainly-induced damages

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.

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

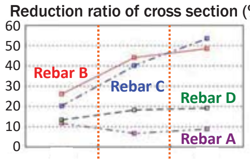
Observed damage (bottom of the slab)

Delamination of concrete

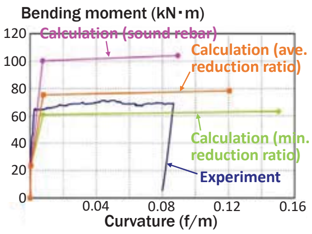
Significant corrosion of main rebars

Kuratanibashi Bridge

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



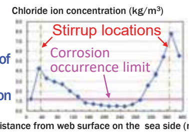
Result of the loading test

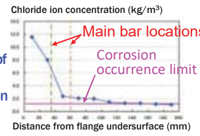


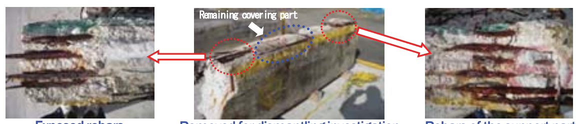
✓ Delamination affects the corrosion.

✓ About 32% reduction of load capacity is expected.

Tsuhabashi Bridge







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

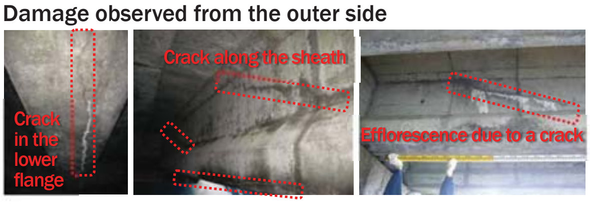
Uenaibashi Bridge

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

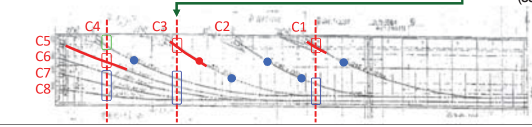
Damage observed from the outer side



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.

Nou-Ohashi Bridge

The girder was removed and damage was investigated.

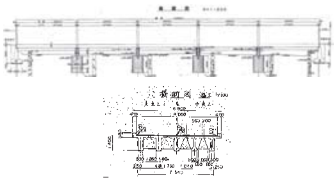
✓ Remarkable damage including rupture of PC cables was observed.

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

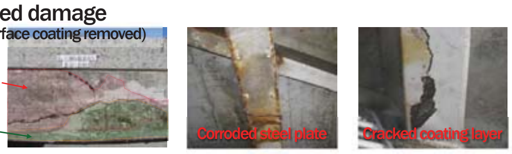
Osahashi Bridge

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)



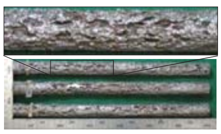
✓ Deteriorated again despite the repair and strengthening
✓ Chloride ion level at the location of steel rebar: 5.7 kg/m^3

Osahashi Bridge

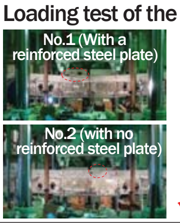
Dismantling survey of the floating part

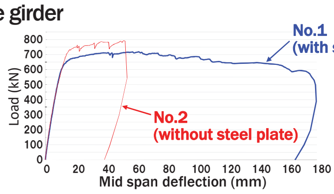
Remarkable pitting corrosion

Mass reduction ratio in the steel rebars



Loading test of the girder





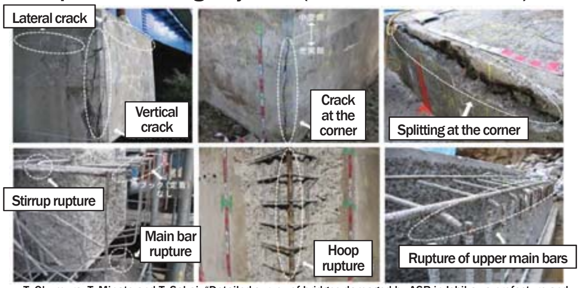
✓ Less effect of the steel plate on load capacity

ASR mainly-induced damages

ASR
（網目状のひび割れ）

Structures Damaged by ASR

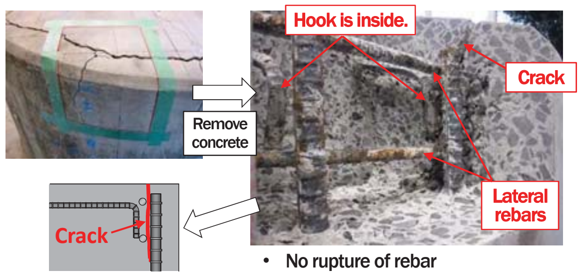
Examples of damage by ASR (Alkali Silica Reaction)



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

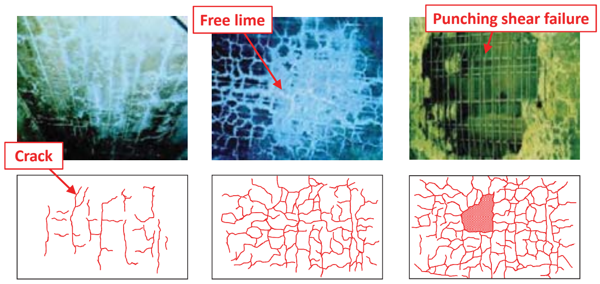
P1 pier

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.

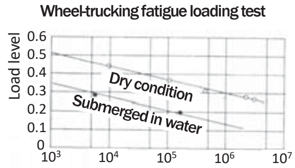
✓ Concrete was disaggregated in the severe environment.

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

Slab surface | Bottom of slab | Corrosion

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

Situations in Other Asian Countries (1)

Bridge type: RC structures with span length around 40 m (approaching part).
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Situations in Other Asian Countries (1)

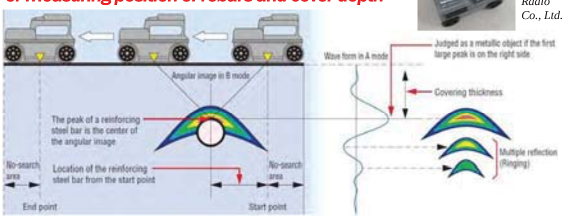
Bridge type: RC structures with span length around 40 m (approaching part).
Location: Rayong Province, Thailand
Age: 7 to 10 years
Environmental condition: Around 500 m away from the sea but sea water comes to the bottom of the structure.

Strange shape observed in the columns

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

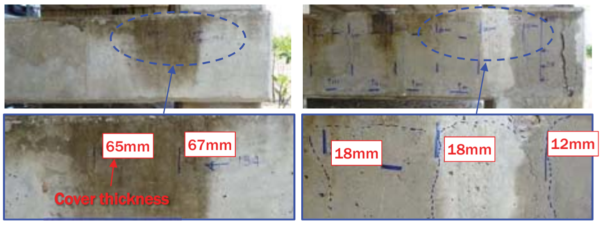
Air Permeability Test

Electric Resistivity Test

For evaluating surface concrete quality

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.

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

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)

Thakhut bridge
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)

Thakhut bridge
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...???

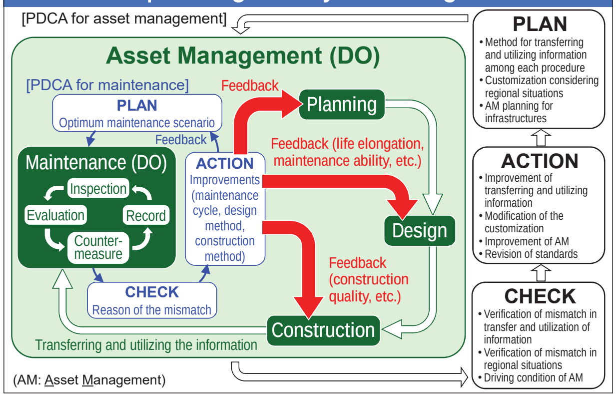
Maintenance of Road Infrastructures

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



Inspection Procedure

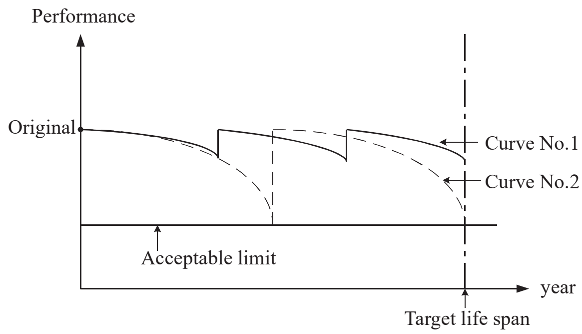
1) Preliminary investigation for efficiency
(check drawings, former records and etc.)
↓
2) Make inspection plan
↓
3) (visual)inspection by access and record
↓
4) Diagnosis (Evaluation of performance)

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

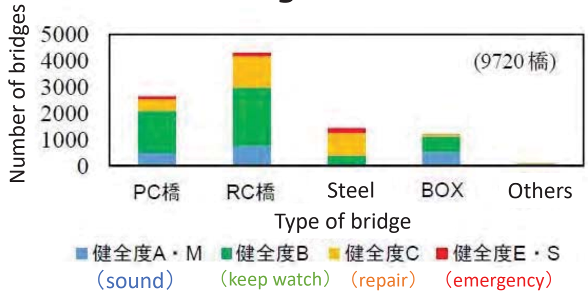
Record (example)

Deterioration curve of performance



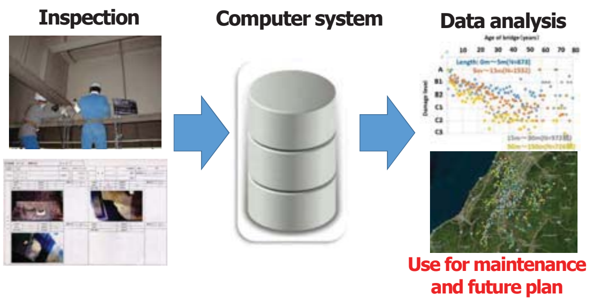
Inspection data

Damage level



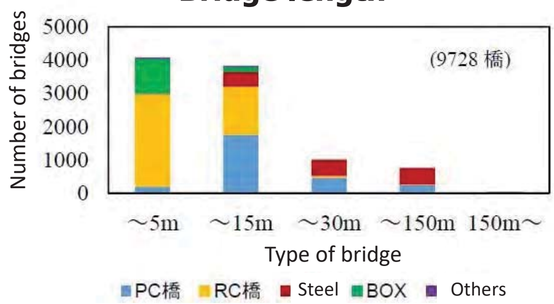
Damage of steel structure is worse than concrete bridges.

Database is important for management



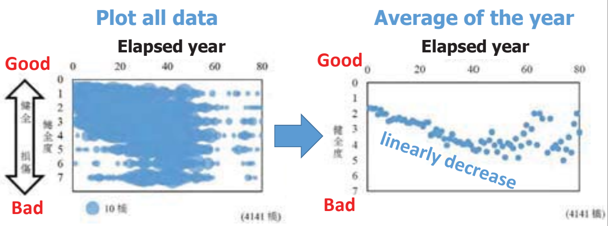
Database is a basis for rational management of infrastructures

Bridge length



Steel bridges are longer (heavy traffic etc. ?)
Difficult to judge, steel or concrete.

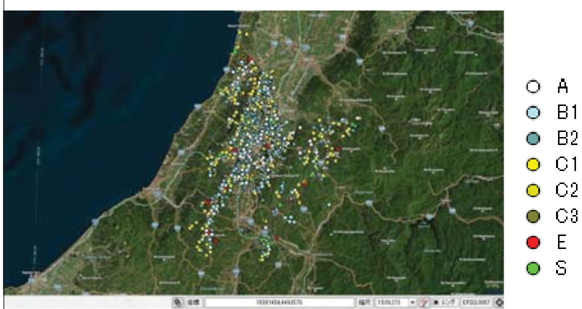
Damage by year



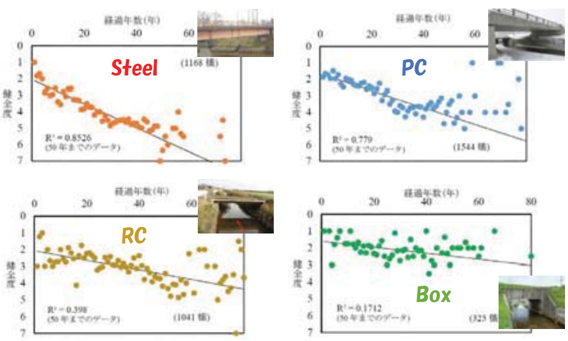
Damage is recovered after 50 years??

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

Damage level

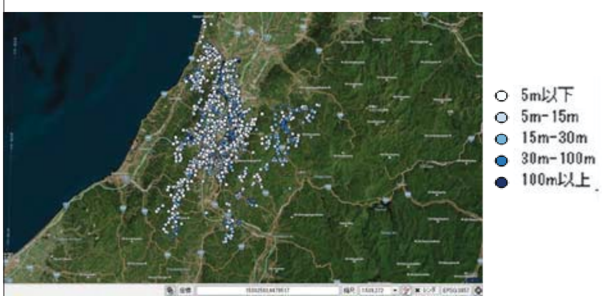


Damage progress

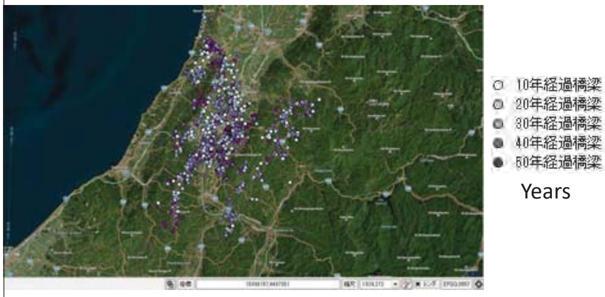


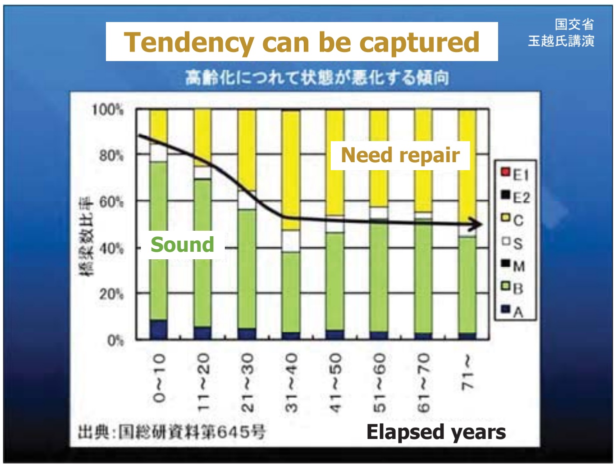
Damage speed: Steel, PC, RC, Box
Steel structure is not maintained well (painting…)?

Length of bridge



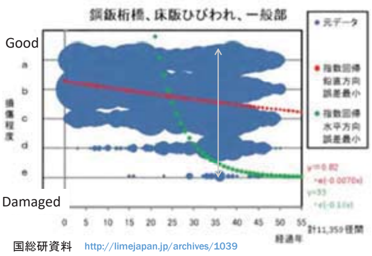
Elapsed year





Prediction of Deterioration

Example of National road damage data



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

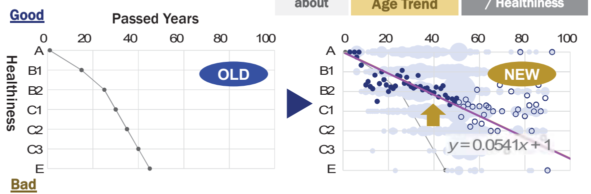
How to estimate

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

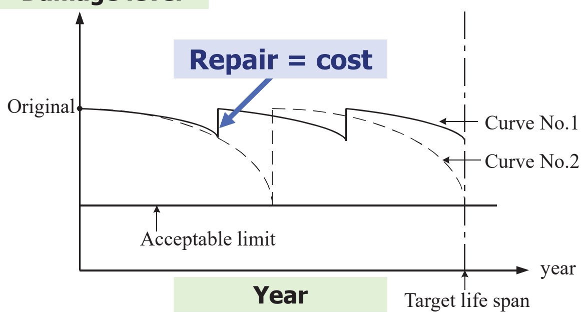
Round-about | Healthiness-Age Trend | Future Budget / Healthiness



N = about 100
Prefecture Samples Only

N = 4,816 Inspection Data
27 Municipal Samples

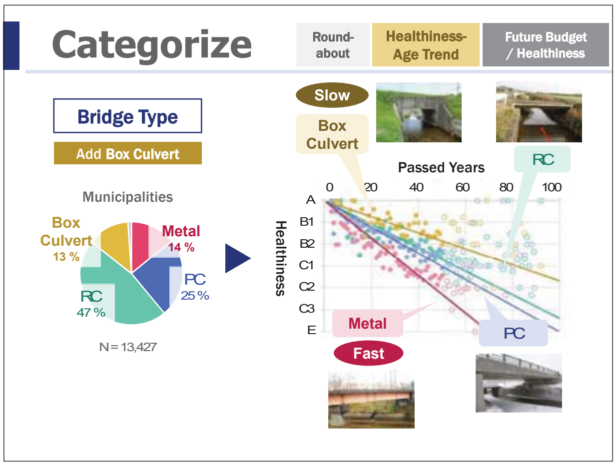
The Deteriorating Speed is Slower than Previously Used

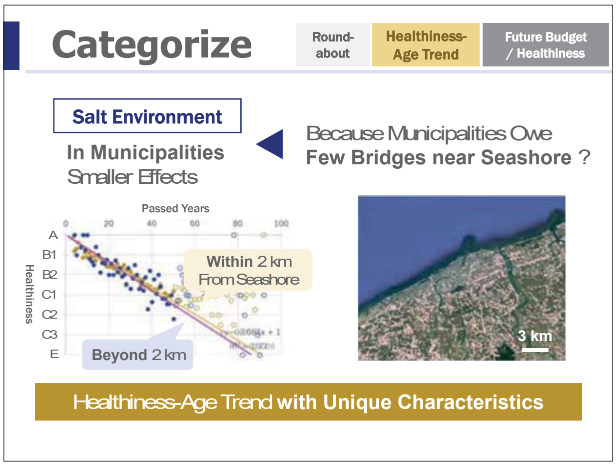


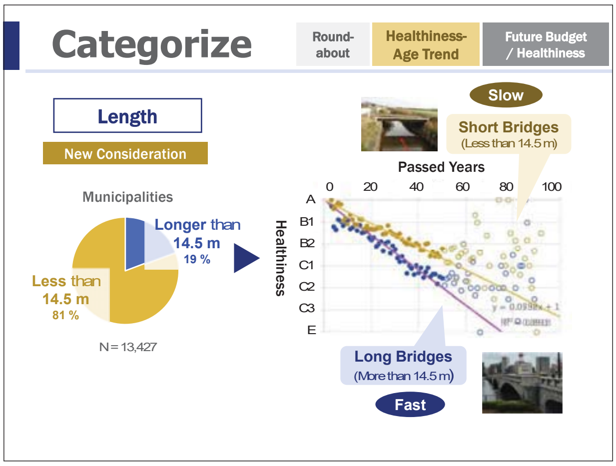
Categorize

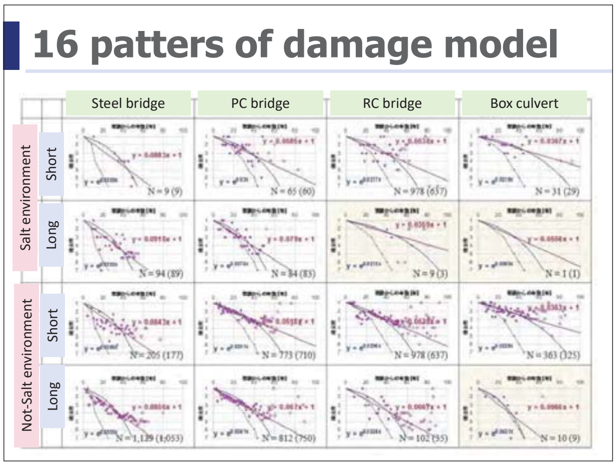
Round-about | Healthiness-Age Trend | Future Budget / Healthiness

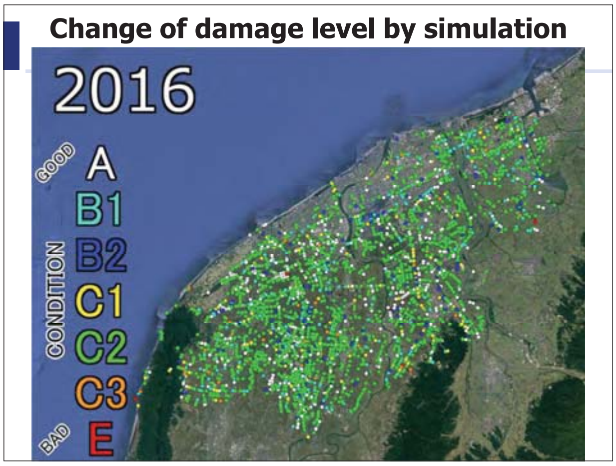
Bridge Type × Length × Salt Environment

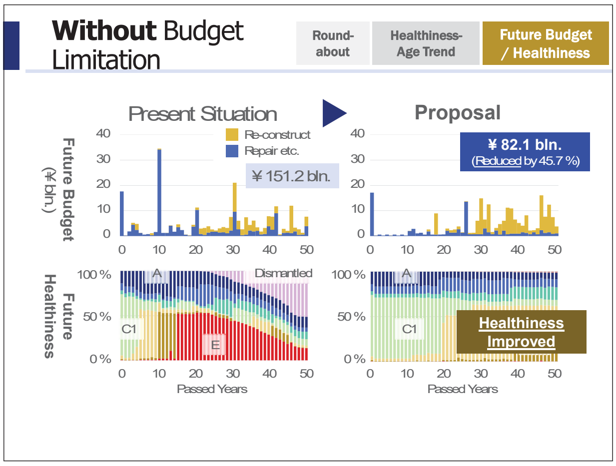

Categorize
Round-about
Healthiness-Age Trend
Future Budget / Healthiness
Bridge Type
Add Box Culvert
Municipalities
Box Culvert 13 %
Metal 14 %
PC 25 %
RC 47 %
N=13,427
Slow
Box Culvert
RC
Passed Years
0 20 40 60 80 100
Healthiness
A
B1
B2
C1
C2
C3
E
Metal
Fast
PC

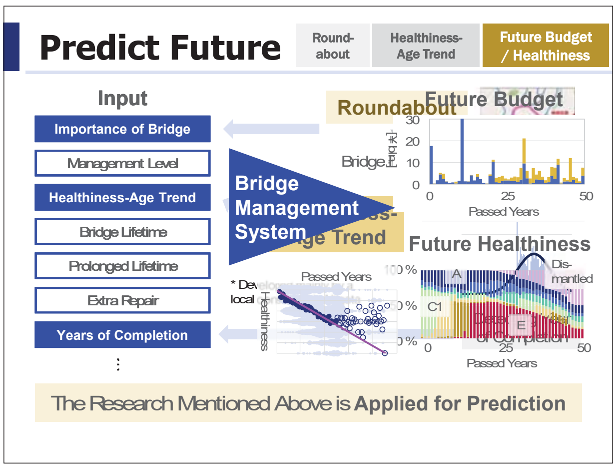

Categorize
Round-about
Healthiness-Age Trend
Future Budget / Healthiness
Salt Environment
In Municipalities Smaller Effects
Because Municipalities Owe Few Bridges near Seashore ?
Passed Years
Healthiness
Within 2 km From Seashore
Beyond 2 km
3 km
Healthiness-Age Trend with Unique Characteristics

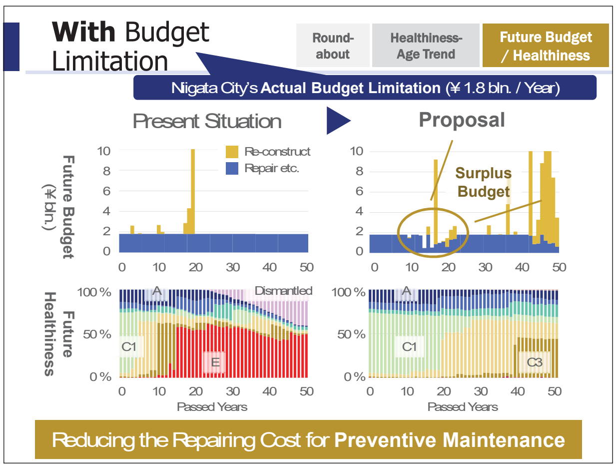

Categorize
Round-about
Healthiness-Age Trend
Future Budget / Healthiness
Length
New Consideration
Municipalities
Longer than 14.5 m 19 %
Less than 14.5 m 81 %
N=13,427
Slow
Short Bridges (Less than 14.5 m)
Passed Years
0 20 40 60 80 100
Healthiness
A
B1
B2
C1
C2
C3
E
Long Bridges (More than 14.5 m)
Fast


16 patters of damage model
Steel bridge
PC bridge
RC bridge
Box culvert
Salt environment
Not-Salt environment
Short
Long


Change of damage level by simulation
2016
GOOD
CONDITION
BAD
A
B1
B2
C1
C2
C3
E

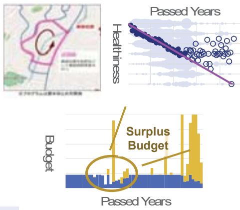

Without Budget Limitation
Round-about
Healthiness-Age Trend
Future Budget / Healthiness
Present Situation
Proposal
Future Budget (¥ bln.)
Re-construct
Repair etc.
¥ 151.2 bln.
¥ 82.1 bln. (Reduced by 45.7 %)
Future Healthiness
Dismantled
Healthiness Improved
Passed Years


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
Roundabout
Future Budget
Bridge Management System
Future Healthiness
Passed Years
The Research Mentioned Above is Applied for Prediction


With Budget Limitation
Round-about
Healthiness-Age Trend
Future Budget / Healthiness
Niigata City's Actual Budget Limitation (¥ 1.8 bln. / Year)
Present Situation
Proposal
Future Budget (¥ bln.)
Re-construct
Repair etc.
Surplus Budget
Future Healthiness
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 Nigata ▶ Implemented Successfully

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

Application to Nigata-City

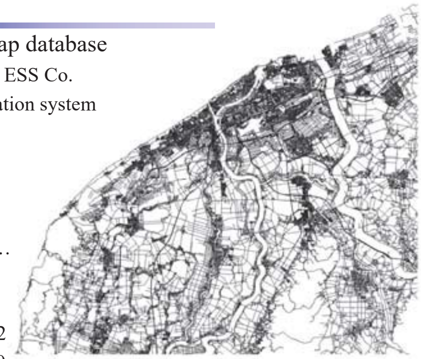
- Basic information of Nigata-city
  - One of prefectural capital
  - Population: 807,450
  - Area: 726.25km^2
  - Shinano-river, which is the longest in Japan
- About the bridges
  - Over 4000!
  - Nagai-sensei has very informative data about their condition :-)

Example

Bridge data and Road network

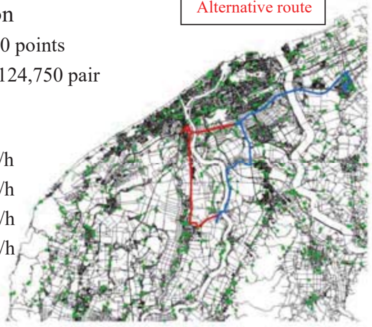
Road Network Data

- 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



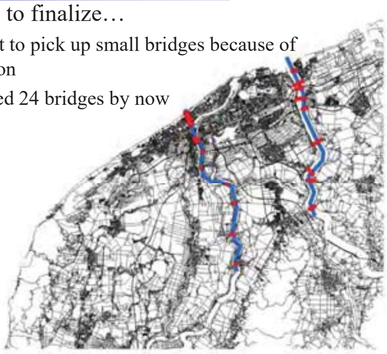
How to Calculate

- 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

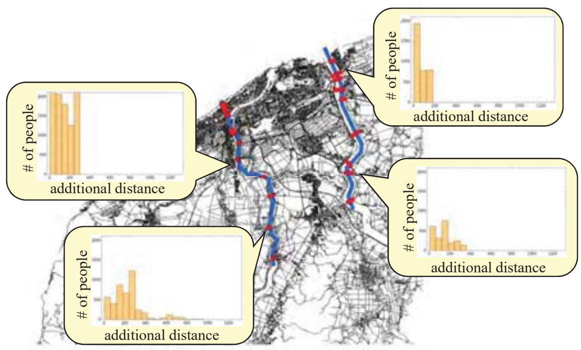


Location of River and Bridge

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

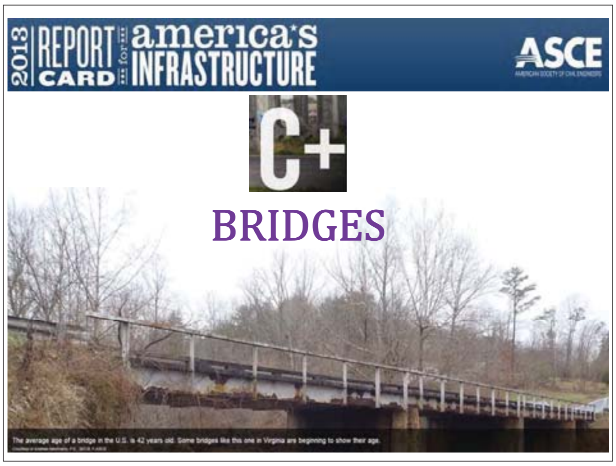


Distribution of Additional Distance

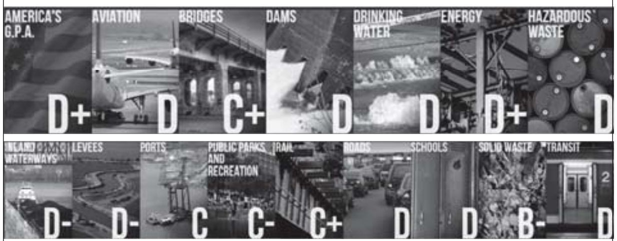


Example

Open Data and Open Public

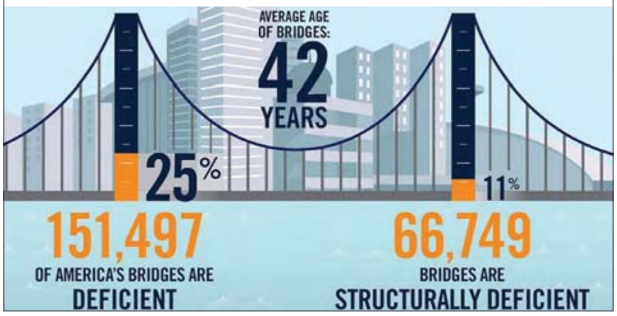


INFRASTRUCTURE REPORT 2013

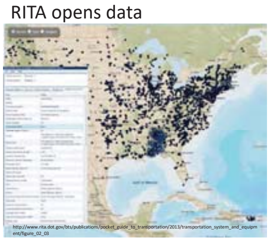


Overview: Click

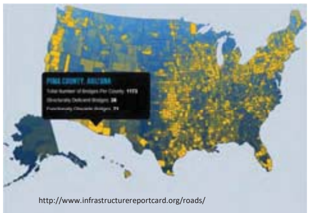
- One in nine of the nation's bridges are rated as structurally deficient, while the average age of the nation's 607,380 bridges is currently 42 years
- estimates that to eliminate the nation's bridge deficient backlog by 2028
- need to invest $20.5 billion annually, while only $12.8 billion is being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



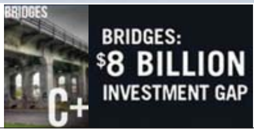
RITA opens data



ASCE converts the data for public



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 SIP 戦略的イノベーション創造プログラム Cross-ministerial Strategic Innovation Promotion Program

(supported by Cabinet office of Japan)

SATREPS Project JICA JST

(supported by Cabinet office of JICA)

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

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 戦略的イノベーション創造プログラム Cross-ministerial Strategic Innovation Promotion Program

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

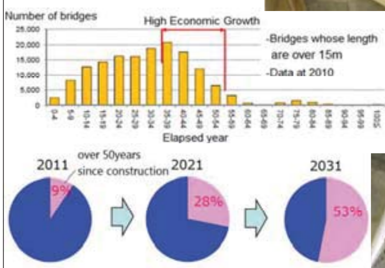
Head of the project:
Prof. Koichi Maekawa (University of Tokyo)

東京大学 THE UNIVERSITY OF TOKYO

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

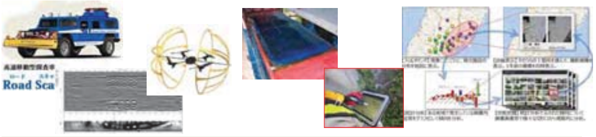
---

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

北海道大学 ICUS

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 Tangtermsirikul
New Partner: Chulalongkorn U

Implement Japanese technology and system for management of infrastructure assets

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

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)

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)

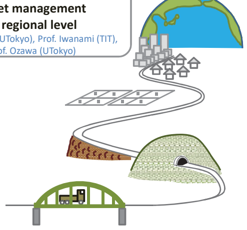
Sub group 3
Asset management in regional level
Prof. Horita (UTokyo), Prof. Iwanami (TIT), Prof. Ozawa (UTokyo)

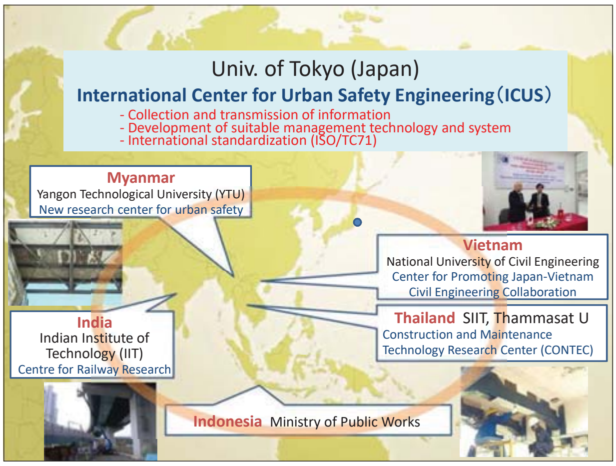
Sub group 2
Road asset management
Prof. Nasu (Kochi TU)
Prof. Kobayashi (Kyoto U)

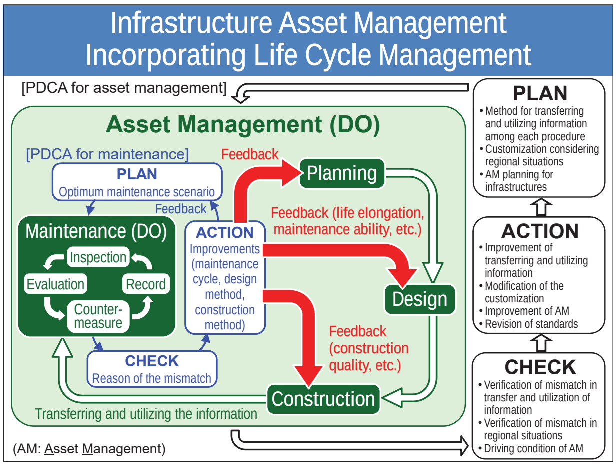
Sub group 1
Life-span estimation of RC Slab
Prof. Iwaki (Nihon U)

Management of road asset

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



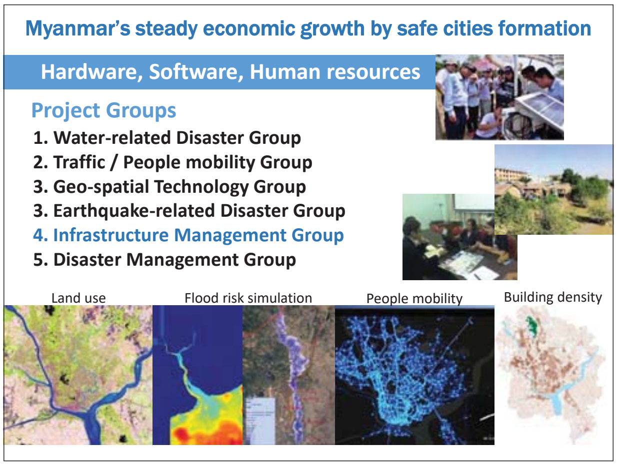



Infrastructure Asset Management
Incorporating Life Cycle Management
[PDCA for asset management]
Asset Management (DO)
[PDCA for maintenance]
Feedback
PLAN
Optimum maintenance scenario
Feedback
Planning
Maintenance (DO)
Inspection
Evaluation
Record
Counter-measure
ACTION
Improvements (maintenance cycle, design method, construction method)
Feedback (life elongation, maintenance ability, etc.)
Design
Feedback (construction quality, etc.)
CHECK
Reason of the mismatch
Construction
Transferring and utilizing the information
(AM: Asset 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

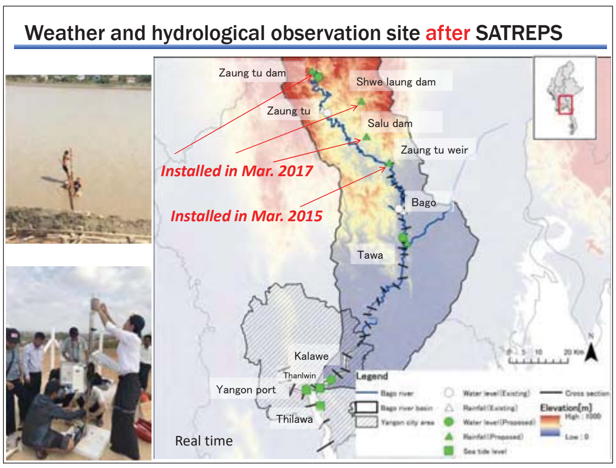

SATREPS: JICA-JST Fund Research Project (FY2015-2019)
Development of a Comprehensive Disaster Resilience and Collaboration Platform in Myanmar
Not technical cooperation, but research based project
Principal institutions: University of Tokyo, JAPAN
Yangon Technological University, MYANMAR
JICA
JST
ANZEN SATREPS : MYANMAR NIPPON ZERO-CASUALTY ENGINEERING NETWORK 2014-2020

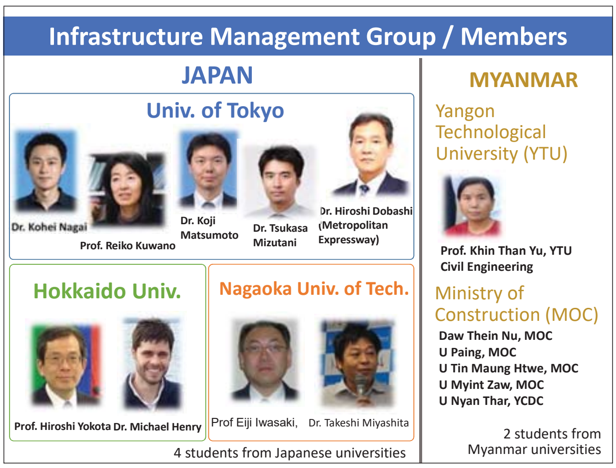

Target area of the Project
Inspection
(Existing & New)
Corrosion
Permeability
Maintenance plan
Data base
Data analysis
GIS
Repair
FRP
Fiber reinforced concrete
Relating leading research
Simulation
Experiment
Material development


Project Office in YTU
Research Centre for Urban Safety
Meeting rooms
Equipment for measurement etc.


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
Land use
Flood risk simulation
People mobility
Building density


Way to the site (Last week)
1 hour to recover
Road infrastructure is important !


Weather and hydrological observation site after SATREPS
Zaung tu dam
Shwe laung dam
Zaung tu
Salu dam
Zaung tu weir
Installed in Mar. 2017
Installed in Mar. 2015
Bago
Tawa
Kalawe
Thanlwin
Yangon port
Thilawa
Real time
Legend


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)

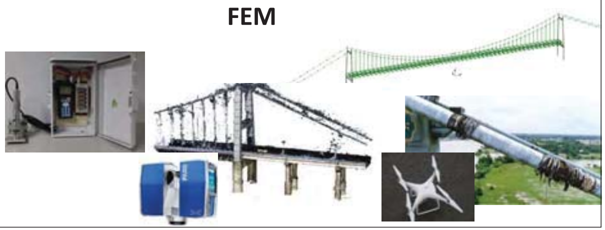
Measurement of road surface roughness
(Pavement management)

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

Assessment of Damaged Suspension Bridge in Myanmar

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

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



Proposal 1

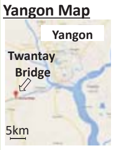
When damage is found,
Install an appropriate
simple monitoring system.

Case studies
Twantay bridge
Thkatut bridge

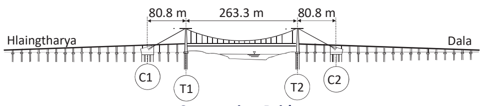
In the future,
For new important structures, we suggest to install the simple monitoring system from the beginning to capture the initial deformation.

Twantay Bridge (constructed 2006)

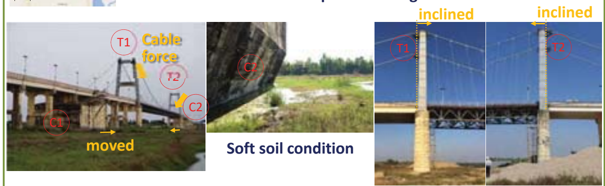
Yangon Map



General view:



Suspension Bridge



Soft soil condition

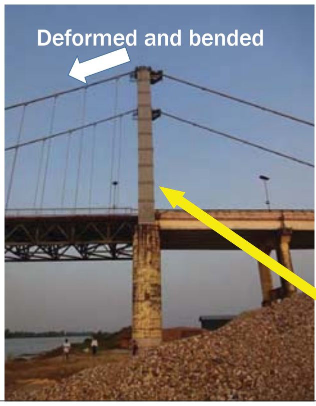
Both anchorage blocks move

Both main towers are inclined

Main tower T2 is the target for the monitoring

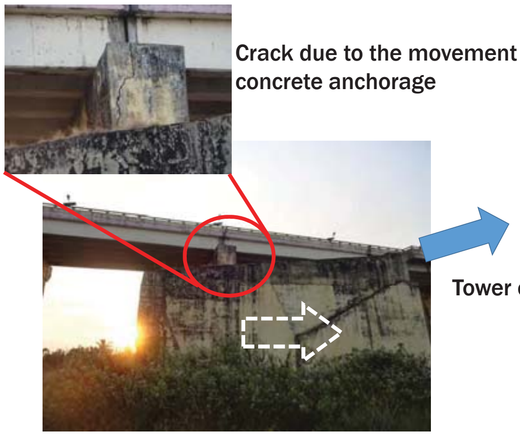
Both main towers are inclined in bridge axis direction

Tower is inclined



Concrete anchorage

Inclinometer installed

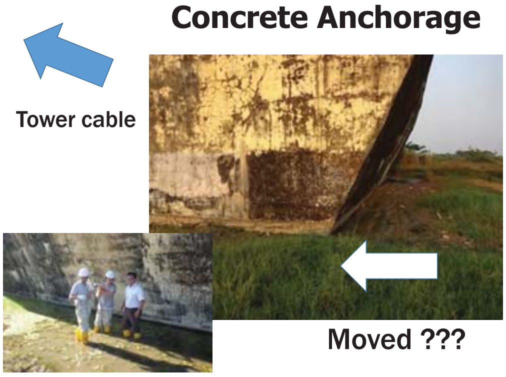


Crack due to the movement of concrete anchorage

Tower cable

Concrete Anchorage

Tower cable



Moved ???

Simple soil test

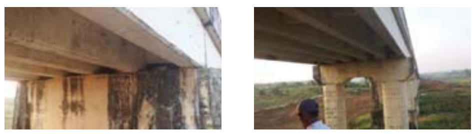
Recent cracked anchorage survey

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

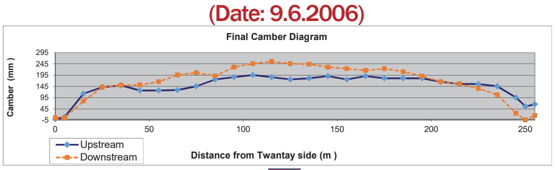


No crack in north side

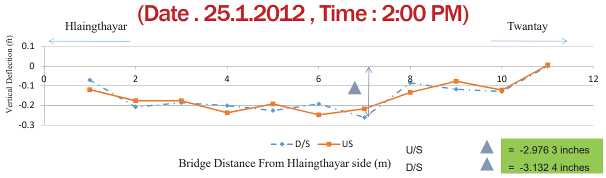
Measurement of road level by MOC

TWANTY BRIDGE MAIN BRIDGE STEEL TRUSS LEVEL CHECKING HLAINGTHAYAR TO DALA

(Date: 9.6.2006)



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



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)

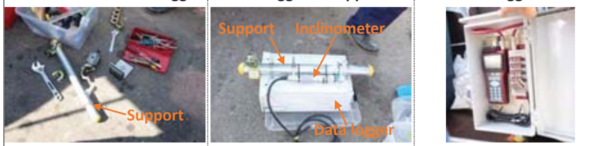
Monitor the inclination in bridge axis direction of the main tower

Question

Is this bridge safe ?

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

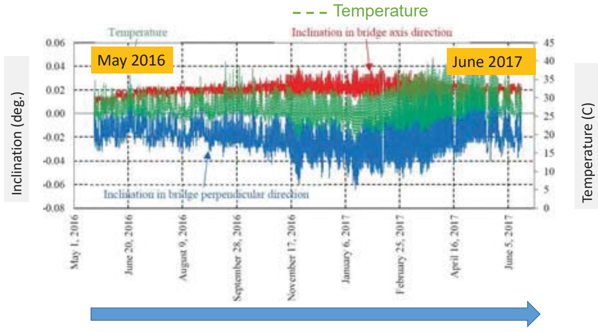
Set an inclinometer and a data logger at the height of 10 m above read level

Twantay Bridge

Monitoring system was set successfully at South Main Tower

Tower inclination

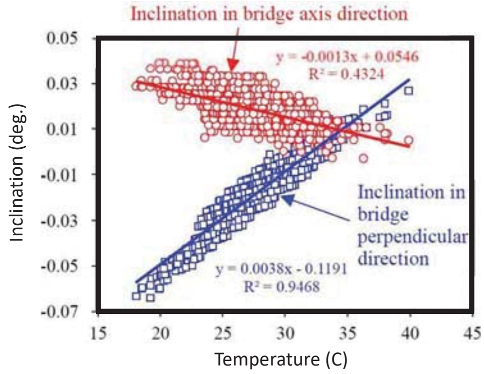
Bridge axial direction
Bridge perpendicular direction
Temperature



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

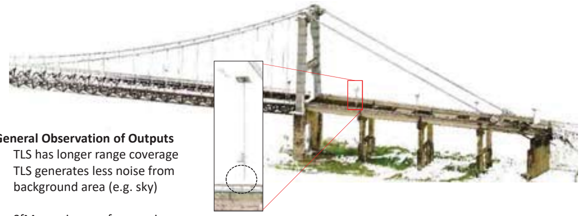
Correction of data

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

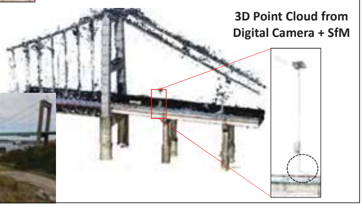


General Observation of Outputs

- 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



3D Point Cloud from Terrestrial Laser Scanner

Is this bridge still safe??

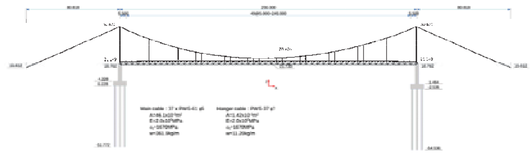
Analysis of Twantay Bridge

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

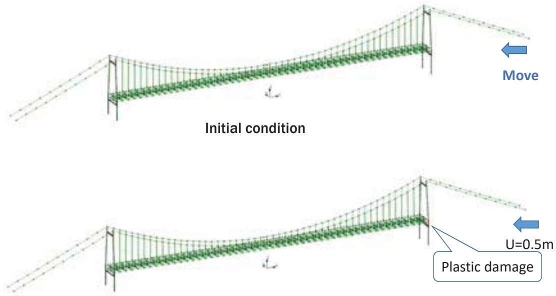
1.
Set the initial condition considering the tension force of cable and hangers.

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



Overall deformation

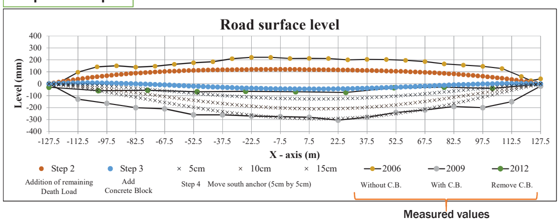
長岡技術科学大学 Nagaoka University of Technology



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

Road Surface Level

Step 3 & Step 4



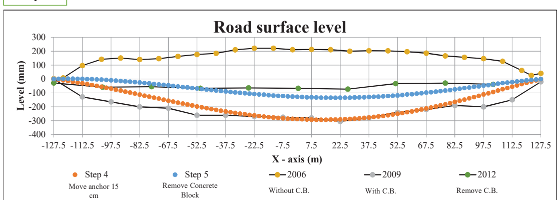
Measured values

- 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

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

Tower Shape

Tower displacement results

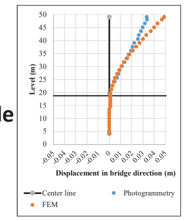
FEM results are from Step 5 (Twantay Bridge current state)

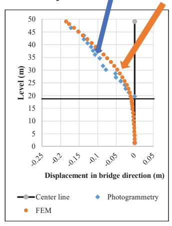
Photogrammetry results were obtained after FEM analysis

Measurement at site

FEM simulation

Tower North side





Tower South side

Error: 1.7cm	3D: top of tower disp = 3.3cm FEM: top of tower disp = 5.0 cm	3D: top of tower disp = 17.2cm FEM: top of tower disp = 18.5cm	Error: 1.3cm

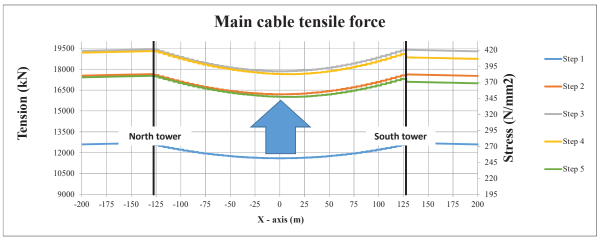
Results from photogrammetry well matched FEM analysis

Cable Stress Level

Main cable

Yield point $\sigma_y = 1180$ N/mm2

Tensile strength $\sigma_t = 1570$ N/mm2



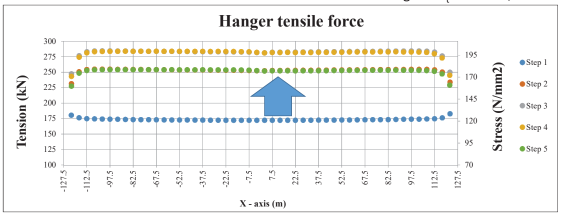
Stress level: 270 MPa -> 370 MPa (yielding stress: 1,180 MPa)

Cables are still in a safe level

Hanger Stress Level

Hanger cables

Yield point σ_y = 1180 N/mm2
Tensile strength σ_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:

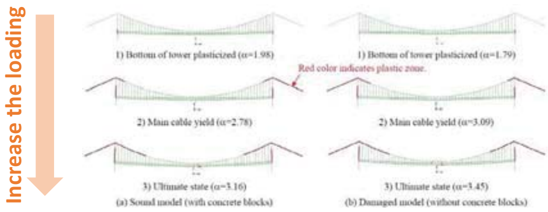
- Anchorage block moved 13.5cm (from expansion device)
- Camber at center of main span was reduced by 55cm
- Towers inclined in bridge direction

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

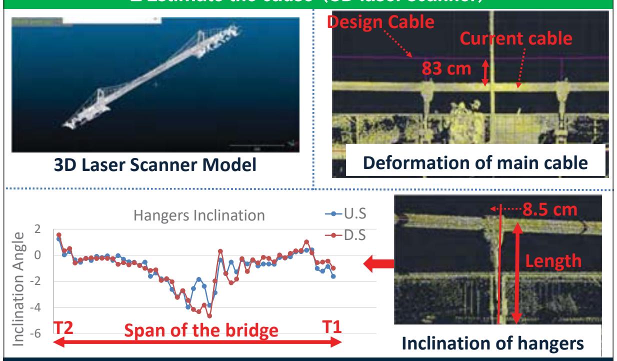
Pathein Bridge

Pathein bridge was constructed in 2004 by MOC of Myanmar

Pathein bridge is located in southern part of Myanmar

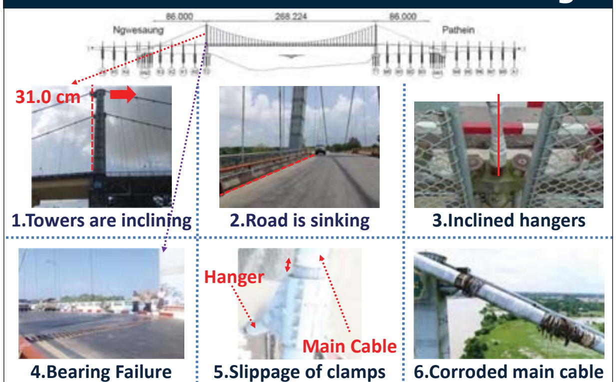
3D laser scanner

1-Estimate the cause（3D laser scanner）



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

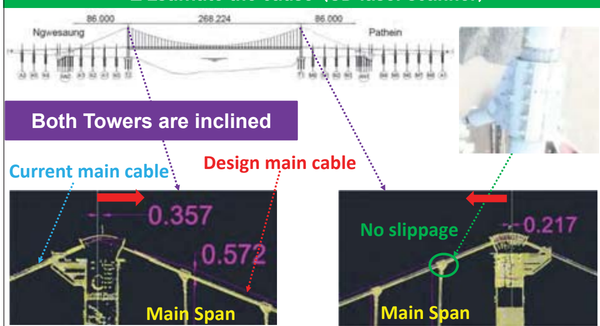
Problem Statement of Pathein Bridge



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

3D laser scanner

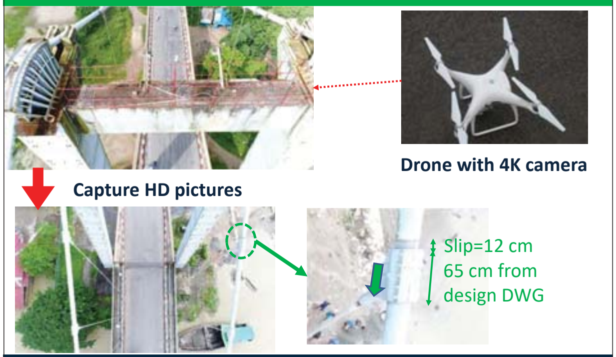
1-Estimate the cause（3D laser scanner）



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

Drone

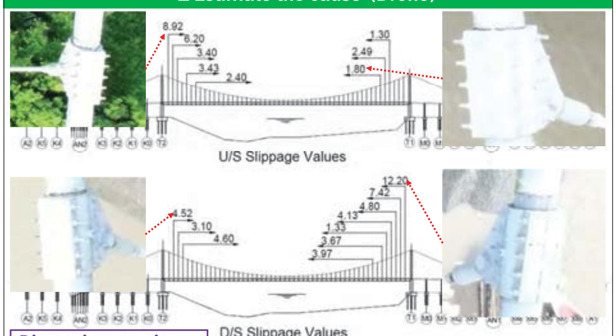
1-Estimate the cause（Drone）



Drone was used to measure the slippage at all clamps

Drone

1-Estimate the cause（Drone）

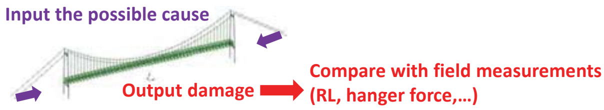


Dimensions are in cm

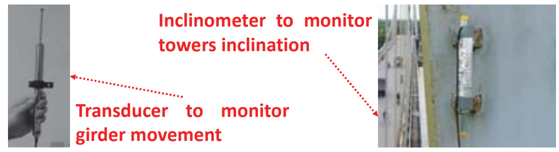
Drone could be used to measure slippage of all clamps

Future Works

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

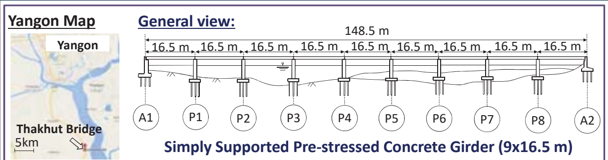


2- Collect data from the monitoring system.



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

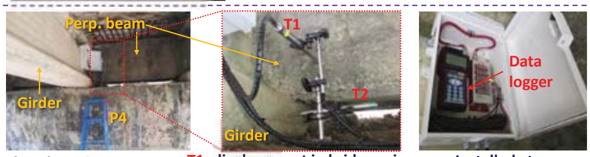
Pier P4 is the target for the monitoring

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

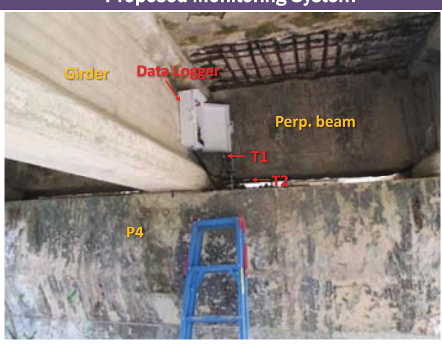
T1: displacement in bridge axis
T2: displacement perp. bridge axis

Installed at concrete girder

To monitor the relative displacement between beams and pier

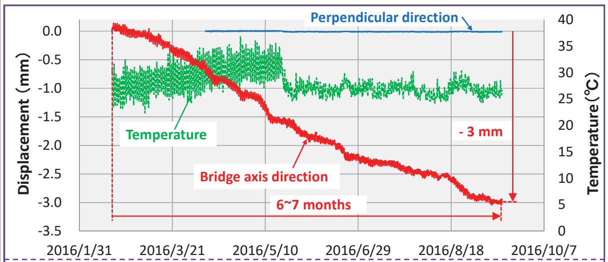
Thakhut Bridge

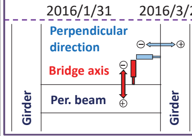
Proposed Monitoring System



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

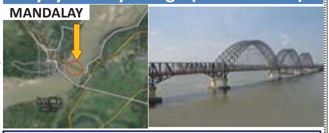
Maintenance strategy should be established for damaged bridges (long term planning)

Case study

Ayeyarwady Bridge (Yadanarbon)

Ayeyarwady Bridge (Yadanarbon) constructed 2008

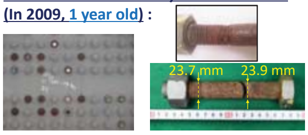
Ayeyarwady Bridge (Yadanarbon)



A steel truss bridge with high strength bolts

Public Works of Ministry of Construction

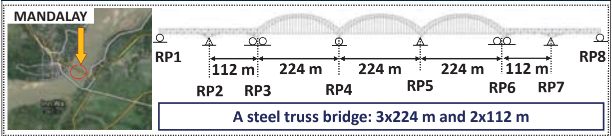
(In 2009, 1 year old) :



Fracture of high strength bolt was found

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

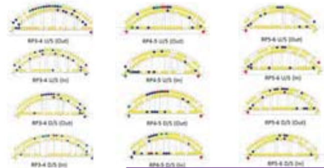
Ayeyarwady Bridge (Yadanarbon)



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

Location of broken bolts: (2009-2015)

RP3-RP4 RP4-RP5: RP5-RP6:

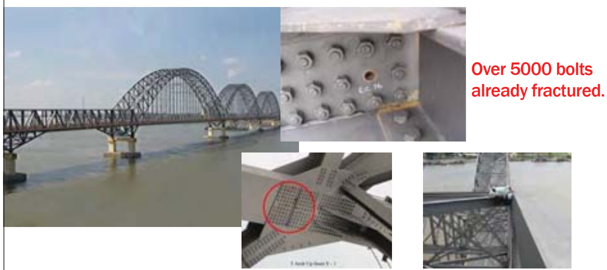


Location	Number of Broken Bolts and Nuts
RP1-RP2 RP7-RP8	257
RP3-RP4	2021
RP4-RP5	1912
RP5-RP6	1378
Total	5568

In all spans, fracture of high strength bolts occurs

Frequent Damages of High Strength Bolts

Bolt Fracture in Yadanarbon Bridge

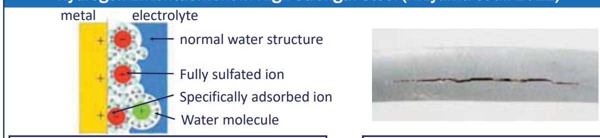


Over 5000 bolts already fractured.

- 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

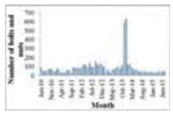
Hydrogen Embrittlement in High Strength Steel (Akiyama et al. 2012)



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 RP7-RP8	257
RP3-RP4	2021
RP4-RP5	1912
RP5-RP6	1378
Total	5568

5568 bolts and nuts were broken from 2009-2015

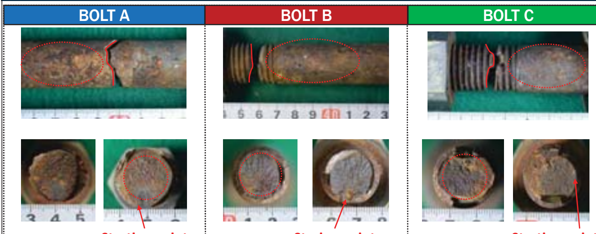
The cause of the early fracture of the bolts is unclear

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)

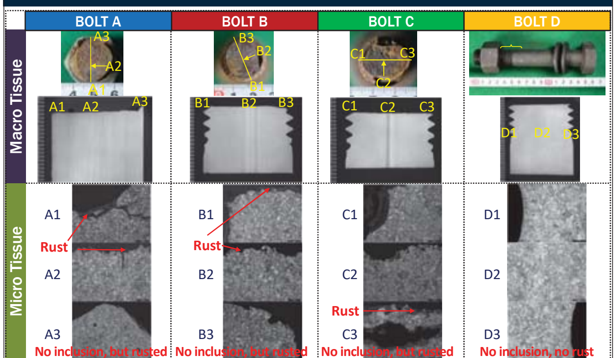
1. Analysis of material (finished)

   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.

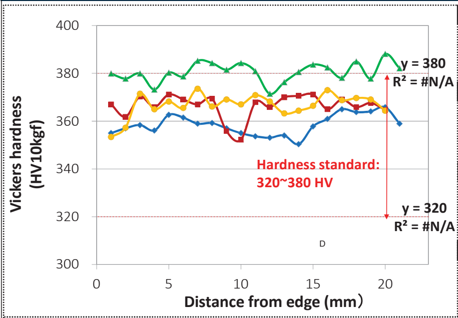
Sectional Tissue

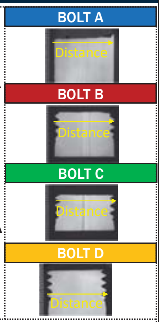


The tissue has a uniform tempered martensite structure with no defects

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

Hardness Testing



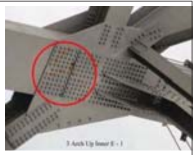


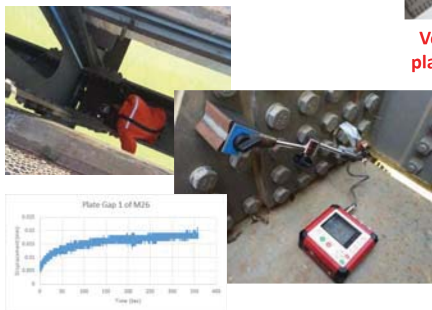
Hardness values: Bolt A = 350~366 HV BOLT C = 371~388 HV
Bolt B = 352~371 HV BOLT D = 353~374 HV
Standard value : 320~380 HV

Hardness of all samples are still within acceptable value

Step 2: Plate gap measurement

Now done by Ms. Yi Yi Mon of MTU

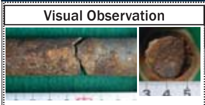
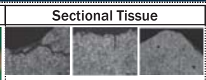
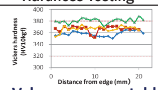




Very complicated plate arrangement.

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.

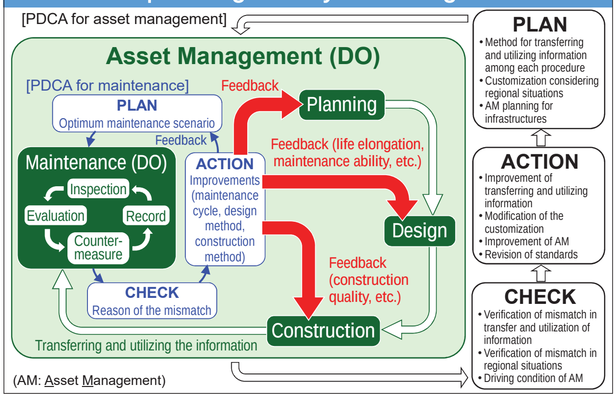
These possible causes are still being investigated.

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



Now.....

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

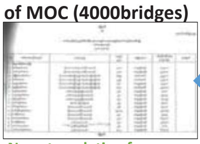
YTU student

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

Bridge Database with GIS

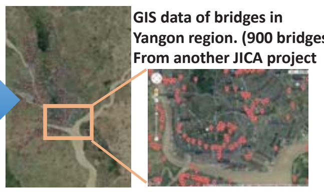
Development of infrastructure database in Myanmar

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

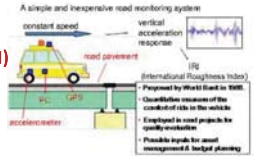
Road surface roughness condition in Yangon is surveyed by simple measurement system.

Survey area
Yangon city area

Road roughness in Yangon

Dynamic Response Intelligent Monitoring System (DRIMS)

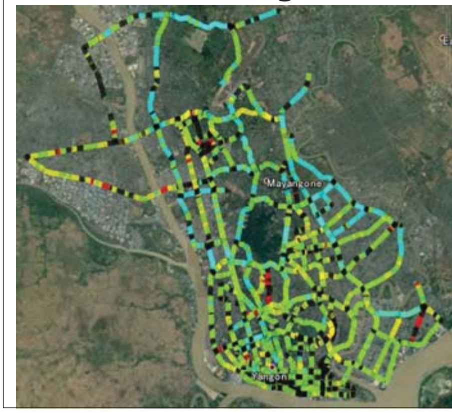
Convert the data into International Roughness index (IRI)



Very easy and simple !!

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

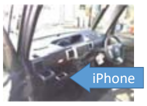
Trial run in Yangon



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

[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

[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)

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)

Safety check

$\Sigma f_i \leqq h \cdot f_a = h \cdot [\{f_{cr}(\leqq f_y)\}/v]$

f : σ, τ
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}
γ_i, Φ : partial factor

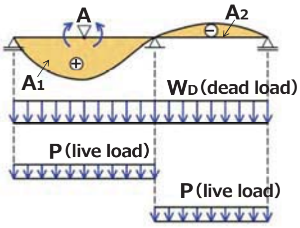
At the design,
We need to know the stress resultants (N, M, Q) and stress (σ, τ) of structures under loading to check the safety.

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

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

$M_D = W_D (A_1 + A_2)$

$M_{L,max.} = PA_1$

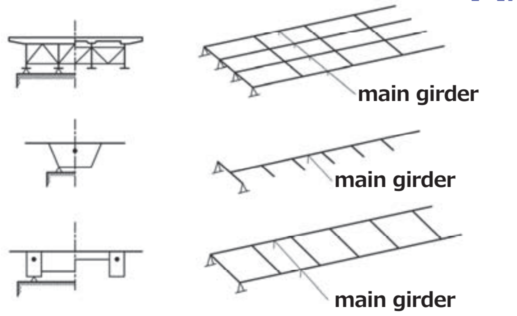
$M_{L,min.} = PA_2$

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

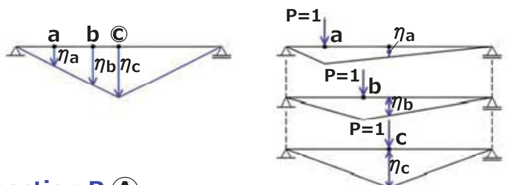
$M_{min.} = M_D - M_{L,min.}$

Fish-bone and Grid models

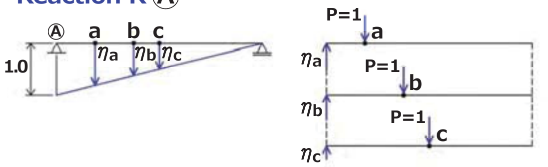
(for obtaining stress resultants [N, M, Q] and deflection[δ])



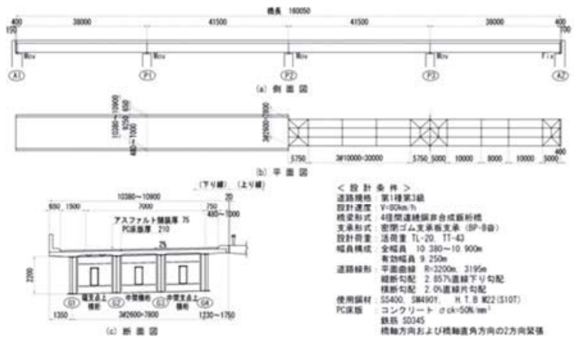
Bending moment （at Ⓒ）



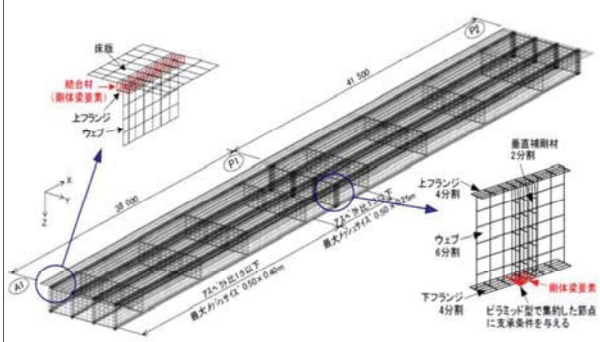
Reaction R Ⓐ



Steel multi-I-girder bridge

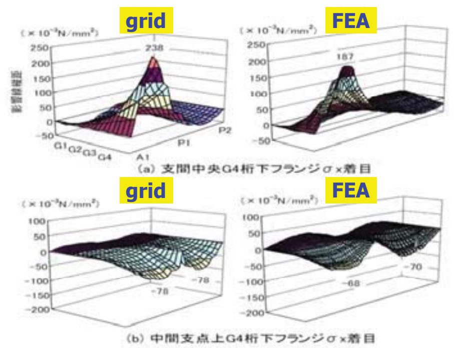


Girder bridge modelled by Shell Element



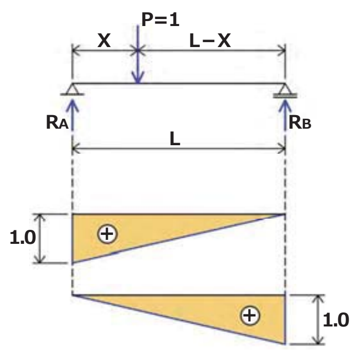
Simple beam

Ex. of influence line analysis



3D influence volume (FEA vs GRID)

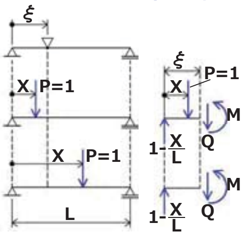
Reactions R_A, R_B



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

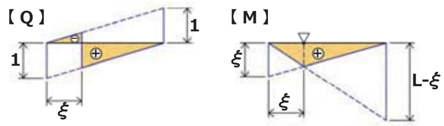
$R_B = \frac{X}{L}$

Moment and Shear at ξ

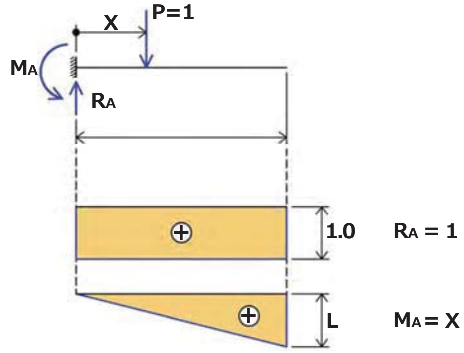


$Q=1-\frac{X}{L}-1=-\frac{X}{L}$, $M=\left(1-\frac{X}{L}\right)\xi-(\xi-X)$
$=X\left(1-\frac{\xi}{L}\right)$

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

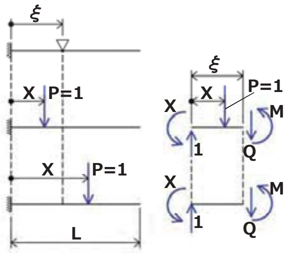


Reactions M_A, R_A



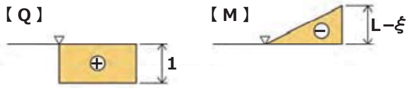
Cantilevered beam

Moment and shear at ξ



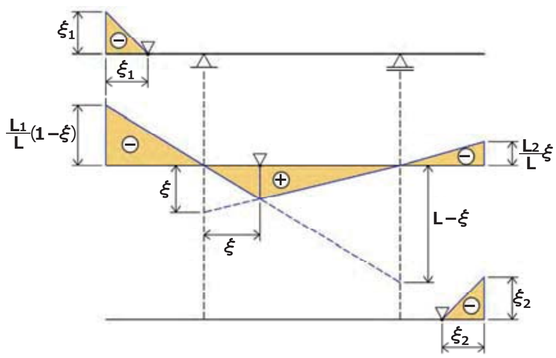
$Q=1-1=0$, $M=\xi-(\xi-X)-X$
$=0$

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

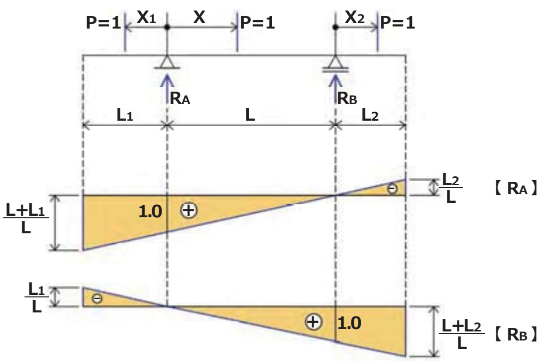


Simple beam with wings

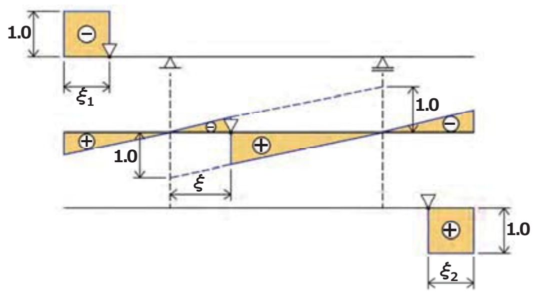
Moment at ξ_1, ξ, ξ_2



Reactions R_A, R_B

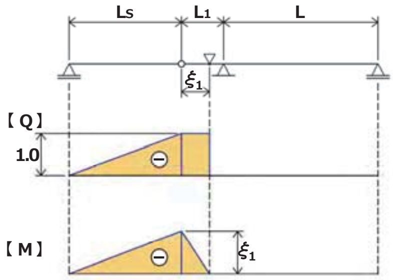


Shear at ξ_1, ξ and ξ_2

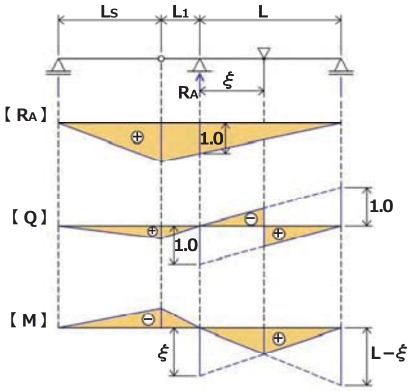


Gerber beam

Moment and shear at ξ_1

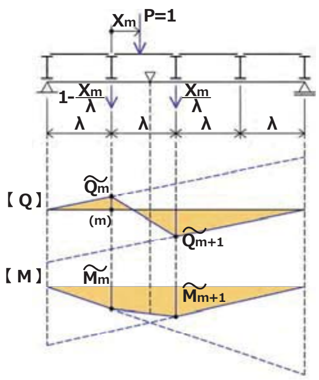


R_A, moment and shear at ξ



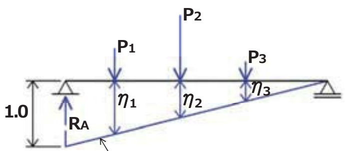
Simple beam
(Indirect loading)

Moment and shear at point (▽)

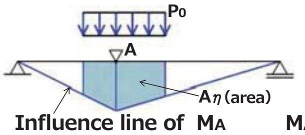


$\tilde{Q}_m$, $\tilde{M}_m$ are shear force and moment at (▽) position when unit load is applied to point (m).

Reaction and moment using influence line



Influence line of R_A $R_A = \sum_{i}^{3} P_i \eta_i$

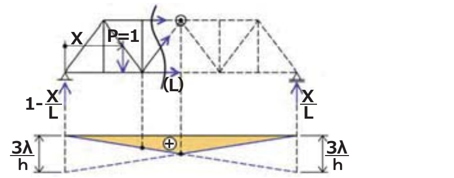


Influence line of M_A $M_A = P_0 A_\eta$

[11-2-2]

Influence line of Truss

Lower chord member (L)



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

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

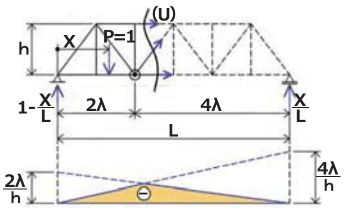
$$L = \frac{1}{h}\left(1 - \frac{3\lambda}{L}\right)X = \frac{1}{h}\frac{3\lambda}{L}X$$

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

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

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

Upper chord member (U)



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

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

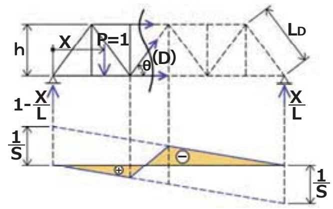
$$U = -\frac{1}{h}\left(1 - \frac{2\lambda}{L}\right)X = -\frac{1}{h}\frac{4\lambda}{L}X$$

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

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

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

Diagonal member (D)



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

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

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

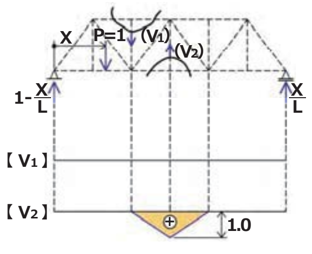
$$D \cdot S + \left(1 - \frac{X}{L}\right) = 1 \implies D = \frac{1}{S}\frac{X}{L}$$

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

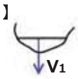
$$D \cdot S + \left(1 - \frac{X}{L}\right) = 0 \implies D = -\frac{1}{S}\left(1 - \frac{X}{L}\right)$$

Vertical members

Vertical member (V_1, V_2)



[V_1]



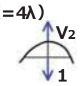
$V_1 = 0$

[V_2]

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

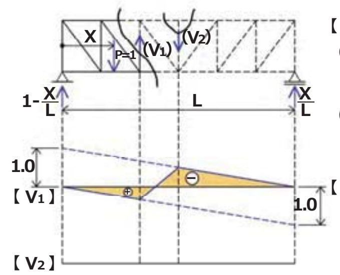
$V_2 = 0$

($X = 4\lambda$)



$V_2 = 1$

[lower deck type]



[V_1]

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

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

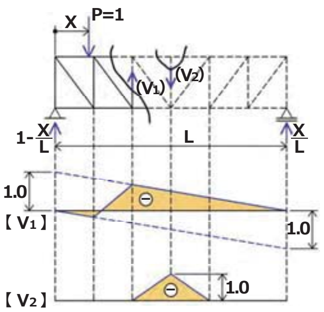
($3\lambda \le X \le L\ (=6\lambda)$)

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

[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 $X = 3\lambda$)

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

[11-2-3]

Bridge Design (Japan, AASHTO LRFD, EC)

Process of the Planning

Necessity of the Project (Feasibility Study)

↓

Road Design

↓

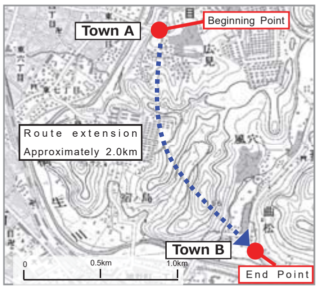
Basic Design of Structure (Selection of Bridge Type)

← Natural Condition Surveys

Road-Building Plan

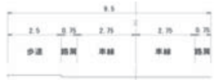
From the study of a new road location to the selection of Bridge Type

Road Design



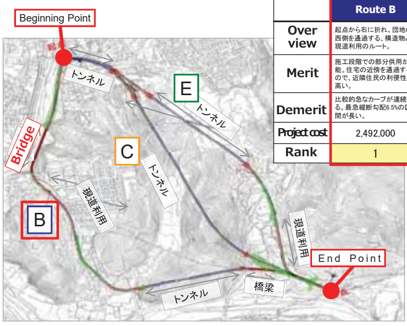
Connect from the beginning point to the end point. Design a route of 2.0km

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

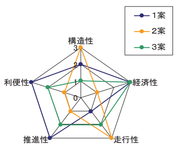


Component of Road Cross Section

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

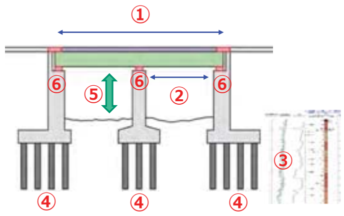


⇒ Select the Route B that excels in terms of economical, progress of the project and neighborhood convenience.

Basic Design of Bridge

Study Items

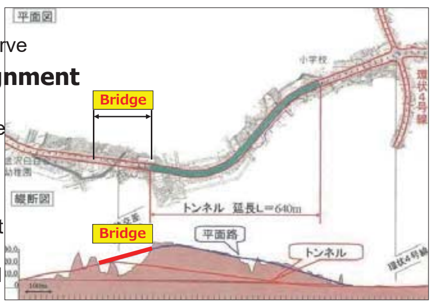
① Bridge Length
② Span Length
③ Bearing Layer
④ Embedment of Foundation
⑤ Clearance under Girder
⑥ Support Condition



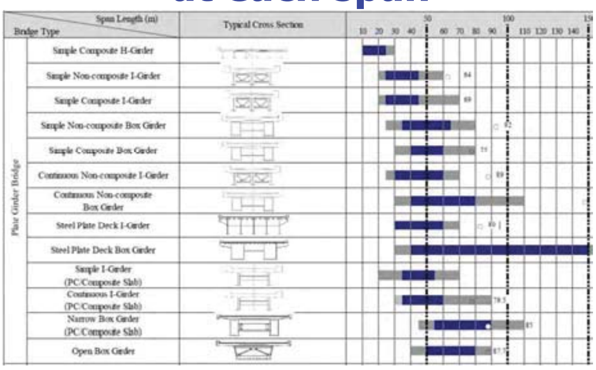
Road Plan –Alignment-

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

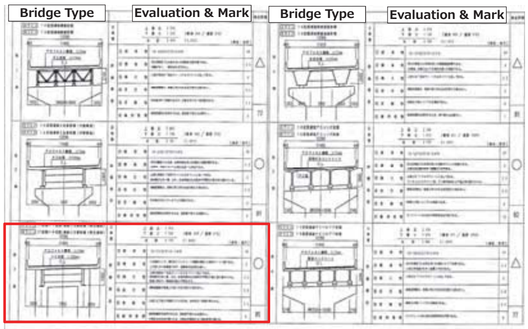
3-D Road Alignment (Harmony of Horizontal & Vertical Alignment)



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

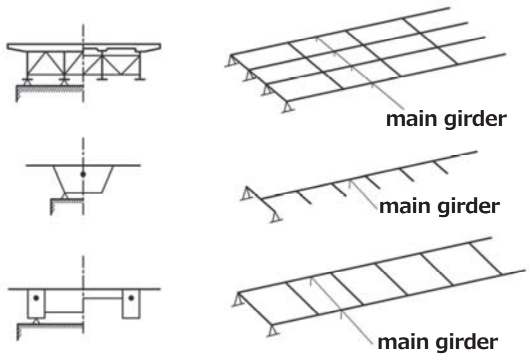
ΣM : Total Maintenance Cost

ΣR : Total Cost to Replacing

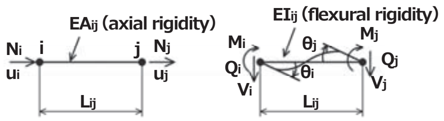
The request in recent years

→ **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

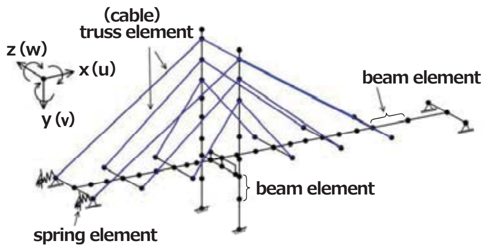
[truss element]

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

↑Stiffness matrix

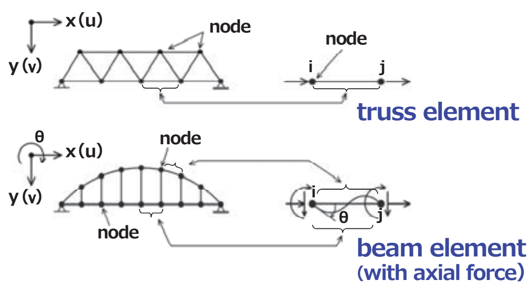
[beam element]

Spatial（3D）model

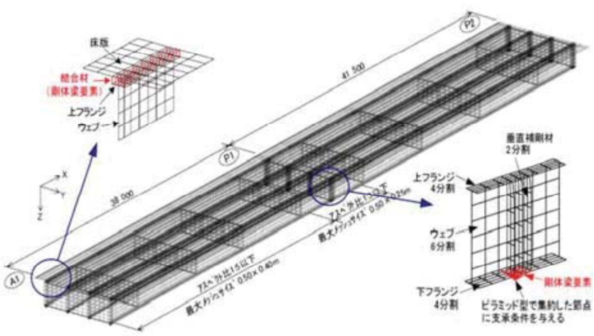


a cable-stayed bridge modeled by
truss and beam elements

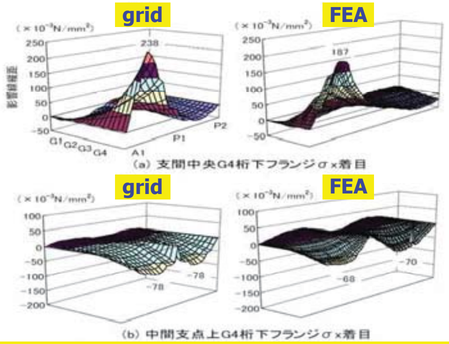
Plane（2D）model
(by truss and beam elements)



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

δ(without impact) $\leqq \delta_a$

where, σ_a, τ_a and δ_a are allowable values

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

Design method (Japan)

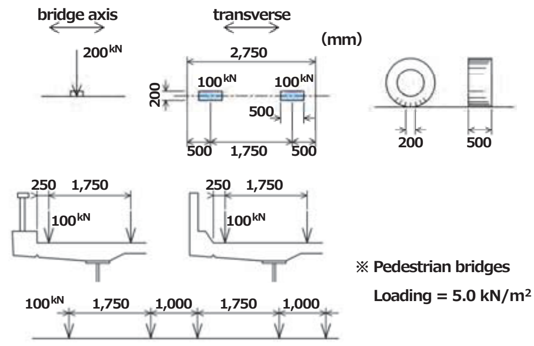
- Allowable stress design method -

Loading

（ ）: notation

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

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



※ Pedestrian bridges
Loading = 5.0 kN/m²

Live Loading

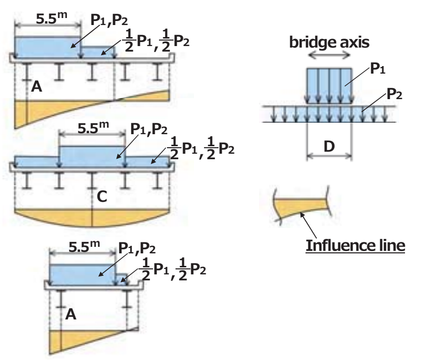
1) Motor vehicle loading （T-load & L-load）
2) Pedestrian loading
3) Tram car （Surface car）

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)

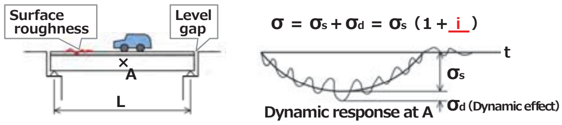
Load intensity for pedestrian bridges

	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)

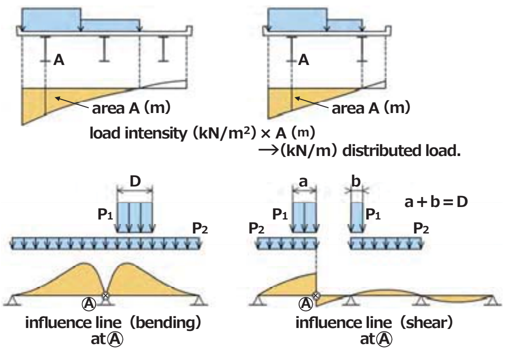
$\sigma = \sigma_s + \sigma_d = \sigma_s\,(1 + i)$



Span (L) for Impact (i)

Continuous girder bridges (＊)	① ① ③ ② L_1 L_2	① L_1 ② L_2 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)



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

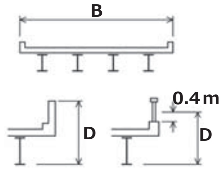
G : gust factor＊＊ (= 1.9)

＊＊ 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

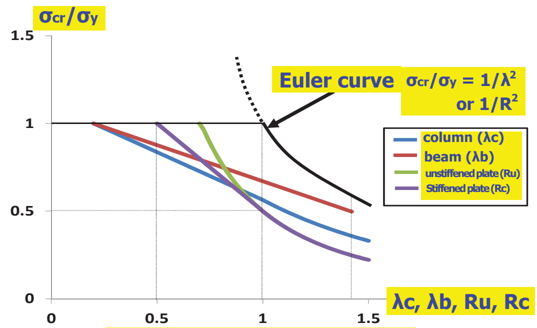


Combination of loading and h＊ (≥ 1.0)

＊ 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

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



$\lambda_c, \lambda_b, R_u, R_c = \sqrt{\sigma_y/\sigma_E}$

σ_y : yield stress, σ_E : elastic buckling stress

[1] Safety check

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

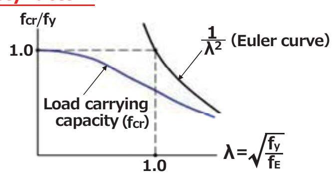
f : stress (σ (or) τ)

f_a : allowable stress

f_y : yield stress

f_{cr} : buckling stress

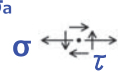
1.7 : safety factor



f_E : Elastic buckling stress

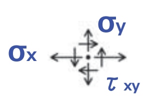
In case of combination (σ, τ)

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



In case of combination (σ_x, σ_y, τ_{xy})

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



σ_e : equivalent stress

1.1 : used in Japanese practice

Material grade and allowable stress

(N/mm²)

	SS400 SM400 SMA400W	SM490	SM490Y SM520 SMA490W	SM570 SMA570W
(mm) $t \leqq 40$ t : thickness	140 80 (235)	185 105 (315)	210 120 (355)	255 145 (450)
$40 < t \leqq 75$	125 75 (215)	175 100 (295)	195 115 (335)	245 140 (430)
$75 < t \leqq 100$			190 110 (325)	240 135 (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	$L \leqq 10$	L/20,000	L/12,000
		$10 < L \leqq 40$	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

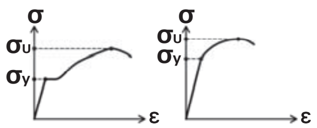
L = span (m)

Safety factor

Safety factor ($\gamma \fallingdotseq 1.7$) is used.

	SS400 SM400 SMA400W	SM490	SM490Y SM520 SMA490W	SM570 SMA570W
(mm) $t \leqq 40$	1.68	1.70	1.69	1.76*
$40 < t \leqq 75$	1.72	1.69	1.72	1.76*
$75 < t \leqq 100$			1.71	1.75*

* σ_y / σ_u is relatively high, γ = 1.75,1.76 is used.



$\sigma_y / \sigma_u < \sigma_y / \sigma_u$

(σ_y / σ_u) : Yield ratio

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

↓

Ex. (limit state 3)

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

h ($\geqq$1.0) : coefficient to take into account of probability of simultaneous occurrence of multiple load actions
v : safety factor
ξ_1 : analysis factor
ξ_2 : member (or) structural factor
γ_i : load factor
Φ_{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.

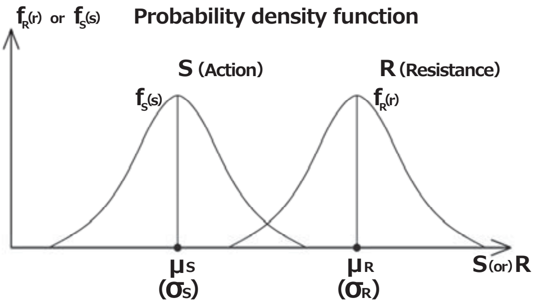
whether or not the required performance level is satisfied

【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
  «$P_f \leqq P_{f,\ target}$»



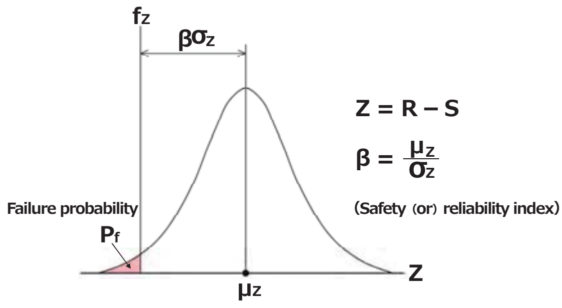
μ : mean value
σ : variance

(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]



$Z = R - S$

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

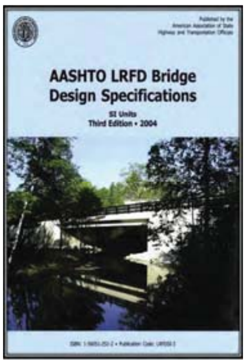
(Safety (or) reliability index)

$\mu_Z = \mu_R - \mu_S$

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

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】

$\Sigma Q_i \leq R_E / FS$

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

【LFD】

$\Sigma \gamma_i Q_i \leq \varphi R$

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

「Ex.」 $1.3(1.0D + 1.67L) = 1.3D + 2.17L \leq \varphi R_n$

【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,
$\beta = 3.5$, $P_f = 2.33 \times 10\text{-}4$ (75-year design life)

EC(target)

$\beta = 3.8$, $P_f = 7.23 \times 10\text{-}5$ (100-year design life)
(↑ ISO)

Limit State to be checked

・Strength Limit State Ⅰ～Ⅴ

Ex. Ⅰ:vehicle running(no wind)

Ⅱ:allowed special type of vehicle (no wind)

・Extreme Limit State Ⅰ, Ⅱ

Ⅰ:erathquake load

Ⅱ:collisoion (ship, ice)

・Serviceability Limit State Ⅰ～Ⅳ

Ex. Ⅱ:Yielding of material

・Fatigue Limit State Ⅰ(forever), Ⅱ(limited duration)

[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)

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

γ_i : load factor*

Φ: resistance factor*

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

[η_D :ductility, η_R : redundancy, η_I: importance]

Q : load effect

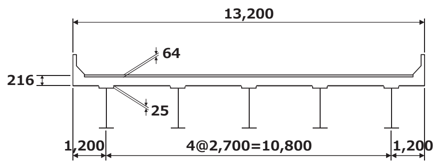
R_n : (Nominal) resistance

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]	

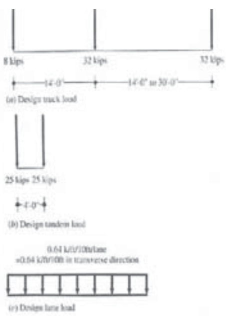
Section classification

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) 72kips(320kN truck)

b) [111.2kNx2] Tanadem load
{↑for short span bridge}

c) 9.34kN/m lane loading
{↑excluding impact effect}

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

Compact, Noncompact & Slender sections

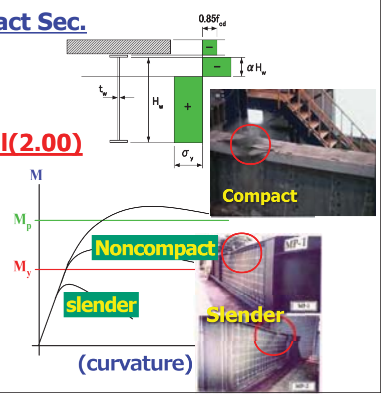
Criteria for compact Sec.

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

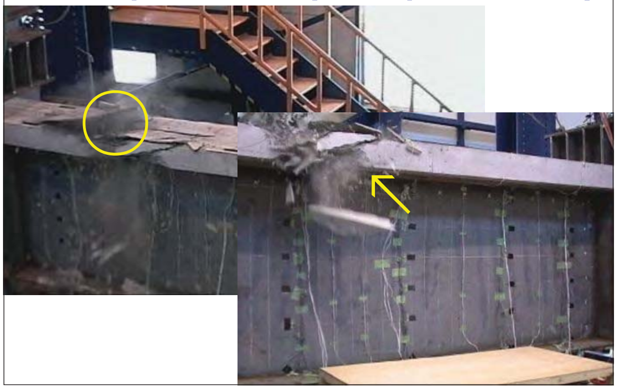
↑our proposal(2.00)

AASHTO LRFD : 1.88

EC (Class 2) : 1.42



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

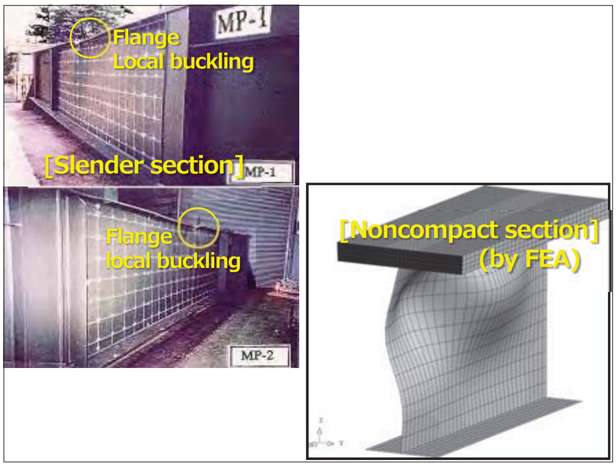


Check of outer girder

[Strength Limit State Ⅰ (flexure and shear)]

M = 1.25(2,119 + 302.5) + 1.50(388.9)
↑DC ↑parapet ↑DW
+1.75(2,961)
↑live load
= 8,792kft ≦ $\Phi_f \cdot M_n$ = 10,973kft
↑=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)
= 44.3kf/in2 ≦
$\Phi_b \cdot F_y$ = 0.95 x 50 = 47.5kf/in2

SI Unit {305N/mm^2 ≦ 327N/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

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)

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

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]

$$\Sigma \gamma_{G,j} G_{k,j} + \gamma_p P + \underbrace{\gamma_{Q,1} Q_{k,1}}_{\uparrow \text{leading variable}} + \underbrace{\Sigma \gamma_{Q,i} \Psi_{0,i} Q_{K,i}}_{\uparrow \text{accompanying variable}}$$

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}

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

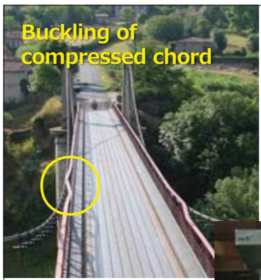
3)Quasi-permanent SLS combination

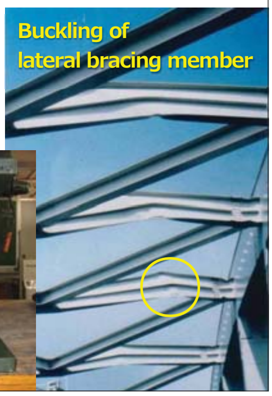
$$\Sigma G_{k,j} + P + \Sigma \Psi_{2,i} Q_{k,i}$$

Thank you for your kind attention


[11-3-1]

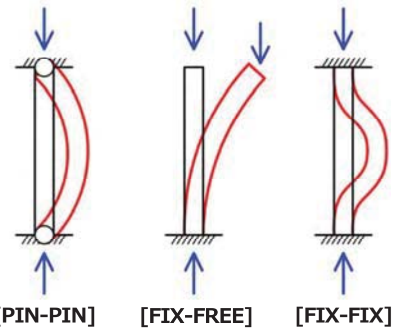
Buckling of Columns & Strength Design



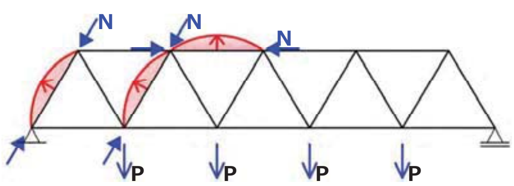


Bucking
of stiffener

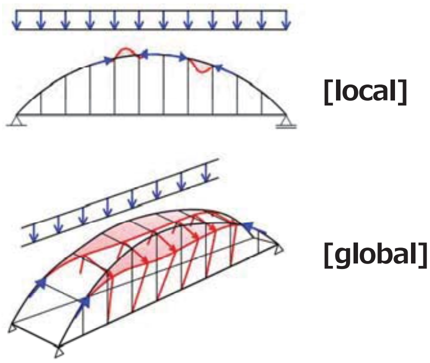
Column Buckling

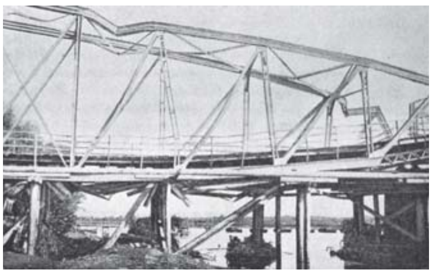


Buckling of Truss Members

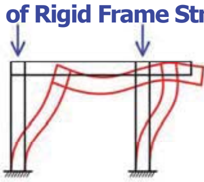


Buckling of Arch Members

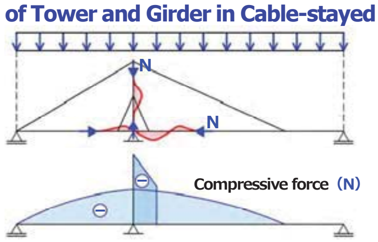


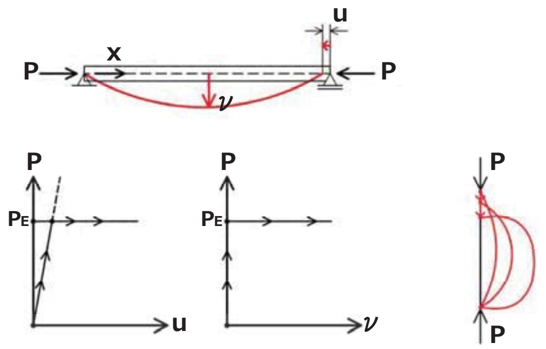


Buckling of Rigid Frame Structures



Buckling of Tower and Girder in Cable-stayed Bridge





Simple support [PIN-PIN]

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

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

From eq. (1) B = D = 0

From eq. (2)

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

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

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

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

Fundamental equation

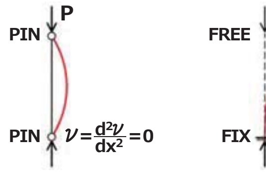
$$EI\frac{d^4\nu}{dx^4} + P\frac{d^2\nu}{dx^2} = 0$$

EI : Flexural rigidity of column

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

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

Boundary conditions



PIN – 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)

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

↑ Buckling load

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

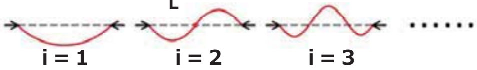
$$\min. P_E = \left(\frac{\pi}{L}\right)^2 EI \quad \text{——(7)}'$$

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

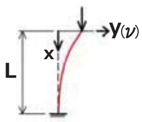
C = 0

Buckling mode shapes are given

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



【FIX - FREE】



Buckling equation

$\cos\alpha L = 0$

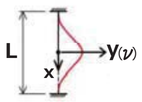
$\alpha L = \frac{2i-1}{2}\pi$ (i=1,2,⋯)

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

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

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

【 FIX - FIX 】



Buckling equation

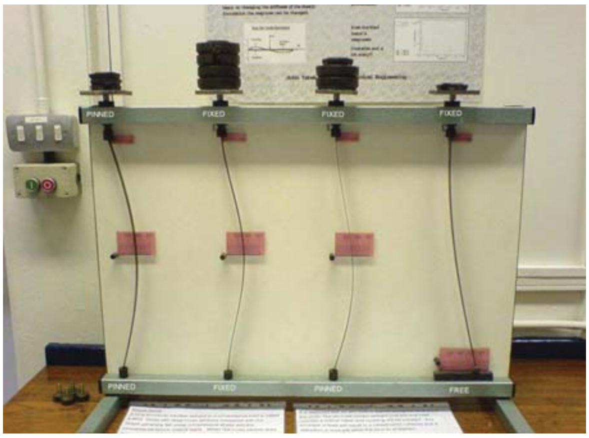
$\sin\frac{\alpha L}{2} = 0$ (or) $\tan\frac{\alpha L}{2} = \frac{\alpha L}{2}$

(This eq. gives min. buckling load)

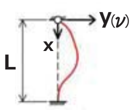
$\frac{\alpha L}{2} = i\pi$ (i=1,2,⋯)

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

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



【PIN - FIX】



Buckling equation

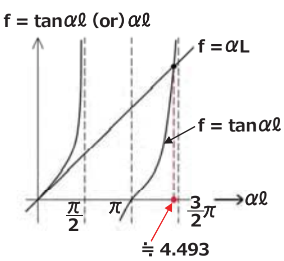
$\tan\alpha L = \alpha L$

$\alpha L \cong 4.493$ (←min. value)

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

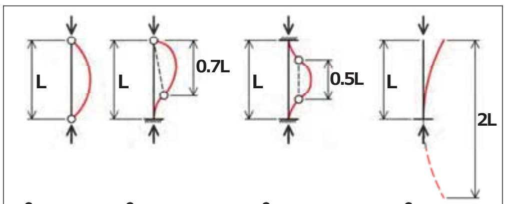
$\min. P_E = \left(\frac{4.493}{L}\right)^2 EI \cong \left(\frac{\pi}{0.7L}\right)^2 EI$

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



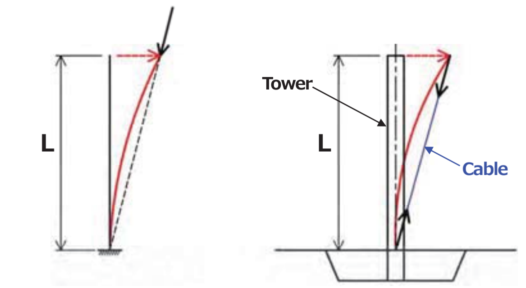
Le : Effective buckling length

β : Buckling coefficient (Le = βL)



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

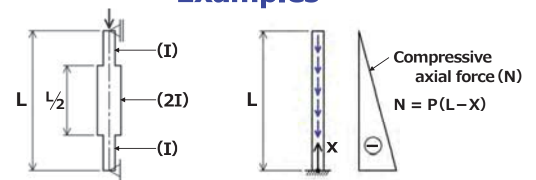
$$P_E = \left(\frac{\pi}{Le}\right)^2 EI = \left(\frac{\pi}{\beta L}\right)^2 EI$$



Le = L　　　Le = L

【Follower force】

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

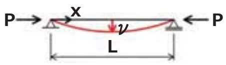
$P_E = \left(\frac{\pi}{0.74L}\right)^2 EI$ ← 0.644 (correct value)　　$P_E = \left(\frac{\pi}{1.09L}\right)^2 EI$ ← 1.22 (correct value)

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

More accurate buckling load will be obtained.

Energy method

(Approximate estimation of buckling load)



・Strain energy of column (due to bending)

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

・Potential energy from load

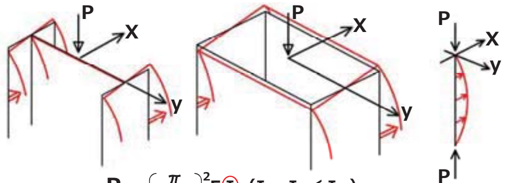
$$\nu = -\frac{1}{2}\int_0^L \left(\frac{d\nu}{dx}\right)^2 dx \quad \text{—— (2)}$$

・Total potential energy of the system

$$\pi = U + \nu$$

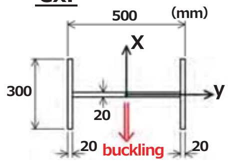
$$\delta\pi = \delta(U+\nu) = 0 \quad (\leftarrow \text{Stationary condition}) \quad \text{—— (3)}$$

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



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

ex.



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

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

$\text{Iy} < \text{Ix}$

$[A = 2\times30\times2 + 46\times2 = 2.12\times10^2\,(\text{cm}^2)]$

Calculation of the buckling load

$(\text{I} = \text{Iy} = 9\times10^7\,\text{mm}^4)$

1) L = 5^m (PIN−PIN)

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

2) L = 10^m (PIN−PIN)

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

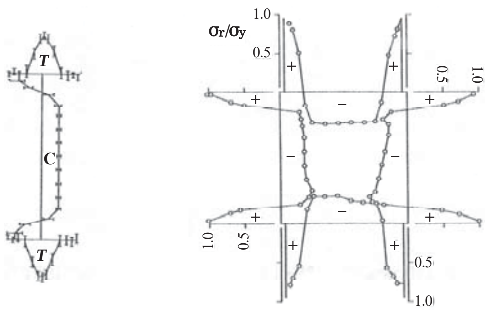
3) L = 10^m (FIX−FIX)

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

4) L = 10^m (FIX−FREE)

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

Distribution of residual stress due to welding

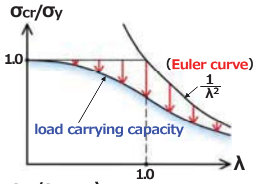


I-section　　Box-section

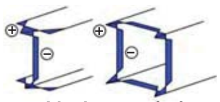
Buckling stress (σE) and ultimate strength (σcr)

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

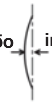
$\frac{\sigma_E}{\sigma_y} = \frac{1}{\lambda^2}$　　$\lambda = \sqrt{\frac{\sigma_y}{\sigma_E}} = \frac{1}{\pi}\sqrt{\frac{\sigma_y}{E}}\cdot\frac{Le}{r}$ (← slenderness ratio)



σcr/σy ∼ λ curve



residual stress (σr) due to welding



initial deflection (δo)

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

Ultimate strength of columns (σcr) by JHBS

$\frac{\sigma_{cr}}{\sigma_y} = 1.0$	$\lambda \leqq 0.2$
$\frac{\sigma_{cr}}{\sigma_y} = 1.109 - 0.547\lambda$	$0.2 < \lambda \leqq 1.0$
$\frac{\sigma_{cr}}{\sigma_y} = \frac{1.0}{0.773 + \lambda^2}$	$1.0 < \lambda$

ex.

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

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

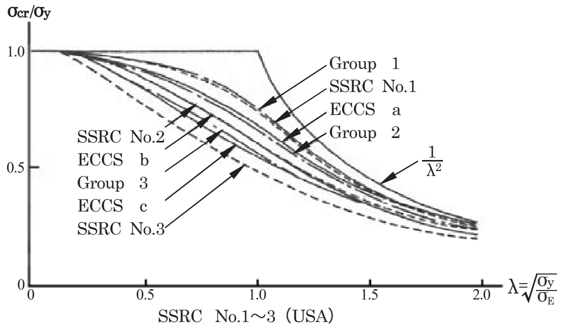
$\sigma_E = P_E/A = 7.0989\times10^6/2.12\times10^4 = 355.9\ \text{N/mm}^2$

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

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

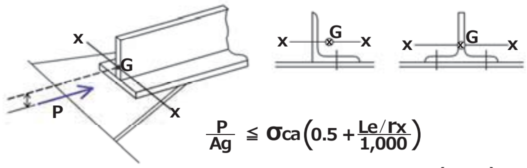
σ_a (allowable stress) $\cong \sigma_{cr}/1.7 = 91.8\ \text{N/mm}^2$

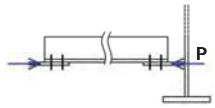
Load carrying capacity curve for columns



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

Eccentricity of loading



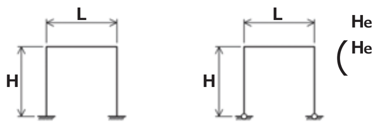


$$\frac{P}{Ag} \leqq \sigma ca\left(0.5+\frac{Le/rx}{1{,}000}\right)$$

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



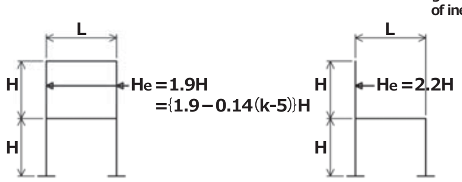
He = 1.5H
={1.5 − 0.04(k-5)}H

He = 3.5H
={3.5 − 0.2(k-5)}H

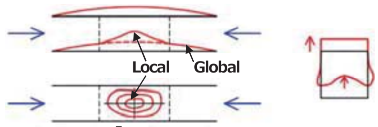
He : effective buckling length
(He = upper value (k ≦ 5)
Lower value (5 < k ≦ 10))

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

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



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



【JHBS , Product formula】

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

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

【American practice* , Q-factor method】

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

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

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

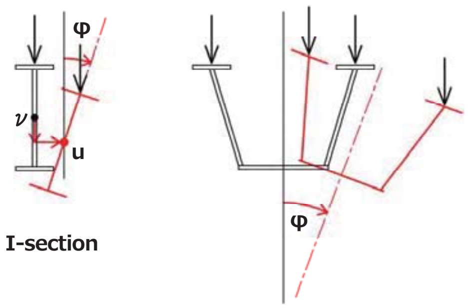
$f(\bar{\lambda}_i)$: column buckling strength

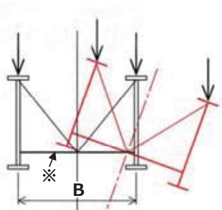
* American Institute of Steel Construction

[11-3-2]

Buckling of Beam & Beam-column & Strength Design

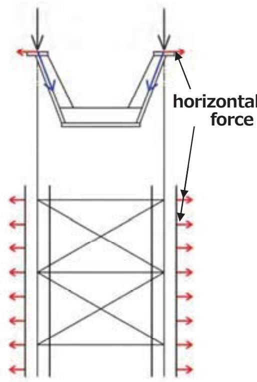
Lateral Torsional Buckling



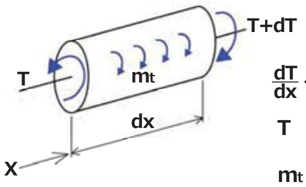


※ lower lateral bracing

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



Member under torsion



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

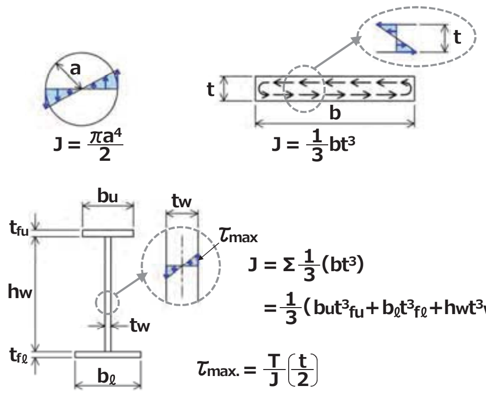
T : torsional moment (torque)

m_t : torsional load

$T = T_w + T_s$

T_w : Warping torsion

T_s : St.Venant (or Pure) torsion

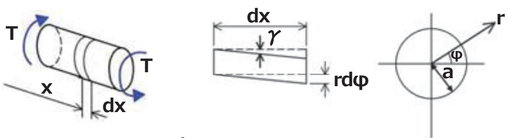


$$J = \frac{\pi a^4}{2} \qquad J = \frac{1}{3} b t^3$$

$$J = \Sigma \frac{1}{3}(bt^3) = \frac{1}{3}(b_u t_{fu}^3 + b_\ell t_{f\ell}^3 + h w t^3 w)$$

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

Pure torsion of round bar



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

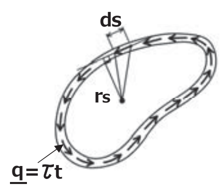
$$T = \int_A (\tau dA)\cdot r = G\underbrace{\boxed{\int_A r^2 dA}}_{Ip}\cdot\frac{d\varphi}{dx} = GJ\frac{d\varphi}{dx} \text{ ——(2)}$$

J : St.Venant torsional constant

GJ : St.Venant torsional rigidity

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

Pure torsion of tubular structures



$q = \tau t$
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, $\oint du = 0$

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

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

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

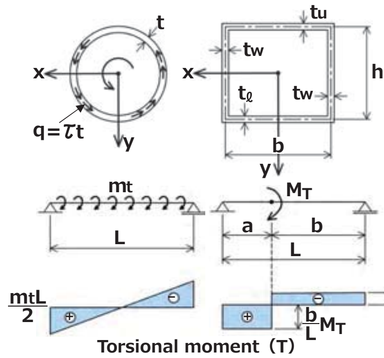
$$\left(q = \frac{T_s}{2A}\right)$$

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

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

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

Pure torsion (T=Ts)



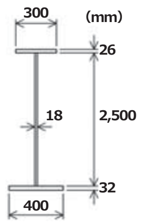
Torsional moment (T)

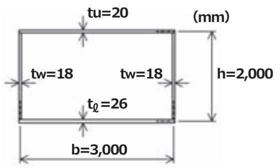
$$A = bh$$

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

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

ex.





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

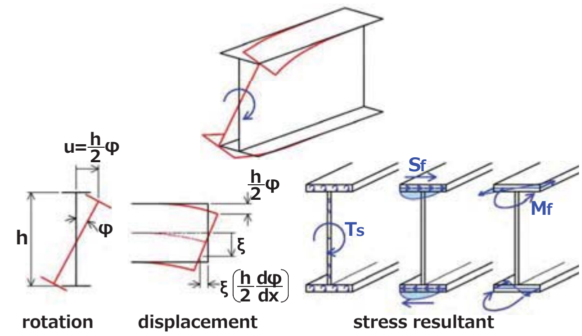
$$= \underline{3{,}296\times10^7\ (\text{mm}^4)}$$

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

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

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

Torsion of open section



$$T_w = S_f \cdot h$$

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

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

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

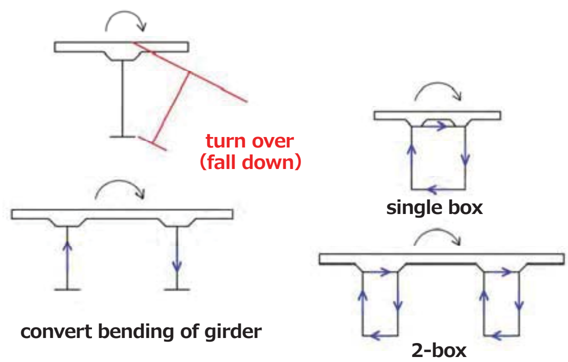
$$T_s = GJ\frac{d\varphi}{dx}$$

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

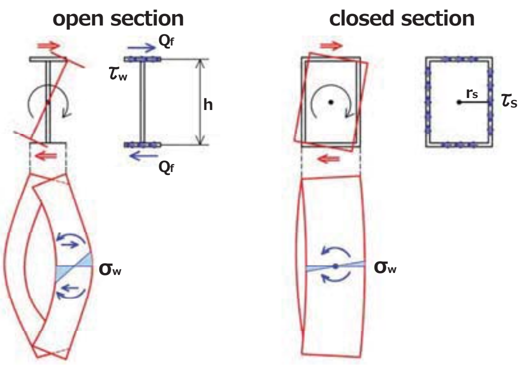
$$\underline{EI_w\frac{d^4\varphi}{dx^4} - GJ\frac{d^2\varphi}{dx^2} = m_t} \quad \text{(for open section)}$$

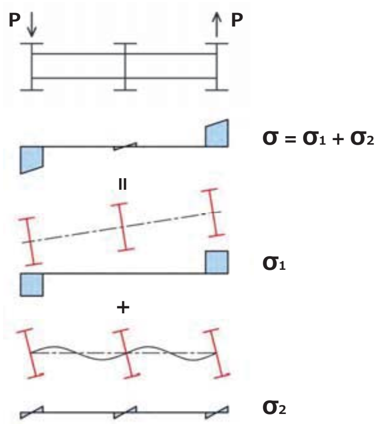
when EIw is small compared to GJ

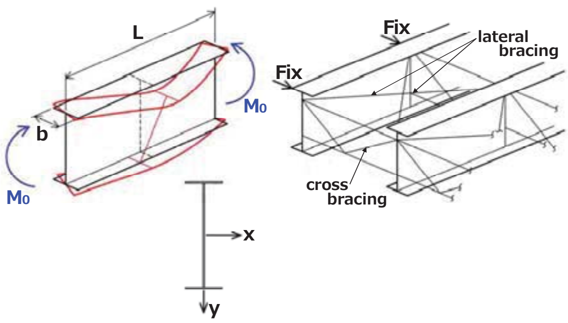
$$\underline{GJ\frac{d^2\varphi}{dx^2} = -m_t} \quad \text{(for closed section)}$$



Behavior under torsion





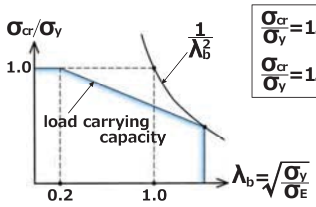


$$(M_0)_E = \left(\frac{\pi}{L}\right)\sqrt{EI_yGJ\left(1+\frac{\pi^2EI_w}{L^2GJ}\right)}$$

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

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

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



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

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

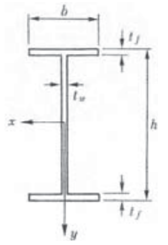
$A_W/A_F \leqq 2$ $\quad K = 2$

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

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

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

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



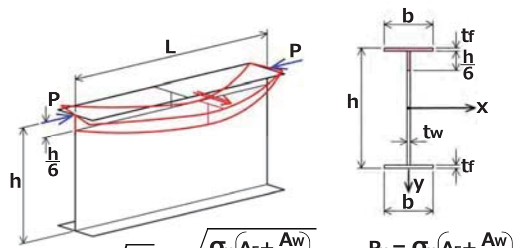
$$A = 2bt_f + (h-2t_f)t_w = 2A_F + A_W$$

$$I_y \fallingdotseq A_F b^2/6$$

$$I_w \fallingdotseq I_y h^2/4$$

$$J = \frac{A_F}{3}(2t_f^2 + A_W t_w^2/A_F)$$

$$W = A_F h(1 + A_W/6A_F)$$



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

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

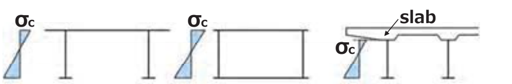
$$K = \sqrt{3+\frac{A_W}{2A_F}}$$

the same as λ_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



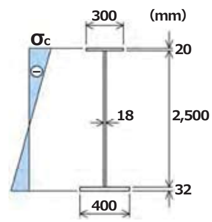
Allowable compressive stress in flexure (σ_{ba})

(N/mm²)

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

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

ex.



$L = 5{,}000$ mm

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

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

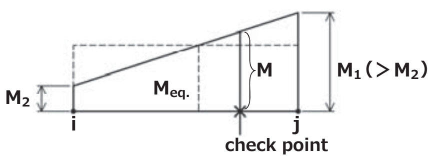
$$\lambda_b = \frac{2}{\pi}\times2.60\times\sqrt{\frac{235}{2\times10^5}}\times\left(\frac{5{,}000}{300}\right) = 0.946$$

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

$$\sigma_{cr} = 0.693\times235 = 162.9 \text{ (N/mm}^2\text{)}$$

$$\sigma_{ba} = \sigma_{cr}/1.7 = 95.8 \text{ (N/mm}^2\text{)}$$

In case of ununiform bending moment

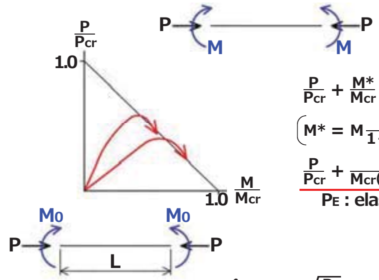


$$M_{eq.} = \min.\{(0.6M_1 + 0.4M_2) \text{ or } 0.4M_1\}$$

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

however not exceeding (σ_y/γ)

Bending moment (M) and compressive force (P)



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

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

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

P_E : elastic buckling load

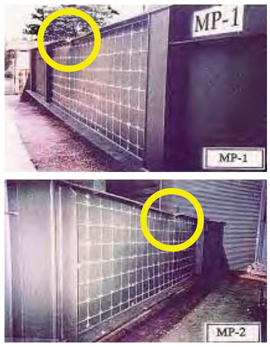
$$M_{max.} = M_0\sec\frac{\alpha\ell}{2}\ , \quad \alpha = \sqrt{\frac{P}{EI}}$$

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

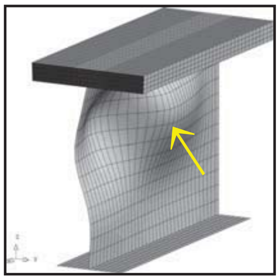
[11-3-3]

Buckling of Plate & Strength Design

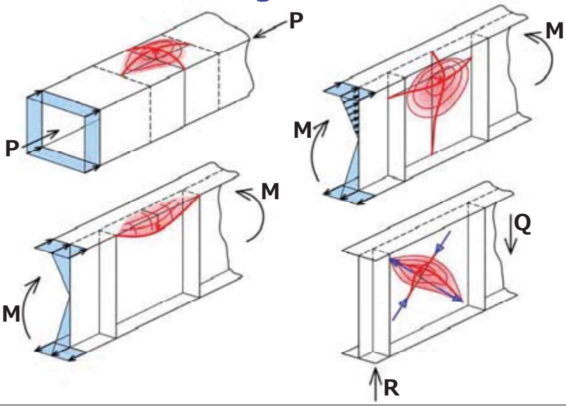
Local buckling of compressed plate



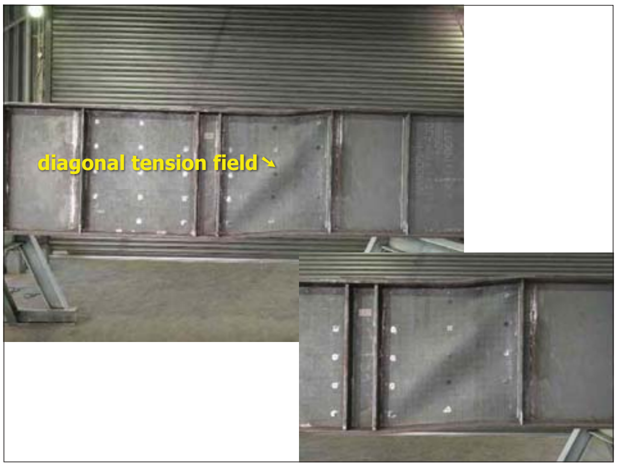
Local buckling of web



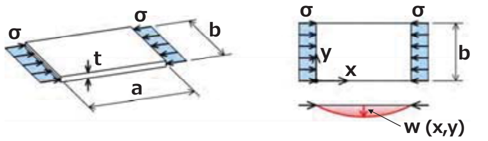
Buckling of Plates



Shear buckling



Fundamental equation



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

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

Assuming, $w(x,y) = \underbrace{A_{m,n}\sin\frac{m\pi}{a}x\cdot\sin\frac{n\pi}{b}y}_{(m,\,n=1,2,3\cdots\cdots)}$ (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 $n=1$, and setting $\alpha=a/b$ (α: aspect ratio)

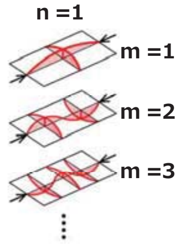
$$\sigma_E=\frac{\pi^2 B}{b^2 t}\left(\frac{m}{\alpha}+\frac{\alpha}{m}\right)^2$$

We rewrite

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

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

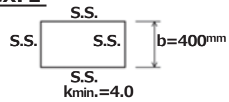
k : buckling coefficient of plate



(a/b= 2)

Examples

ex. 1



$k_{min.}=4.0$

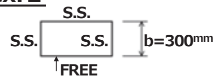
[t=10mm]

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

[t=6mm]

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

ex. 2



$k_{min.}=0.426$

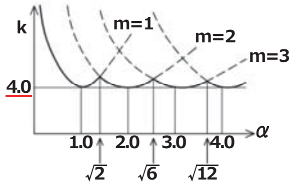
[t=25mm]

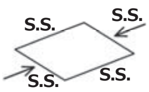
$$\sigma_{E0} = 1.8058\times10^5\times\left(\frac{25}{300}\right)^2$$
$$= 1254.0\ (\mathrm{N/mm^2})$$
$$\sigma_E = 0.426\times\sigma_{E0} = 534.2\ (\mathrm{N/mm^2})$$

[t=15mm]

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

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

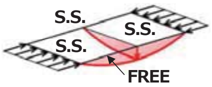




S.S. : Simply Supported

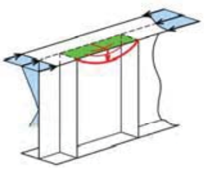
$$\left(w=0,\ \frac{\partial^2 w}{\partial x^2}=\frac{\partial^2 w}{\partial y^2}=0\right)$$

$k_{min.} = 4.0$



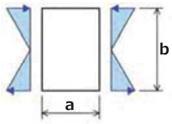
$$k = \frac{1}{\alpha^2} + 0.426$$

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



K value depending on stress condition

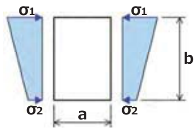
$(\sigma_E = k\sigma_{E0})$



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

$2/3<\alpha$ $K \cong 23.9$

$\alpha<2/3$ $K \cong 15.87+1.87\alpha^2+8.6/\alpha^2$

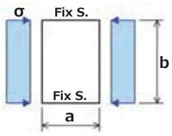


$\phi = \frac{\sigma_1-\sigma_2}{\sigma_1}$

$K = \frac{8.4}{2.1-\phi}$ $0\leq\phi\leq1$

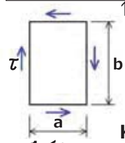
$K = 10\phi-13.73\phi-11.36$ $1<\phi\leq2$

$\alpha>1.0$



$2/3<\alpha$ $K \cong 7.0$

$\alpha<2/3$ $K \cong 2.366+5.3\alpha^2+\frac{1}{\alpha^2}$



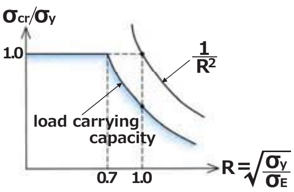
$(\tau_E = k_\tau \sigma_{E0})$

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

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

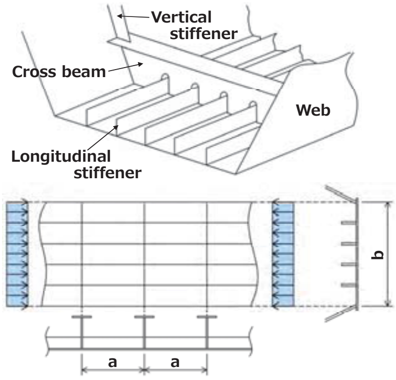
Ultimate strength of unstiffened plate by JHBS

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



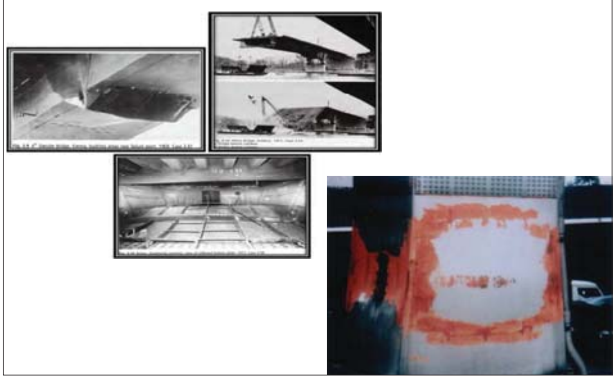
$\frac{\sigma_{cr}}{\sigma_y} = 1.0$	$R \leqq 0.7$
$\frac{\sigma_{cr}}{\sigma_y} = 0.5/R^2$	$0.7 < R$

Stiffened Plate

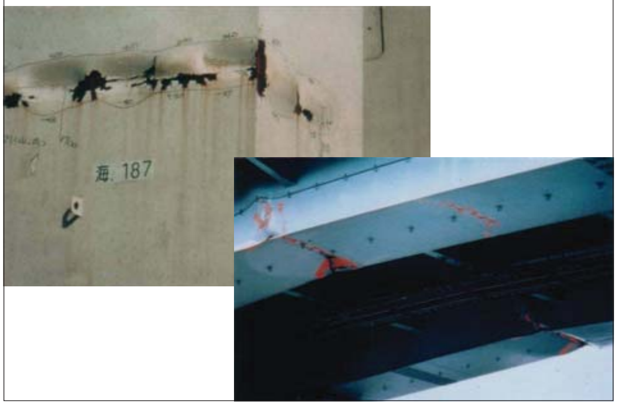


Buckling of stiffened plate

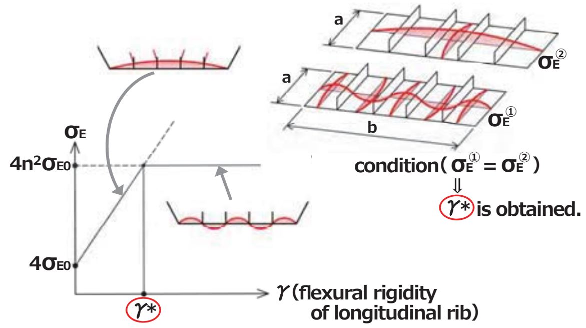
Global buckling of stiffened plates



Local buckling of stiffened plates

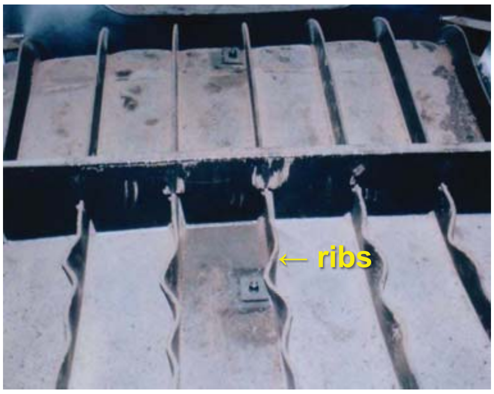


{Local elastic buckling stress} = {Global elastic buckling stress}



n : number of panel
n-1 : number of longitudinal ribs

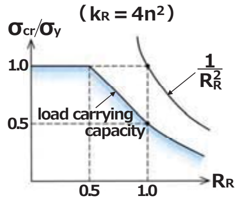
Buckling of longitudinal ribs



Ultimate strength of stiffened plate by JHBS

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

$(k_R = 4n^2)$

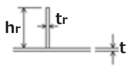


$\frac{\sigma_{cr}}{\sigma_y} = 1.0$	$R_R \leqq 0.5$
$\frac{\sigma_{cr}}{\sigma_y} = 1.5 - R_R$	$0.5 < R_R \leqq 1.0$
$\frac{\sigma_{cr}}{\sigma_y} = \frac{0.5}{R_R^2}$	$1.0 < R_R$

Design of longitudinal ribs (I_ℓ)

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

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

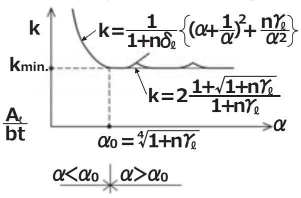


$$A_\ell = h_r t_r \qquad I_\ell = \frac{h_r^3 t_r}{3}$$

From condition,

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

$$k = 4n^2$$

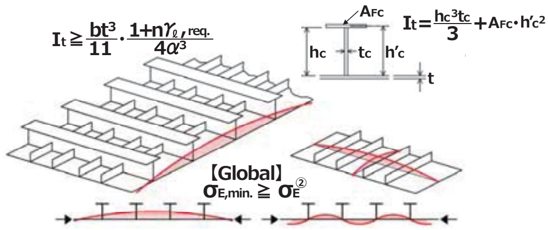


$$\delta_\ell = \frac{A_\ell}{bt}$$

Design of cross beam (I_t)

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

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



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

$$\gamma_{t,req.} = \frac{1+n\gamma_{\ell,req.}}{4\alpha^3}\underline{-\frac{\alpha}{2} + \frac{5}{4(1+n\delta_\ell)}}$$ 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) $\alpha \leqq \alpha_0$ (&) $I_t \geqq \frac{bt^3}{11}\cdot\frac{1+n\gamma_{\ell,req.}}{4\alpha^3}$

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

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

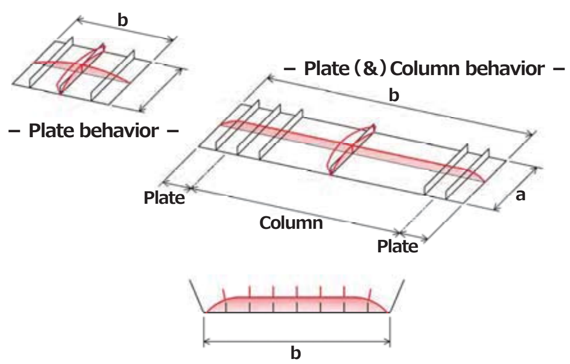
(t_0 is the thickness when $R_R = 0.5$)

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

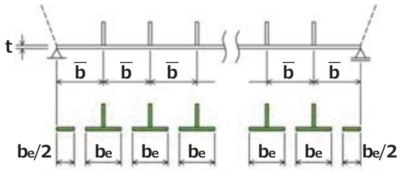
$\gamma_{\ell,reg.} = \frac{1}{n}\left[\left\{2n^2\left(\frac{t_0}{t}\right)(1+n\delta_\ell) - 1\right\}^2 - 1\right]$ $\quad (t \geqq t_0)$ $\quad (R_R < 0.5)$

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

Buckling mode shape of plate with large α



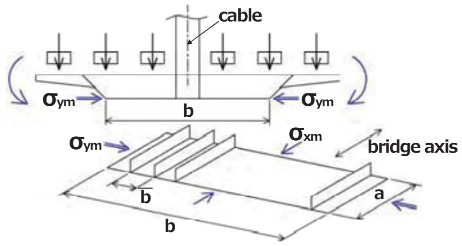
Column approach



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

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)

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

Evaluation of strength

$(\sigma_{cr})_c$

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

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

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

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

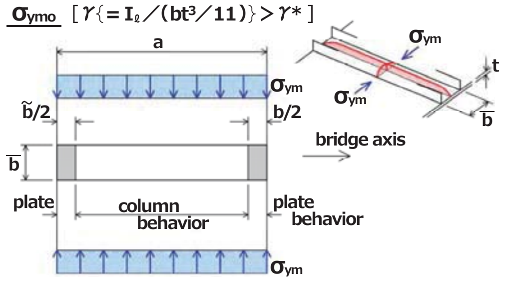
$$R_e = 0.526\frac{b}{t}\sqrt{\frac{\sigma_{cr}}{E}}$$

First, σ_{cr} is assumed and repeat calculation until converged σ_{cr} is obtained

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

Strength of plate under σ_{ym} only (σ_{ymo})

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



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

σ_{ymo} : strength under σ_{ym} only

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

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

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

↑proposed by Komatsu (1978)

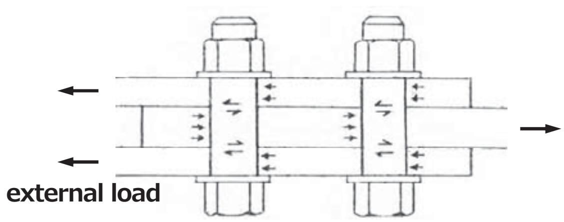
$$R = \frac{1}{\pi}\sqrt{\frac{\sigma_y}{E} \cdot \frac{12(1-\nu^2)}{\pi^2 k}} \cdot \left(\frac{b}{t}\right) \quad (k = 4.0)$$

[11-4-1]

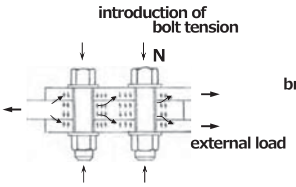
Design of Connection (Bolt)

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

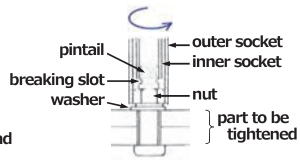
Bearing connection



Friction grip connection

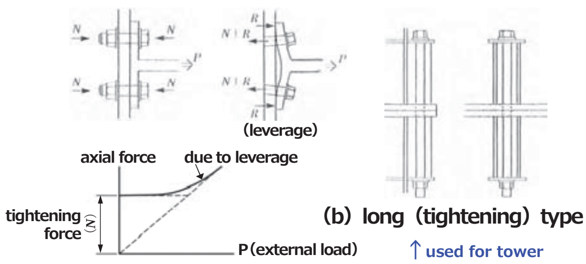


【 hexagon headed bolt 】



【 torque-shear bolt 】 tension introduction

Tensile-type bolted connection

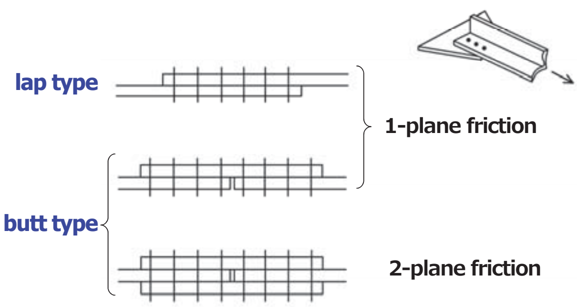


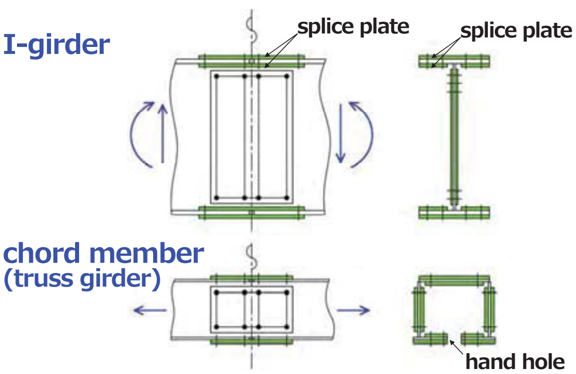
(a) short (tightening) type

(b) long (tightening) type

↑ used for tower of suspension bridges

Bolt Connection





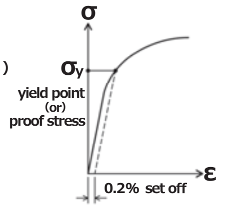
Tensile test

Allowable bolt force (ρ_a) per one friction plane

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

$$N = \alpha \sigma_y A_e$$

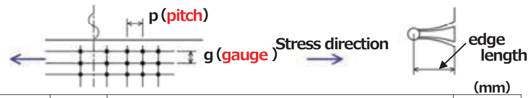
μ : friction (slip) coefficient ($\mu = 0.4$)
γ : Safety factor ($\gamma = 1.7$)
α : ratio to yield point
$\alpha = 0.85$ (F8T)
$= 0.75$ (F10T, S10T)
σ_y : yield point
A_e : effective area of bolt thread



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 (staggered type) 15t − $\frac{3}{8}$g not exceeding 12t	24t not exceeding 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 (ρ_a)

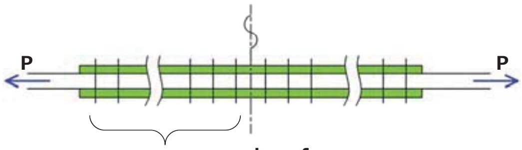
bolt grade	nominal designation of thread	γ	μ	α	σ_y (N/㎟)	A_e (㎟)	N (kN)	ρ_a (kN)
F8T*	M20** M22 M24	1.7	0.4	0.85	640	245 303 353	133 165 192	31.3(31) 38.8(39) 45.2(45)
F10T *** S10T	M20 M22 M24	1.7	0.4	0.75	900	245 303 353	165 205 238	38.8(39) 48.2(48) 56.0(56)

(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
($n_k \leq 8$) 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 (σ_a : allowable stress)

[In tension]

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

σ_{ta} : allowable tensile stress
A_n : net cross-sectional area*

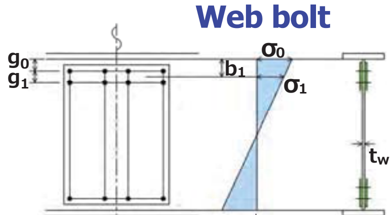
*hole width (d + 3 : mm) for bolt with a diameter (d : mm) is subtracted

[In compression]

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

σ_{ca} : allowable compressive stress
A_g : gross cross-sectional area

Web bolt



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

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

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

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

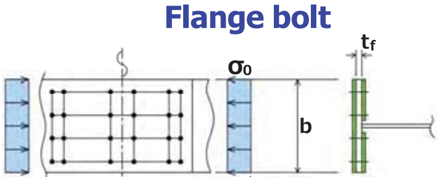
n : number of bolt

$g_0 = 100$ 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



$P = \sigma_0 b t_f$

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

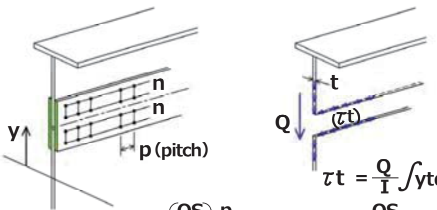
$n_{req.} > \frac{p}{\rho_a}$

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



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

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

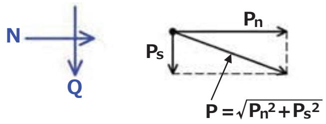
$n_{req.} > \frac{\left(\frac{QS}{I}\right) p}{\rho_a}$

When subjected to shear force (Q)

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

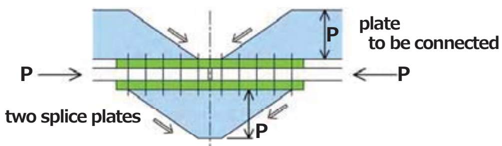
When subjected to axial force (N) and shear

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



Design of splice plates

Force transfer



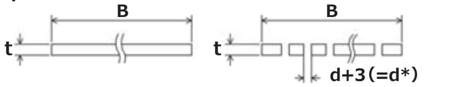
(1) in compression

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

(2) in tension

net sectional area (A_n) has to be used

plate to be connected also has to be checked



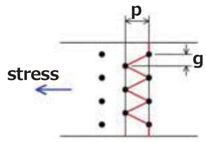
$A_g = Bt$ (gross area)

$A_n = (B - n \times d^*)\, t$

n : number of bolt

d : diameter of bolt

【 in case of staggered arrangement 】

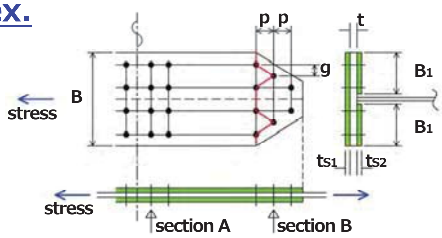


$$A_n = A_g - \{4d^* - 3w\}\, t$$

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

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

ex.



Net area of splice plate (Section A)

$$A_n = (B - 4d^*)\, t_{s1} + 2\,(B_1 - 2d^*)\, t_{s2}$$

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)

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

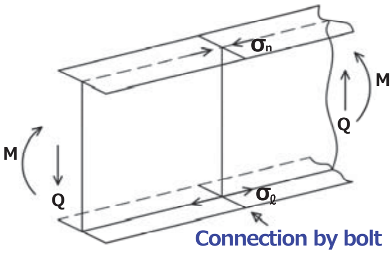
σ : stress at tip (fiber) of splice plate

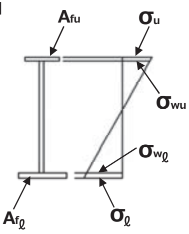
M : bending moment (carried by splice plate)

I : geometrical moment of inertia of splice plate

ex. will be given later

Design of bolt connection





Shear force : Q

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

[upper flange]

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

σ_{ca} : allowable compressive stress

(1) Number of bolt ($n_{req.}$)

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

(2) Splice plate

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

σ_{ta} : allowable tensile stress

(or) $A_{SpL} > A_{fu}$

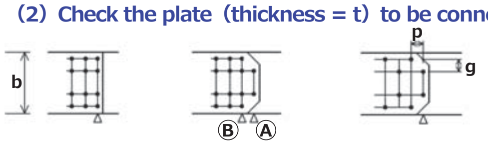
[lower flange]

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

(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



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

Ⓐ $A_{net} = \{b - 2(d+3)\}\, t$

Ⓑ $A_{net} = \{b - 4(d+3)\}\, t$

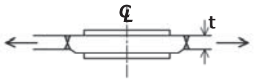
if $w = d - p^2/4g > 0$
$A_{net} = \{b - 2(d+3) - 2w\}\, t$

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

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

if the above is not satisfied,

(a) plate thickness is increased



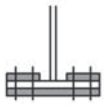
(b) change bolt arrangement

(3) Splice plate

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

A_{SpL} : net cross sectional area

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



(3) Splice plate

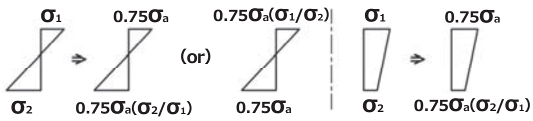
$$\sigma_{SpL} = \frac{M_S}{W_S} < \sigma_{ta}$$

M_S : moment on splice plate

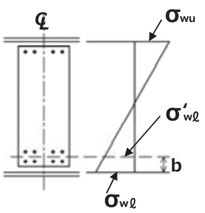
W_S : section modulus of splice plate

(or) $I_{SpL} > I_w$

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



[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

Bearing connection

allowable shear stress (N/㎟)

B8T	B10T
150	190

Tension-type bolt connection

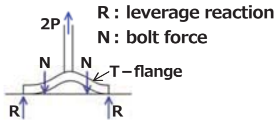
【 short bolt 】

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

P : tension force

p_y : leverage action

n : number bolt



ρ_{at} (allowable force) (kN)

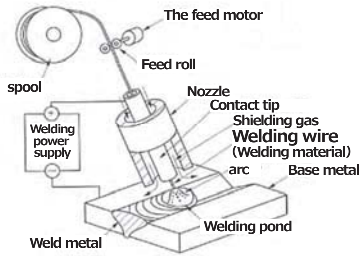
	F10T	S10T
M20	130	130
M22	160	160
M24	185	185

【 long bolt 】

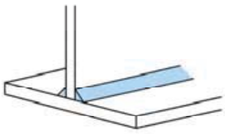
$$P_p = \frac{P}{n} < \rho_{a\ell}$$

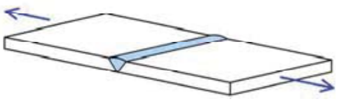
Design of Connection (Welding)

Gas shield arc welding method



Welding

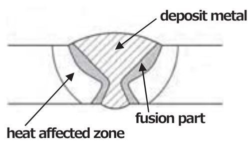




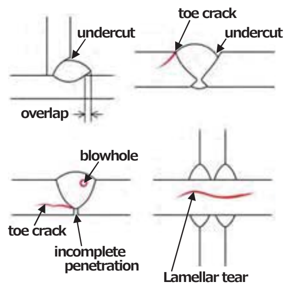
Fillet Welding   Groove Welding

Welded section and defects

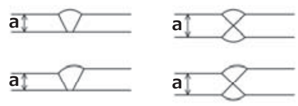
[Section]

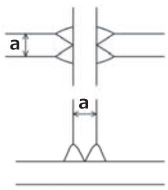


[Defects]



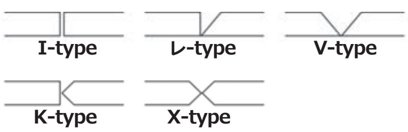
Welding type (groove welding)





a : throat

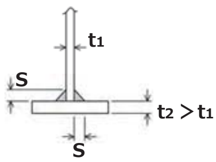
groove shape



Design method

1) Size of fillet welding

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



t_1 : thinner plate thickness

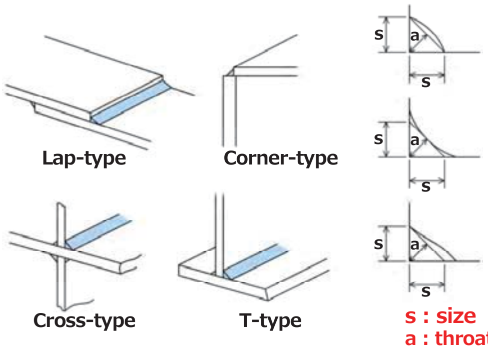
t_2 : thicker plate thickness

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

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

$18 > S > \sqrt{2 \times 36} = \sqrt{72} = 8.5 \quad \rightarrow \underline{S = 9\text{mm}}$

Welding type (fillet welding)

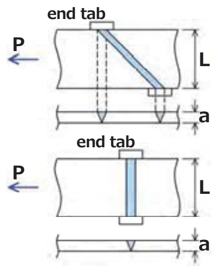


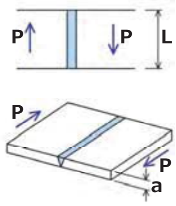
s : size
a : throat

2) Axial force and shear

a) groove welding

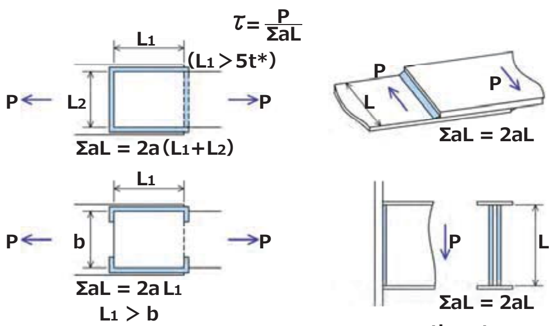
$\sigma = \frac{P}{\Sigma aL}$ $\tau = \frac{P}{\Sigma aL}$





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

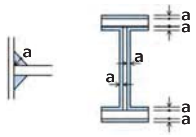
【 groove welding 】

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

【 fillet welding 】

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

– fillet welding –



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 】

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

σ : normal stress due to axial force and/or bending

τ : shear stress due to axial force and/or bending

τ_s : shear stress due to shear force

σ_a : allowable tensile stress

τ_a : allowable shear stress

Allowable stress

(N/mm²)

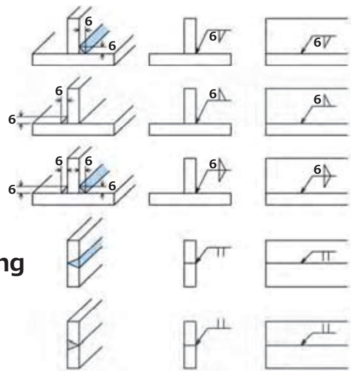
		SM400 SMA400W	SM490	SM490Y SM520 SMA490YW	SM570 SMA570W
groove welding	σ_a	140	185	210	255
	τ_a	80	105	120	140
fillet welding	τ_a	80	105	120	140

($t \leq 40$mm)

Welding symbol (1)

Fillet welding

Groove welding (I-type)

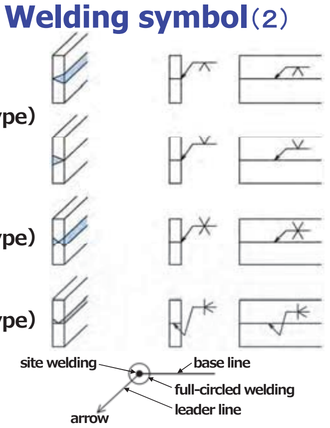


Welding symbol (2)

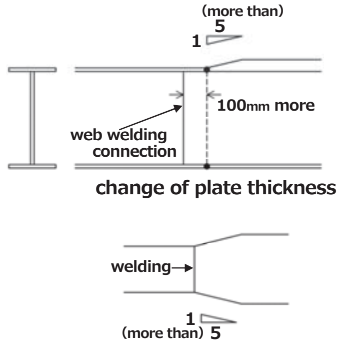
(V-type)

(X-type)

(K-type)



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

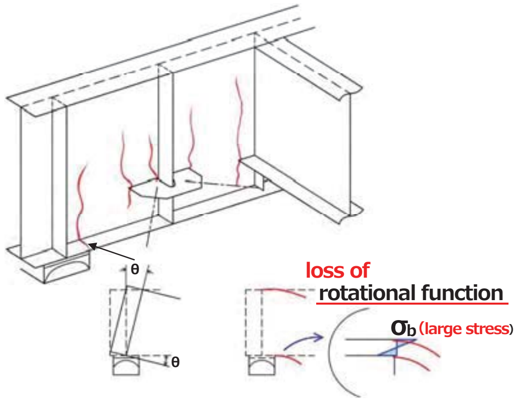
Example of fatigue cracks

Collapse, Failure of Steel Structures

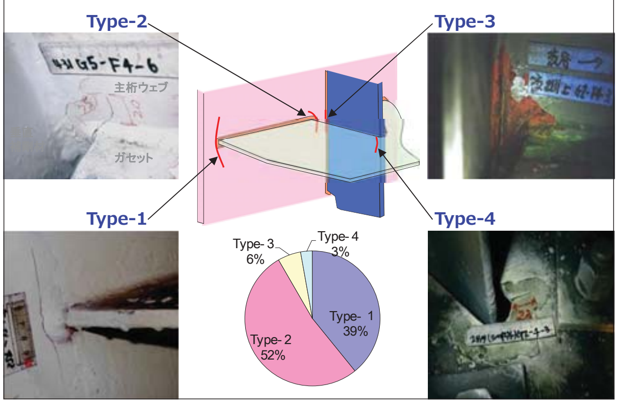
<u>Buckling strength</u> and <u>material break</u> depend on material strength

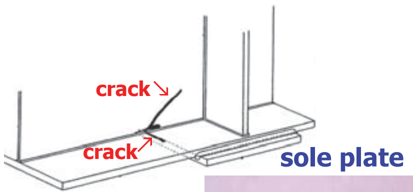
However, <u>fatigue strength</u> does not depend on material strength

Example of cracking（1）

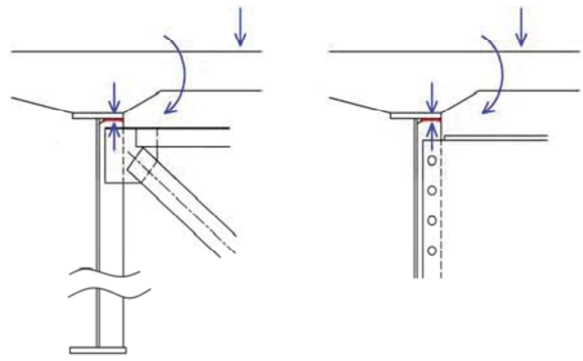


Fatigue cracking at gusset

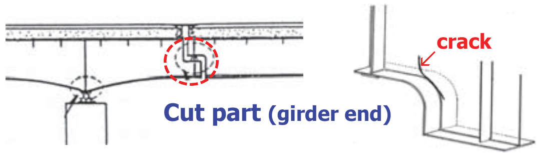


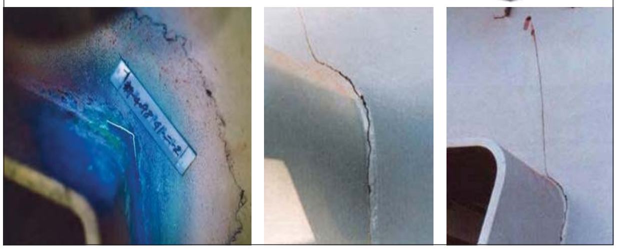


Example of cracking（2）



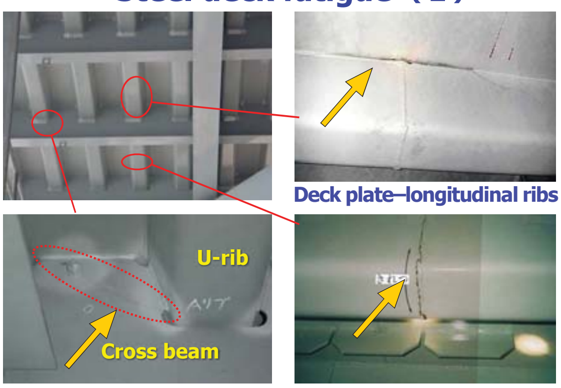
Example of cracking（3）





Example (4)
Steel deck fatigue

Steel deck fatigue（1）



Deck plate–longitudinal ribs

Cross beam–longitudinal rib　Welding at longitudinal ribs

[Recommendation]

Structural details with stress concentration should be avoided.

Evaluation of fatigue strength of such part is difficult.

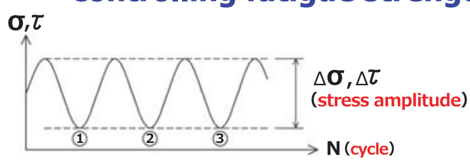
Steel deck fatigue（2）

Penetrating to deck surface

Fatigue limit state

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

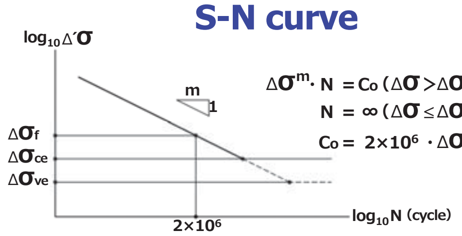
Dominate parameter controlling fatigue strength



{(Δσ,Δτ) and N}

important parameter

S-N curve

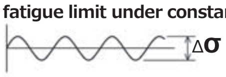


$\Delta\sigma^m \cdot N = C_0 \ (\Delta\sigma > \Delta\sigma_{ce}, \Delta\sigma_{ve})$

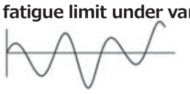
$N = \infty \ (\Delta\sigma \leq \Delta\sigma_{ce}, \Delta\sigma_{ve})$

$C_0 = 2\times10^6 \cdot \Delta\sigma_f^{\,m}$

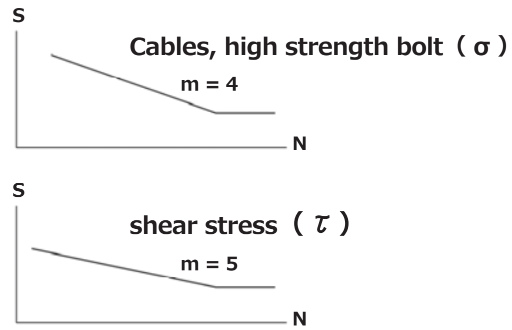
$\Delta\sigma_{ce}$: fatigue limit under constant stress amplitude



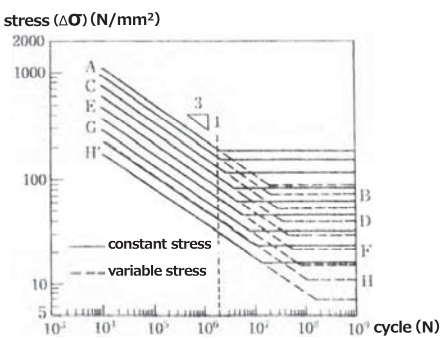
$\Delta\sigma_{ve}$: fatigue limit under variable stress amplitude



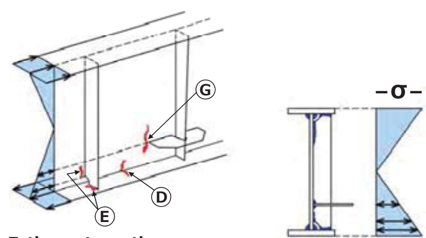
S-N curves



Fatigue design curve（normal stress σ）



Check of fatigue strength（normal stress）



Fatigue strength (N/mm²)

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

$C_R = 1.00$　　$-1.0 < R < 1.0$

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

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

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

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

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

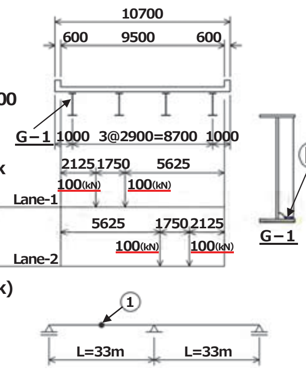
ex.

・Design period 100-year (life)

・ADTT (per one way, lane) = 3,000
Average Daily Truck Traffic

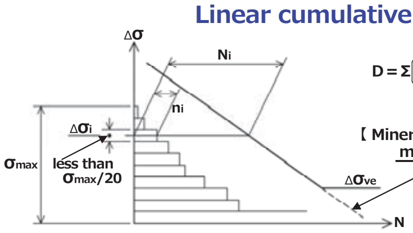
・Design load for fatigue check
(T-Load) × (1 + if)
T-Load : 100kN (2-point)

・if = 10 / (50+L)
↑half value (at fatigue check)



Step－2

Linear cumulative damage rule

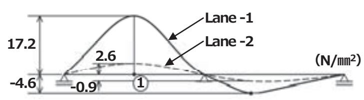


$D = \Sigma\left(\frac{n_i}{N_i}\right) = 1.0$
↑
fatigue damage

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

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

Step－1



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

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

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

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

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

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

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

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))

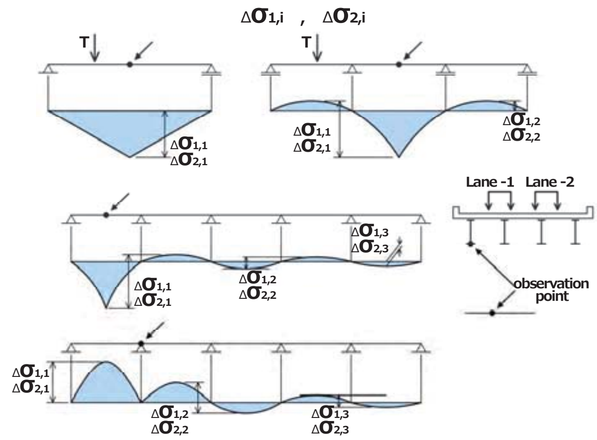
$NT_i = ADTT_i \times \gamma_n$ ($\gamma_n = 0.03$*)

$N_{i,j}$ = 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/mm²) $\rightarrow \Delta\sigma_f C_R C_T = 80$(N/mm²)

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

$m = 3$

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

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

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

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

Preventive method for enhancing fatigue strength

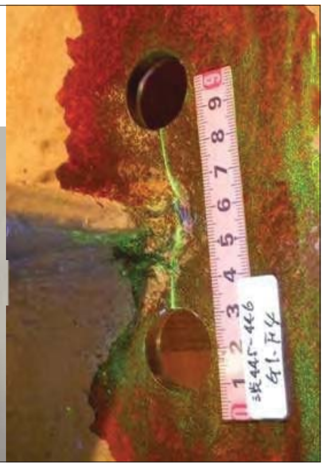
【1】Grinder
- welding (head) shape is made smooth to reduce stress concentration

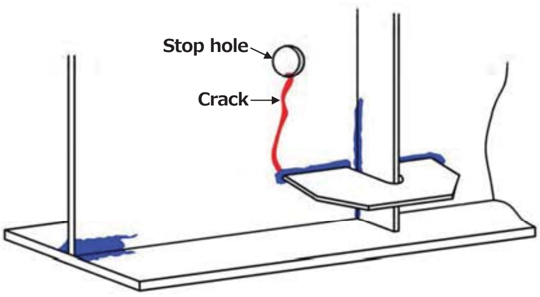
【2】TiG (Tungsten) welding
- good appearance (smooth shape) is obtained
- less possibility of occurrence of welding defects
- take long time for welding work

【3】Hammer peening
- At weld toe, residual stress and stress concentration are reduced by hitting.

Repair methods

Stop hole





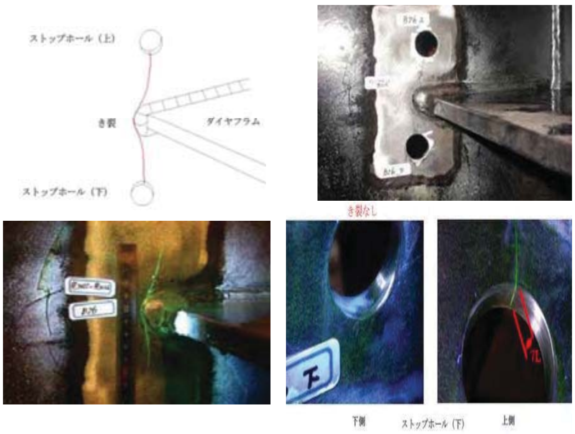


Plate attachment

Appendix

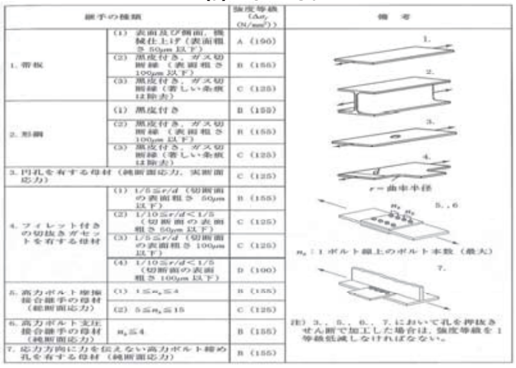
Japanese Spec. Ⅱ Steel Bridge

Fatigue design

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

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

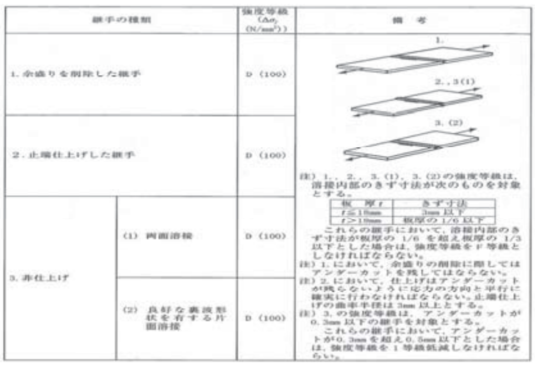
(a) Non-weld



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

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

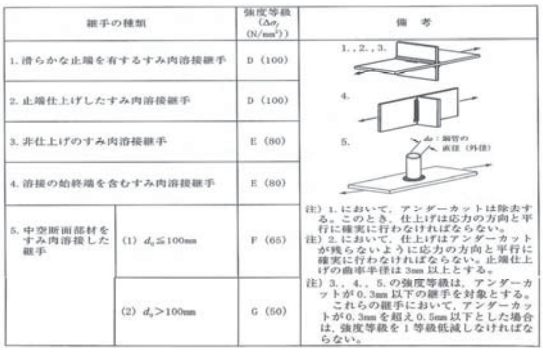
(b) Butt weld



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

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

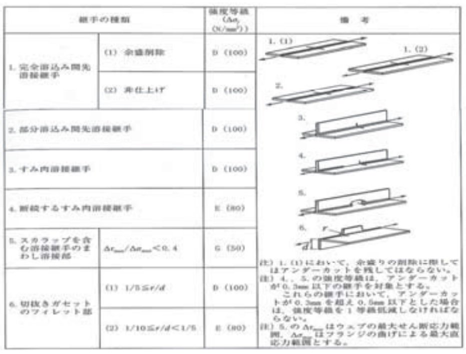
(d) Non-load transfer type cross weld



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

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

(c) Longitudinal weld



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

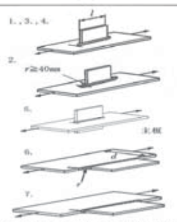
Table-6.3.7 Joint type and strength class under the Vertical stress

(e) Load transfer type cross weld

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

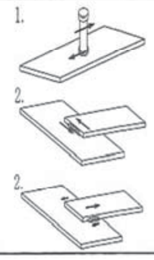
Table-6.3.7 Joint type and strength class under the vertical stress

(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)	
		(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)	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。 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。 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。 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) 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)	

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

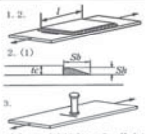
Table-6.3.8 Joint type and strength class under the share stress

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)	

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

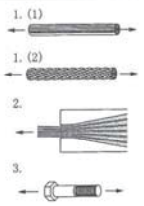
Table-6.3.7 Joint type and strength class under the vertical stress

(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)	
	(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)	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。 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。 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	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）。

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。

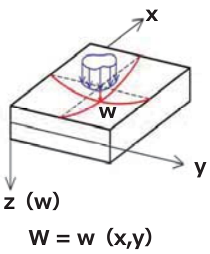
[11-5-1]

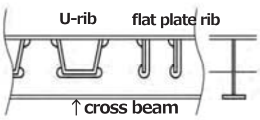
Design of Slabs

Concrete and steel decks

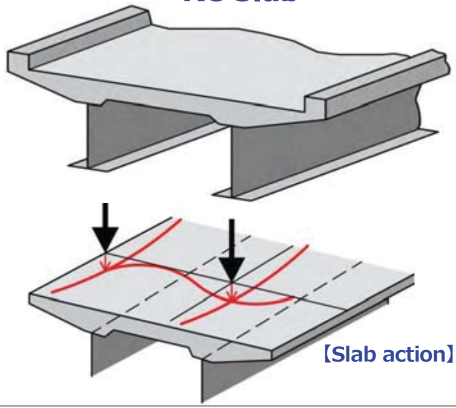
RC deck

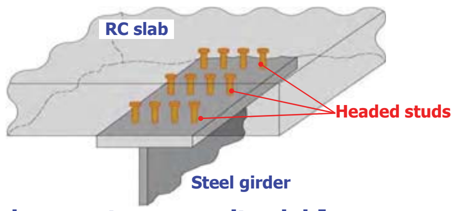
Steel deck



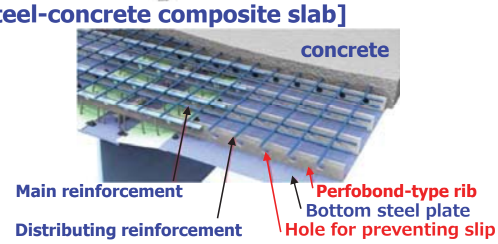


RC Slab

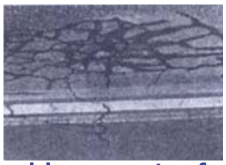




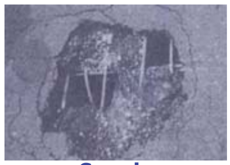
[Steel-concrete composite slab]



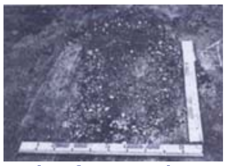
Damage of slabs



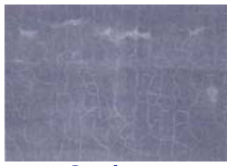
Crack in pavement surface



Cave in



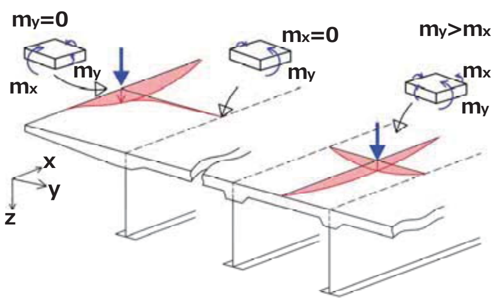
Separation of mortar and aggregate
[upperpart of slab]



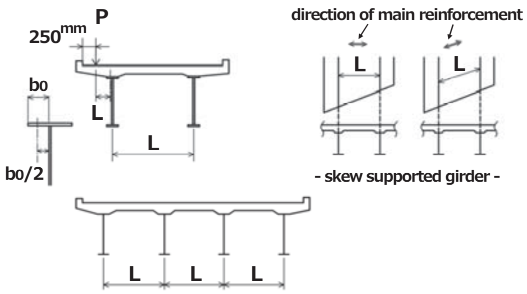
Crack net
[underneath of slab]

Design of RC deck

Deformation of RC slab



Definition of slab span (L)



Coefficient K1 and K2

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

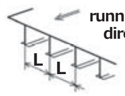
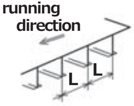
M : $M_0 + \Delta M$ (1 + i)

ΔM : additional moment

i : impact coefficient

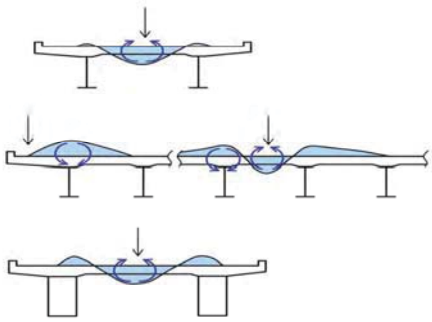
Minimum slab thickness (d) required

d_0 (mm) L : span (m)

			
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) = $k_1 k_2 d_0$

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 $-\frac{wL^2}{8}$; 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 = 100kN

(*) : distributing direction (M=0)

Design of Steel deck plates

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)

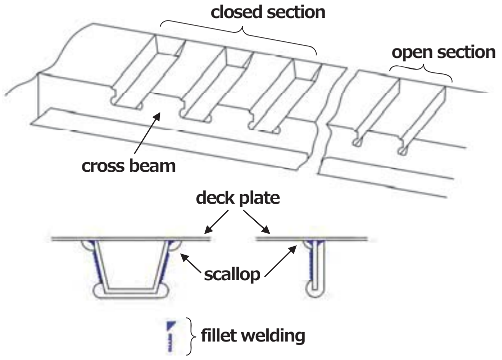
Allowable stress of reinforcement

(N/mm^2)

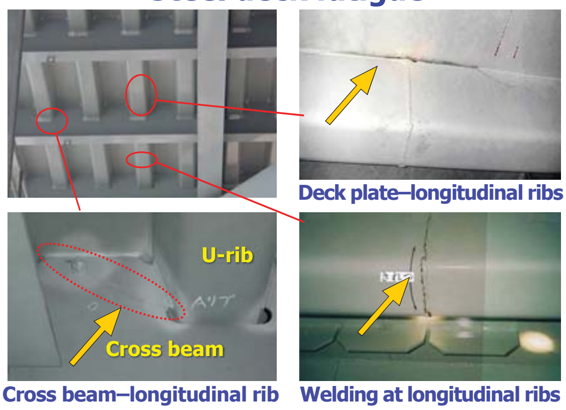
	SD345
tension	140
compression	200

Steel deck

Steel deck plate

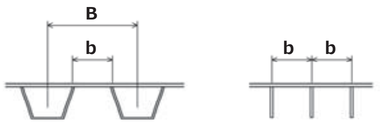


Steel deck fatigue



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

$k = k_0 \quad (L \leqq 4)$

$k = k_0 - (k_0 - 1) \times (L - 4) / 6 \quad (4 < L \leqq 10)$

$k = 1.0 \quad (10 < L)$

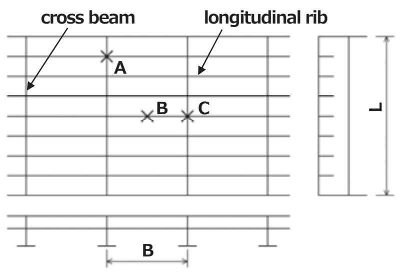
$k_0 = 1.0 \quad (B \leqq 2)$

$k_0 = 1.0 + 0.2 \times (B - 2) \quad (2 < B \leqq 3)$

$k_0 = 1.2 \quad (3 <)$

B : distance of cross beams

Calculation of stress resultants (grid model)

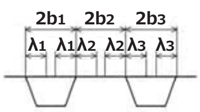


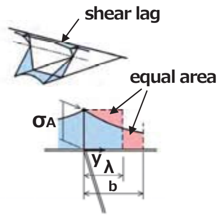
Pts. A,B,C : Observation points

Pts. A,B : for designing longitudinal ribs

Pt. C : for designing cross beams

Effective Width





$\lambda = b \quad (b/L_e \leqq 0.02)$

$\lambda = \left\{1.06 - 3.2\left(\dfrac{b}{L_e}\right) + 4.5\left(\dfrac{b}{L_e}\right)^2\right\} b \quad (0.02 < b/L_e < 0.30)$

$\lambda = 0.15 L_e \quad (0.30 \leqq b/L_e)$

Le : equivalent span length

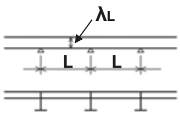
effective width (λ) is introduced to catch (σ_A)

$\lambda \sigma_A = \int_0^b \sigma(y)\, dy$

Equivalent span length（Le）

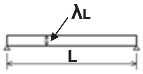
longitudinal ribs

λ_L（Le = 0.6 L）

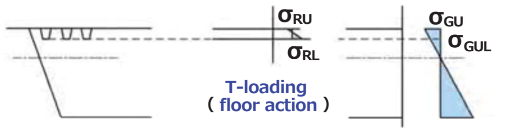


cross beams

λ_L（Le = L）



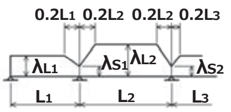
Stress evaluation

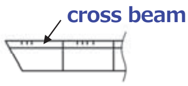


allowable stress (N/mm²)

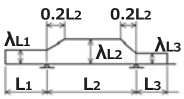
	SM400 SMA400W	SM490	SM490Y SM520 SMA490W	SM570 SMA570W
t ≦ 40	195	260	295	355
40 < t ≦ 75	175	245	275	345
75 < t ≦ 100			265	335

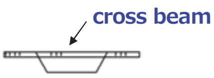
(↑ in case of combined stress check)





λ_{L1}（Le = 0.8 L_1）
λ_{S1}（Le = 0.2（L_1+L_2））
λ_{L2}（Le = 0.6 L_2）
λ_{S2}（Le = 0.2（L_2+L_3））





λ_{L1}（Le = 2 L_1）
λ_{L2}（Le = 0.2（L_1+L_2））
λ_{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 ,

$$\frac{\partial q_x}{\partial x} + \frac{\partial q_y}{\partial y} + p_z = 0 \qquad (1)$$

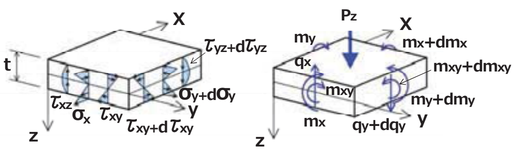
$$\frac{\partial m_x}{\partial x} + \frac{\partial m_{xy}}{\partial y} - q_x = 0 \qquad (2)$$

$$\frac{\partial m_{xy}}{\partial x} + \frac{\partial m_y}{\partial y} - q_y = 0 \qquad (3)$$

$$\frac{\partial}{\partial y}(\text{eq. (2)}) \;\&\; \frac{\partial}{\partial x}(\text{eq. (3)}) \xrightarrow{\text{substitute}} \text{eq. (1)}$$

$$\frac{\partial^2 m_x}{\partial x^2} + 2\frac{\partial^2 m_{xy}}{\partial x \partial y} + \frac{\partial^2 m_y}{\partial y^2} = -p_z \qquad (4)$$

Stress and stress resultants



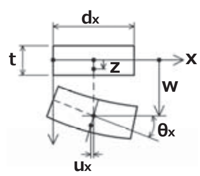
$$m_x = \int \sigma_x z dA, \quad m_y = \int \sigma_y z dA$$

$$m_{xy} = \int \tau_{xy} z dA$$

$$q_x = \int \tau_{xz} dA, \quad q_y = \int \tau_{yz} dA$$

$$\left(\int dA \rightarrow \int_{-t/2}^{t/2} dA \right)$$

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
ν : Poisson's ratio

$$\sigma x = -\frac{Ez}{1-\nu^2}\,(w,_{xx} + \nu w,_{yy})$$

$$\sigma y = -\frac{Ez}{1-\nu^2}\,(\nu w,_{xx} + w,_{yy})$$

$$\tau xy = -2G_z\, w,_{xy}$$

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

$$\sigma_x = \frac{M_x}{I} z$$

$$\sigma_y = \frac{M_y}{I} z$$

$$\tau_{xy} = \frac{M_{xy}}{I} z$$

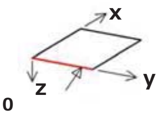
$$I = \frac{t^3}{12}$$

$$\left.\begin{aligned} m_x &= \int \sigma_x z dA = -B\,(w,_{xx} + \nu w,_{yy}) \\ m_y &= \int \sigma_y z dA = -B\,(\nu w,_{xx} + w,_{yy}) \\ m_{xy} &= \int \tau_{xy} z dA = -\frac{Gt^3}{6} w,_{xy} = -(1-\nu) B\, w,_{xy} \\ q_x &= \int \tau_{xz} dA = -B\,(w,_{xxx} + w,_{xyy}) \end{aligned}\right\} (5)$$

$$q_y = \int \tau_{yz} dA = -B\,(w,_{yyy} + w,_{xxy})$$

$$B = \frac{Et^3}{12(1-\nu^2)}$$

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

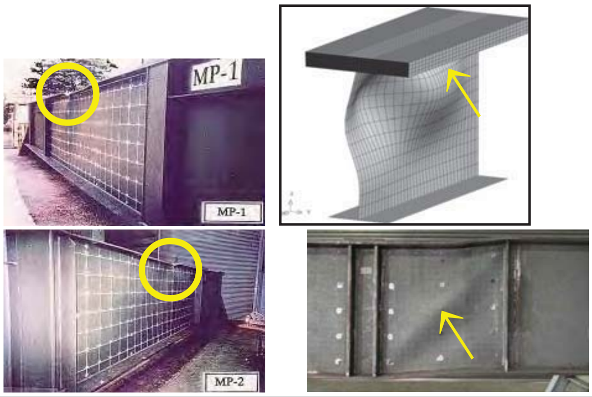
$m_x = \overline{q}_x = 0$

$(\overline{q}_x = q_x + m_{xy},_y \; , \; \overline{q}_y = q_y + m_{xy},_x)$

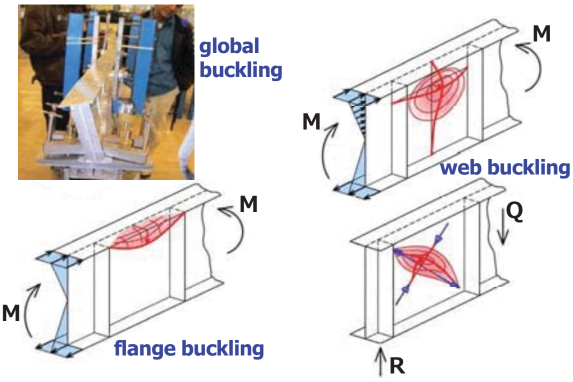
[11-5-2]

Design of Girder Bridges

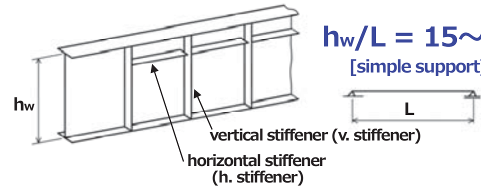
Local buckling of compressed plate and web



Buckling of girder and plates



Minimum web thickness

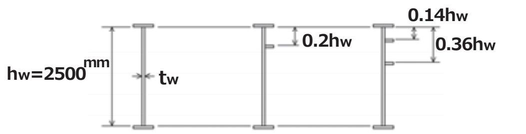


$h_w/L = 15\sim20$

[simple support]

	SS400 SM400 SMA400W	SM490	SM490Y SM520 SMA490YW	SM570 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})

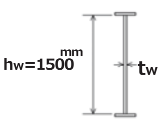
Thickness of web without stiffeners
becomes considerably large.

→ To avoid thick web,
stiffeners (H & V) are employed
to prevent buckling.

h_w without vertical stiffener

	SS400 SM400 SMA400W	SM490	SM490Y SM520 SMA490W	SM570 SMA570W
min.hw	70tw	60tw	57tw	50tw

ex. (SM400) (SM490Y)



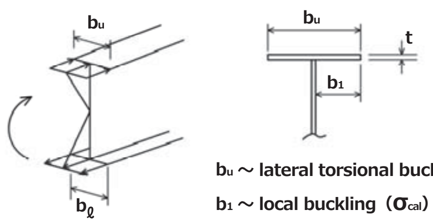
$t_w > \frac{h_w}{70} = 21.4^{mm} \longrightarrow \underline{22^{mm}}$

more than 22^{mm}, no v. stiffener allowed

$t_w > \frac{h_w}{57} = 26.3^{mm} \longrightarrow \underline{27^{mm}}$

more than 27^{mm}, no v. stiffener allowed

Flange in compression



$b_u \sim$ lateral torsional buckling (σ_{ba})

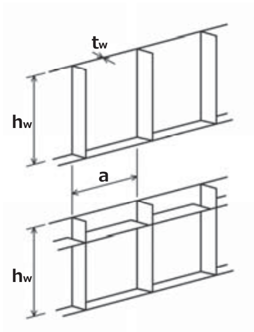
$b_1 \sim$ local buckling (σ_{cal})

$\underline{\sigma_{ult.} = \min.\{\sigma_{ba}, \sigma_{cal}\}}$ strength of beams

In tension

$t > \frac{b_1}{16}$

Design of web



(1) Horizontal stiffeners

See PPT No. 5

(2) Vertical stiffeners

$$\left(\frac{\sigma}{\sigma_E}\right)^2 + \left(\frac{\tau}{\tau_E}\right)^2 \leqq \frac{1}{\gamma^2}$$

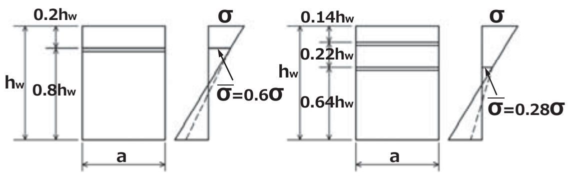
$\sigma_E = k_\sigma(=23.9) \cdot \sigma_{E0}$

$\tau_E = k_\tau \cdot \sigma_{E0}$

$\gamma = 1.25$

verification formula

In case of the web with stiffener



one h. stiffener

$B = 0.8h_w$

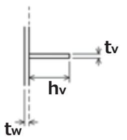
$\bar{\sigma} = 0.6\sigma$

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



$I_v = \dfrac{t_v h_v^3}{3}$

$I_v > \dfrac{h_w t_w^3}{11} \cdot \gamma_{v,req.}$

$\gamma_{v,req.} = 8.0 \left(\dfrac{h_w}{a}\right)^2$

Check of shear strength of web

[ex. In case of no horizontal stiffener]

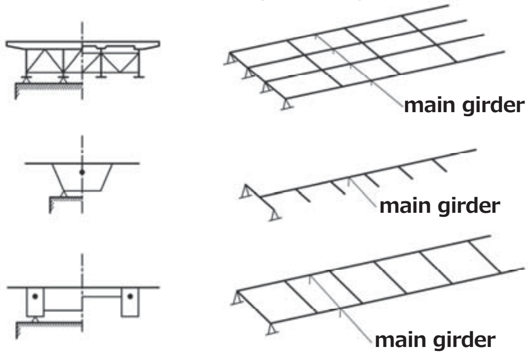
$$\left(\frac{h_w}{100t_w}\right)^4 \left[\left(\frac{\sigma}{345}\right)^2 + \left\{\frac{\tau}{77+58(h_w/a)^2}\right\}^2\right] \leqq 1.0 \quad (a/h_w > 1.0)$$

$$\left(\frac{h_w}{100t_w}\right)^4 \left[\left(\frac{\sigma}{345}\right)^2 + \left\{\frac{\tau}{58+77(h_w/a)^2}\right\}^2\right] \leqq 1.0 \quad (a/h_w \leqq 1.0)$$

must be satisfied

Stress resultants (M, Q)

are calculated using following model



Safety check

(1a) normal stress (σ_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
σ_{ba} : allowable flexural compressive stress
σ_{cal} : allowable local buckling stress

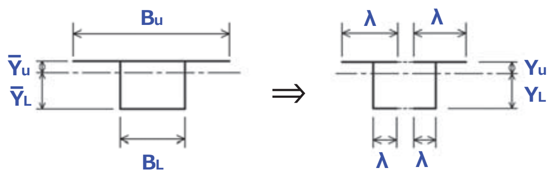
(1b) normal stress (σb) of box girder

$\sigma_b = (M/I) \cdot y \leqq \sigma_{ba}$

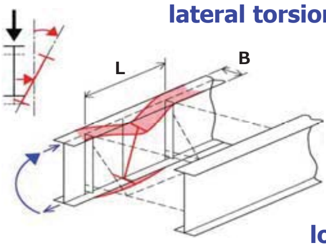
$\sigma_{ba} = \underline{\sigma_y/1.7}$

↑ no lateral torsional buckling

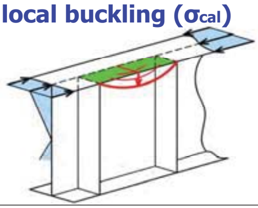
I : geometrical moment of inertia
(calculate using effective width λ)



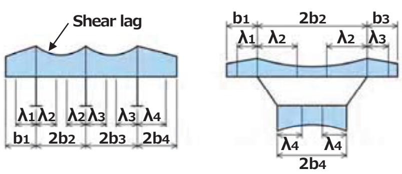
lateral torsional buckling (σ_{ba})

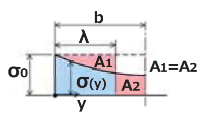


local buckling (σ_{cal})



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 (σ_0)

$\lambda = b \quad \left(\frac{b}{L_e} \leqq 0.05\right)$

$\lambda = \left\{1.1 - 2\left(\frac{b}{L_e}\right)\right\} b \quad \left(0.05 < \frac{b}{L_e} < 0.30\right)$ — (1)

$\lambda = 0.15 L_e \quad \left(0.30 \leqq \frac{b}{L_e}\right)$

- parabolic -

$\lambda = b \quad \left(\frac{b}{L_e} \leqq 0.02\right)$

$\lambda = \left\{1.06 - 3.2\left(\frac{b}{L_e}\right) + 4.5\left(\frac{b}{L_e}\right)^2\right\} b \quad \left(0.02 < \frac{b}{L_e} < 0.30\right)$ — (2)

$\lambda = 0.15 L_e \quad \left(0.30 \leqq \frac{b}{L_e}\right)$

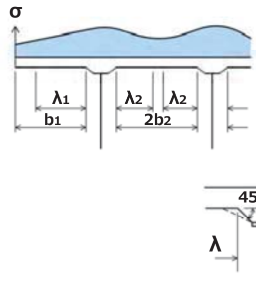
- straight -

⇑

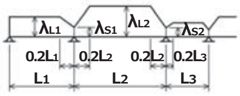
moment distribution

L_e : equivalent span length

Effective width of concrete slab



λ_L ($L_e = L$) by eq. (1)

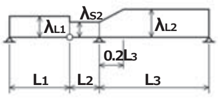


λ_{L1} ($L_e = 0.8L$) by eq. (1)

λ_{L2} ($L_e = 0.6L$) 〃

λ_{S1} ($L_e = 0.2\,(L_1+L_2)$) by eq. (2)

λ_{S2} ($L_e = 0.2\,(L_2+L_3)$) 〃



λ_{L1} ($L_e = L_1$) by eq. (1)

λ_{L2} ($L_e = 0.8L_3$) by eq. (1)

λ_{S2} ($L_e = 2L_2$) by eq. (2)

(2) shear stress (τ_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

τ_a : allowable shear stress

τ_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 (σ_w, τ_s, τ_w) in torsion

in case of I-section, (σ_w, τ_s, τ_w) can be neglected.

in case of box-section, (σ_w, τ_w) can be neglected.

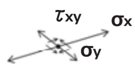
σ_w : warping stress

τ_s : St.Venant shear stress (pure torsion)

τ_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 (σ_e)

10%up

$$\sigma_e = \sqrt{\sigma_x^2 - \sigma_x\sigma_y + \sigma_y^2 + 3\tau_{xy}^2} < \underline{1.1\sigma_a}$$

$$\left(\frac{\sigma_x}{\sigma_a}\right)^2 - \left(\frac{\sigma_x}{\sigma_a}\right)\left(\frac{\sigma_y}{\sigma_a}\right) + \left(\frac{\sigma_y}{\sigma_a}\right)^2 + 3\left(\frac{\tau_{xy}}{\sigma_a}\right)^2 < 1.2$$

$\tau_y = \tilde{\sigma}_y / \sqrt{3} \rightarrow \sigma_a = \sqrt{3}\,\tau_a$ ($\tilde{\sigma}_y$: 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 (σ_b, τ_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

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

$\sigma < \sigma_a$

$\tau < \tau_a$

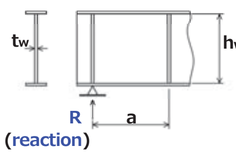
$\sigma = \sigma_b + \sigma_w$

$\tau = \tau_b + \tau_s + \tau_w$

* take into account that loading conditions for $\sigma_{max.}$, τ_{max} are different

Design of stiffeners

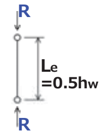
Vertical stiffener at support

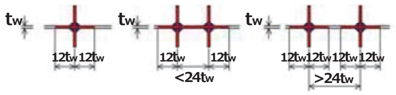


$\sigma = \dfrac{R}{A_{eff}} < \sigma_{ca}$

$\sigma_{ca} = \sigma_{cr}/1.7$

(σ_{cr} : column buckling strength)

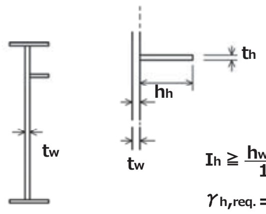




Effective area ($A_{eff} < 1.7A_{stiffener}$)

$A_{eff} = A_{web} + A_{stiffener}$

Horizontal stiffeners



$I_h = \dfrac{t_h h_h^3}{3}$

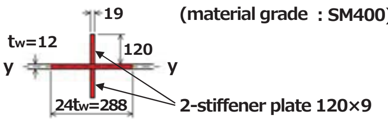
$I_h \geqq \dfrac{h_w t_w^3}{11} \cdot \gamma_{h,req.}$

$\gamma_{h,req.} = 30\ (a/h_w)$

a : distance of vertical stiffeners

h_w : depth of the web

Example



(material grade : SM400)

$A_{web} = 288 \times 12 = 34.^{56}$

$A_{rib} = 2 \times 120 \times 19 = 45.^{60}$

$80.16^{(cm^2)} < 1.7A_{rib} = 82.08^{(cm^2)}$

$I_y = 2{,}534^{cm^4}$, $r_y = 5.62^{cm}$

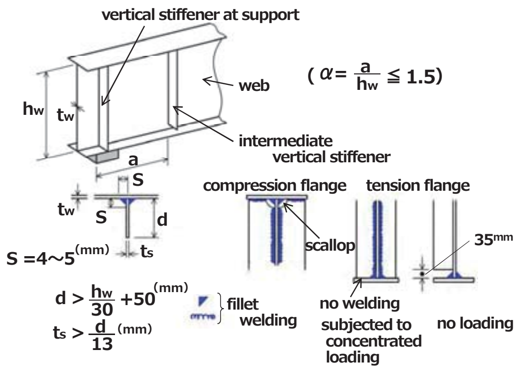
$L_e/r_y = 0.5 \times 160^{(cm)}/5.62 = 14.2$ (h_w : 1600^{mm})

$R = 879.4$ (kN)

$\sigma_c = \dfrac{879.4 \times 10^3}{80.16 \times 10^2} = 109.0^{(N/mm^2)} < \sigma_{ca}$ (ok)

($L_e/r_y < 18 \rightarrow \sigma_{ca} = 140\ ^{N}/_{mm^2}$)

Intermediate vertical stiffeners

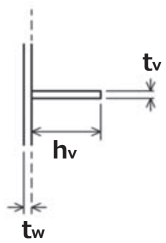


($\alpha = \dfrac{a}{h_w} \leqq 1.5$)

$S = 4 \sim 5^{(mm)}$

$d > \dfrac{h_w}{30} + 50^{(mm)}$

$t_s > \dfrac{d}{13}^{(mm)}$



$I_v = \dfrac{t_v h_v^3}{3}$

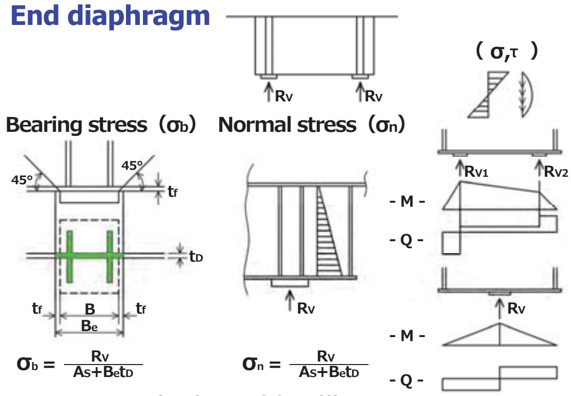
$I_v > \dfrac{h_w t_w^3}{11} \cdot \gamma_{v,req.}$

$\gamma_{v,req.} = 8.0\ \left(\dfrac{h_w}{a}\right)^2$

a : distance of vertical stiffeners

h_w : depth of the web

End diaphragm



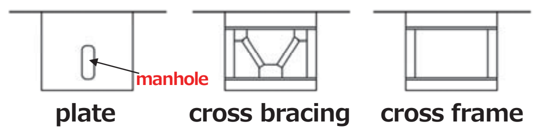
$\sigma_b = \dfrac{R_v}{A_s + B_e t_D}$ $\sigma_n = \dfrac{R_v}{A_s + B_e t_D}$

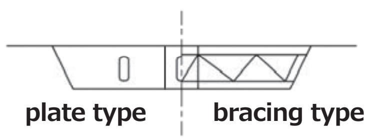
As : cross-sectional area of the stiffeners

Design of support diaphragms & intermediate diaphragms

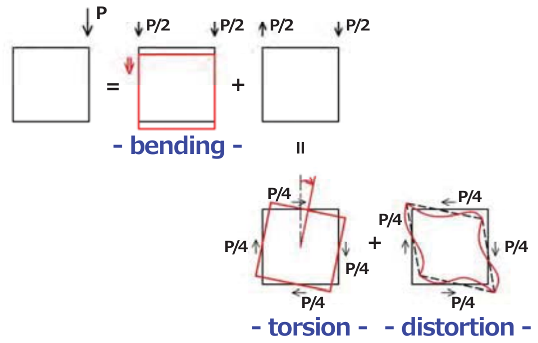
Intermediate diaphragms

(to prevent distortion)

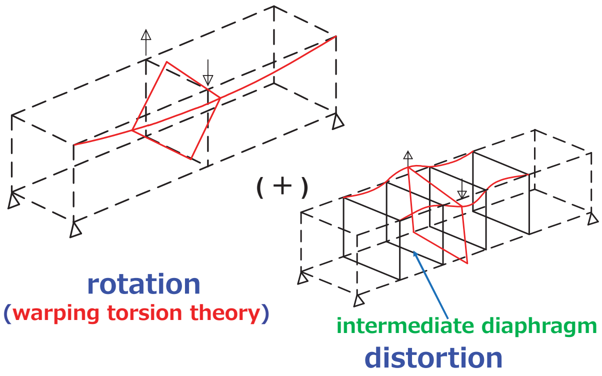


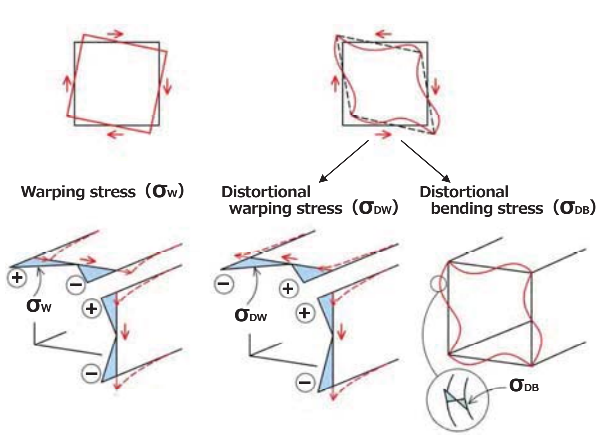


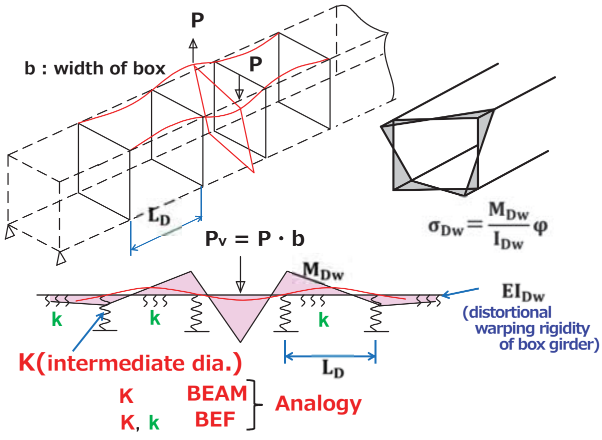
Torsion and distortion under eccentric loading



Torsion and distortion



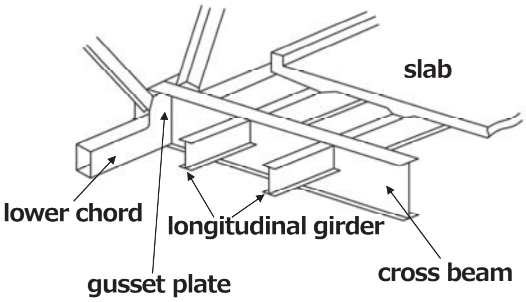




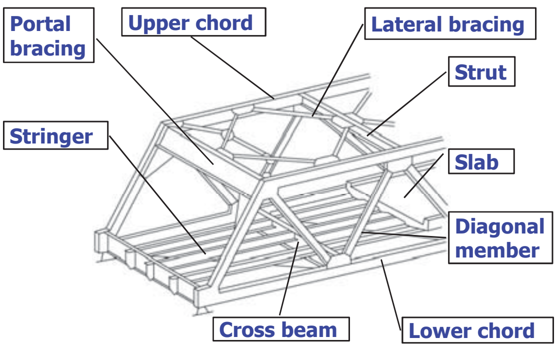
[11-5-3]

Design of Truss Bridges

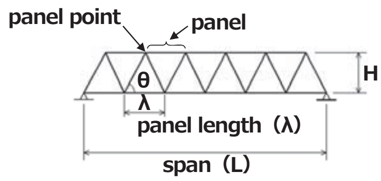
Members



Name of Truss Members



Preliminary design



$H/L = 1/6 \sim 1/8$

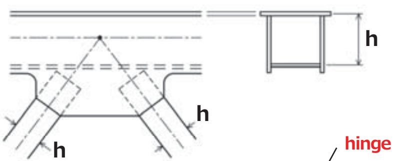
$\theta = 40 \sim 60°$

$\lambda = 6 \sim 10^{m}$

$\delta^{*} < L / 600$

＊deflection by live load（excluding impact）

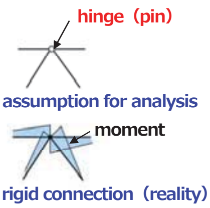
h/λ for pin structure



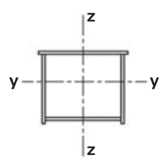
$$\frac{h}{\lambda} = \frac{1}{15} \sim \frac{1}{20}$$

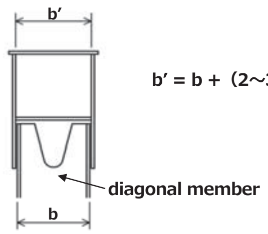
λ : panel length

to reduce moment

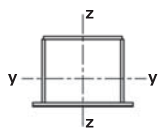


If (h/λ) is large, rigid frame model is used.



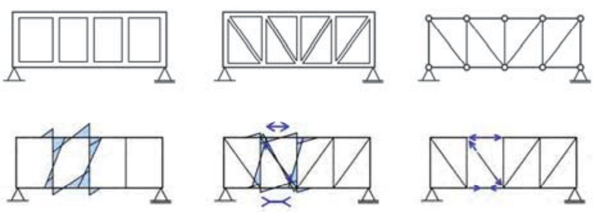


$b' = b +（2\sim3）^{mm}$



・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)

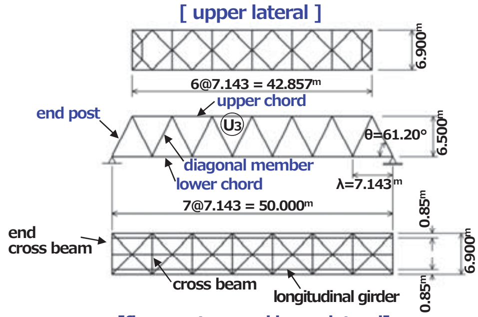
- Rigid frame structure -

Axial force (N)

- Truss -

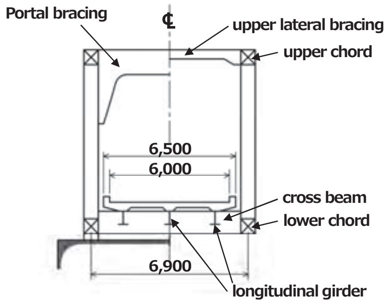
Frame model

[upper lateral]

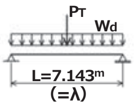


[floor system and lower lateral]

Cross section



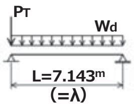
Maximum bending moment and shear force



$$M_d = \frac{W_d L^2}{8}$$

$$M_{L+i} = \frac{P_T L}{4}(1+i)$$

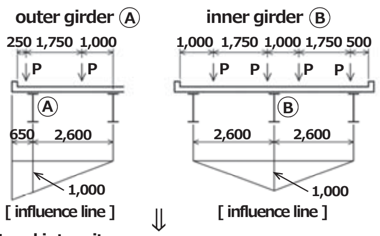
$$i = \frac{20}{50+L} = \frac{20}{50+7.143} = 0.35$$



$$Q_d = \frac{W_d L}{2}$$

$$Q_{L+i} = P_T(1+i)$$

Design of longitudinal girder



⇓

Load intensity

W_d, Ⓐ
W_d, Ⓑ } dead load

P_T, Ⓐ
P_T, Ⓑ } live load

Stress check

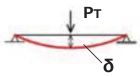
$$\sigma = \frac{(M_d + M_{L+i})}{I} y < \sigma_a (= \sigma_y / 1.7)$$

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)

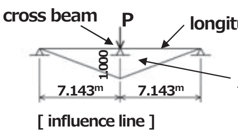
Deflection check



$$\delta < \frac{L}{2{,}000} \; (L < 10^m)$$

Design of cross beam

Span of cross beam is assumed to be distance between chord member



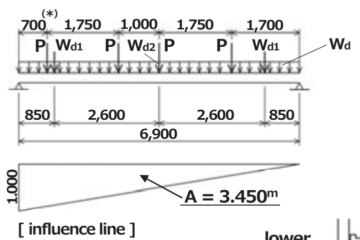
A = 7.143m
area of influence line

$W_{d,1}$ (outer girder) = W_d, Ⓐ × A

$W_{d,2}$ (inner girder) = W_d, Ⓑ × A

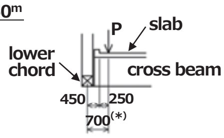
W_d : Self weight of cross beam

Shear force

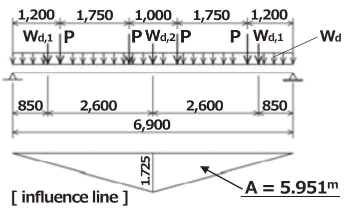


⇓

Q_d, Q_{L+i}



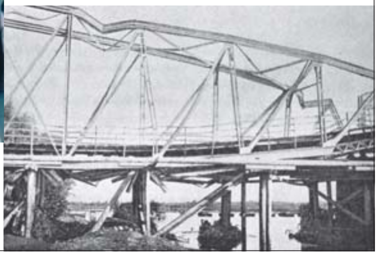
Maximum bending moment



⇓

M_d, M_{L+i}

Design of chord and web members



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

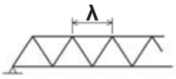
** effective buckling length (in compression)
panel length (in tension)

*** members in cross or lateral bracing

Effective buckling length (Le)

[Chord member]

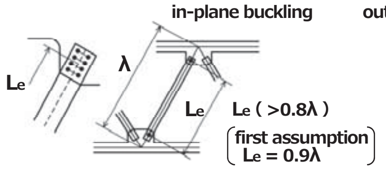
in-plane & out-of-plane buckling $L_e = \lambda$ (panel length)



[Web member]

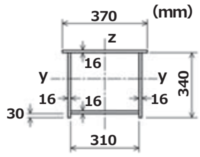
in-plane buckling

out-of-plane buckling $L_e = \lambda$



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

$$L_e/r = 714.3 / \sqrt{37{,}151/217.6} = 54.6$$

$$\sigma_{ca} = 140 - 0.82\,(54.6 - 18) = 110^{N/mm^2}$$

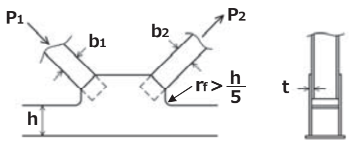
$$\sigma = \frac{2370.1\times10^3}{217.6\times10^2} = 108.9^{N/mm^2} < \sigma_{ca} \text{ (ok)}$$

Design of gusset plate

t (plate thickness, mm) $> 2 \times \frac{P}{b}$

P : maximum force of end post or web member (kN)

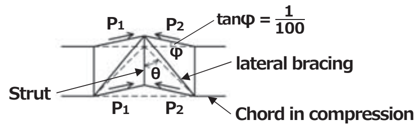
b : width of end post or web member (mm)



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 : $\frac{P_1 + P_2}{100}\sec\theta$



Design of lateral bracing members

buckling length

$L_e = 0.9\lambda$ (λ : panel length)

(*from conservative viewpoint, $L_e = \lambda$)

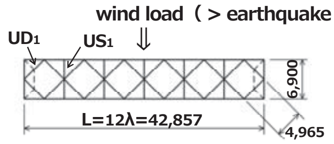
max. allowable slenderness ratio

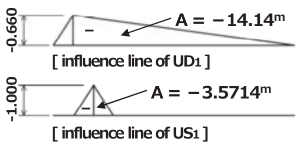
in compression $L_e/r < 150$

in tension $\lambda/r < 240$

Design of upper lateral bracing

wind load (> earthquake load)



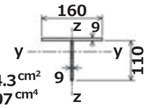


$N = \pm 3.80 \times 14.14$
↑wind load (kN/m)
$= \pm 53.73$ (kN)

$N = -3.80 \times 3.5714$
$= -13.57$ (kN)

UD1

— in compression —



$A = 24.3\ cm^2$

$I_y = 307\ cm^4$

$I_z = 307\ cm^4$

check of slenderness ratio

$L_e/r_y = 496.5/\sqrt{307/24.3} = 140 < 150$

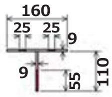
$\sigma_{cay} = \frac{1{,}200{,}000}{6{,}700+140^2} = 45.6\ (N/mm^2)$

$\sigma'_{cay} = 45.6\ (0.5+\frac{140}{1{,}000}) = 29.1\ (N/mm^2)$

$\sigma_c = \frac{53.73\times10^3}{24.3\times10^2} = 22.1\ (N/mm^2) < \underset{\uparrow}{1.2}\sigma'_{cay} = 34.9\ (N/mm^2)$ (ok!!)

extra coefficient

— in tension —



$\sigma_t = \frac{53.73\times10^3}{1{,}485} = 36.2\ (N/mm^2) < 1.2\sigma_{ta}$

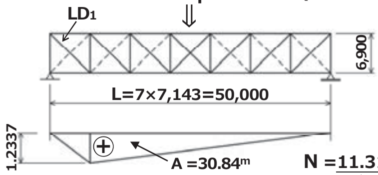
$= 1.2\times140 = 168\ (N/mm^2)$

$A_n = (160-2\times25)\times9+\frac{110}{2}\times9 = 1{,}485\ cm^2$

A_n : net cross sectional area

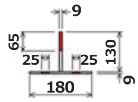
Design of lower lateral bracing

earthquake load (> wind load)



$N = \underset{\uparrow}{11.31}\times30.84 = 348.8\ (kN)$

earthquake load (kN/m)



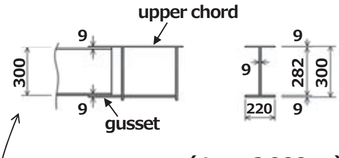
$\sigma_t = \frac{348.8\times10^3}{1{,}755} = 198.8\ (N/mm^2) < 1.5\sigma_{ta}$

$= \underset{\uparrow}{1.5}\times140 = 210\ (N/mm^2)$

extra coefficient

$A_n = (180-2\times25)\times9+65\times9 = 1{,}755\ mm^2$

US1



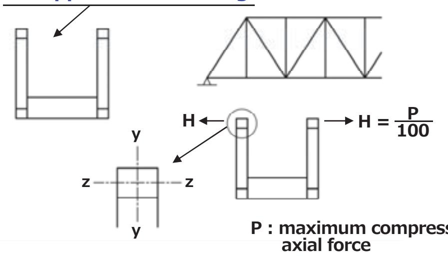
$\sigma_c < \sigma_a$ ($L_e = 6{,}900\ mm$)

∗height of strut in lower deck type bridge shall have the same height of the chord

Pony Truss Bridges

Design of Pony truss

no upper lateral bracing



P : maximum compressive axial force

$r_y \geq 1.5 r_x$

Safety shall be checked under H loading