Structural Design Procedure of Steel Structure in JAPAN

-Seismic design and My recent research activities-

Atsushi SATO, Dr. of Eng.
Nagoya Institute of Technology, Associate. Professor

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ノルウェー
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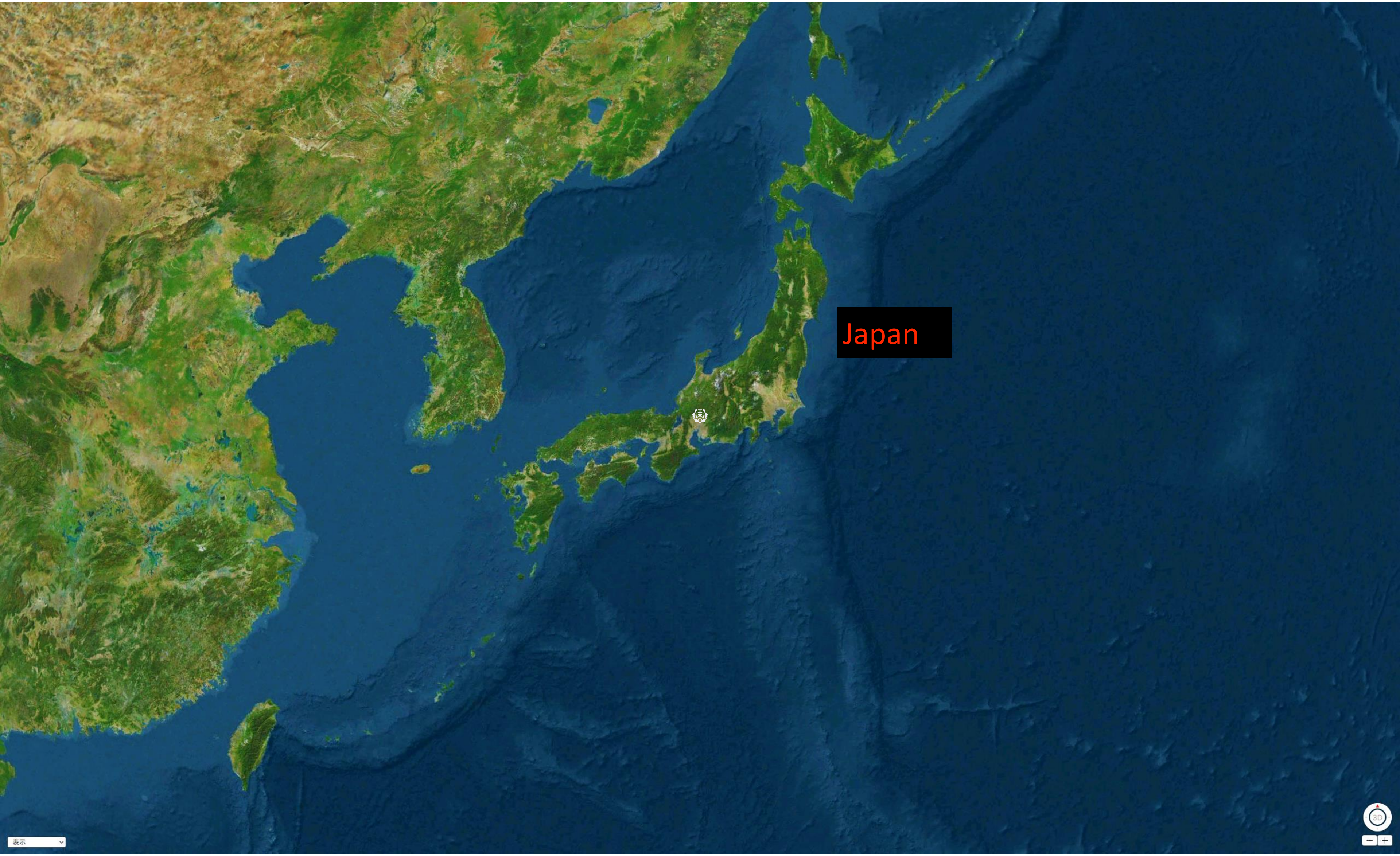
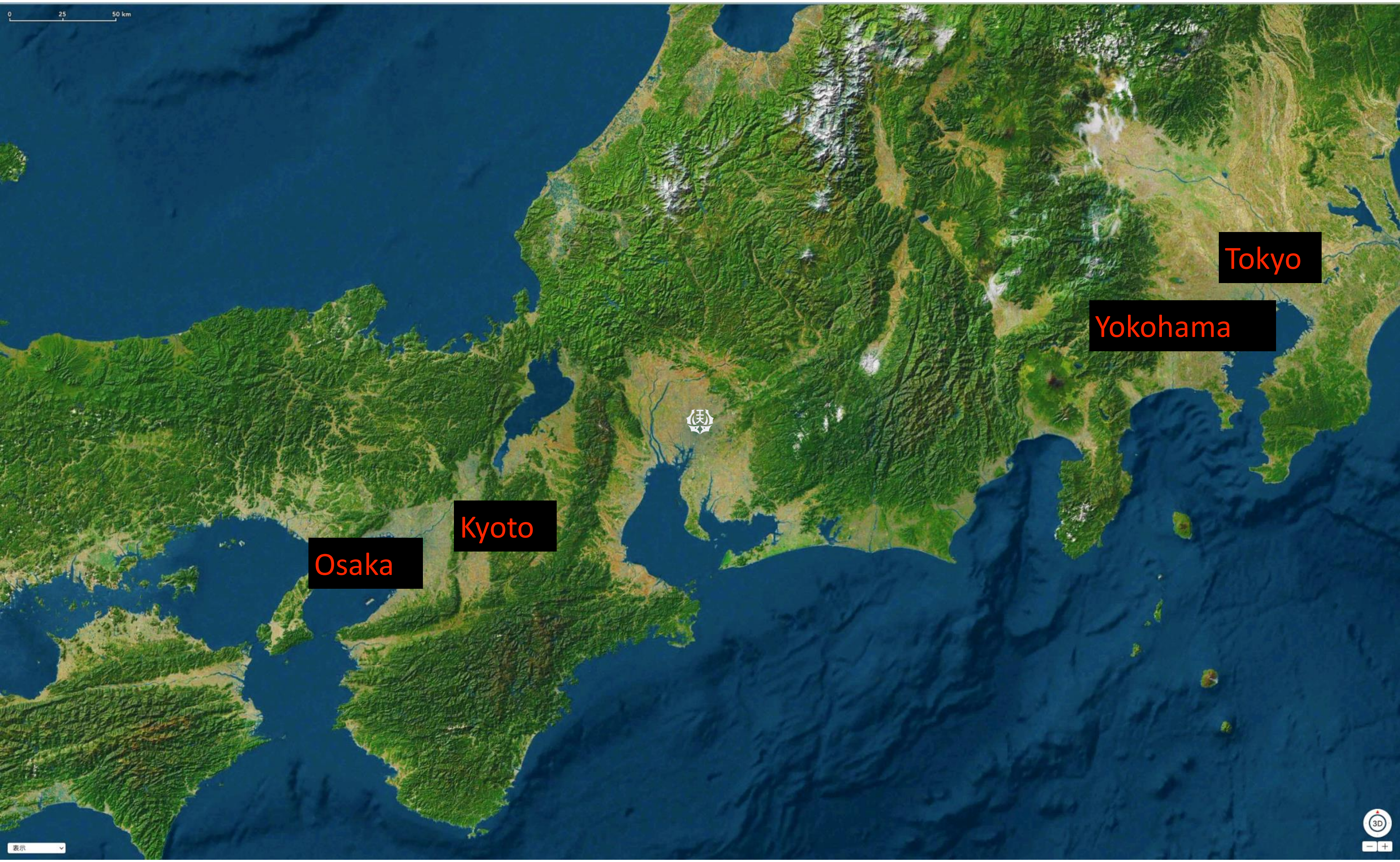

Japan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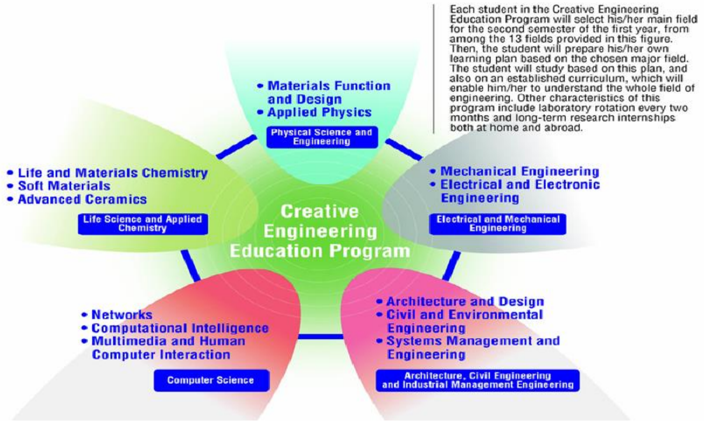
Nagoya City


Nagoya Station
Nagoya Institute of Technology

Atsushi SATO, Dr. of Eng.

- Nagoya Institute of Technology
  - Department of Architecture, Civil Engineering and Industrial Management Engineering
    - Associate Professor
    - Structural Engineer
    - Professional Architect (National License)

- Background
  - Assistant Professor (NITech)
  - Visiting Scholar (University of California, San Diego)
  - Assistant Professor (Kyoto University)
  - Associate Professor (NITech)
  - Visiting Professor, ČVUT



Department *of Architecture, Design, Civil Engineering and Industrial Management Engineering*

Architecture and Design	*Civil and Environmental Engineering*	*Systems Management and Engineering*

Structural Lab. In NITech (Archi.)

Structural Design Procedure of Steel Structure in JAPAN

-Seismic design and My recent research activates-

Topics -Steel Structure-

- Design Procedure (General)
  - History and Concept
- Design of Beam-to-Column Connection
- Design of Column
- Recent Research Topics in my Group
  - Column
  - Beam-to-Column Connection

Topics -Steel Structure-

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  - Column
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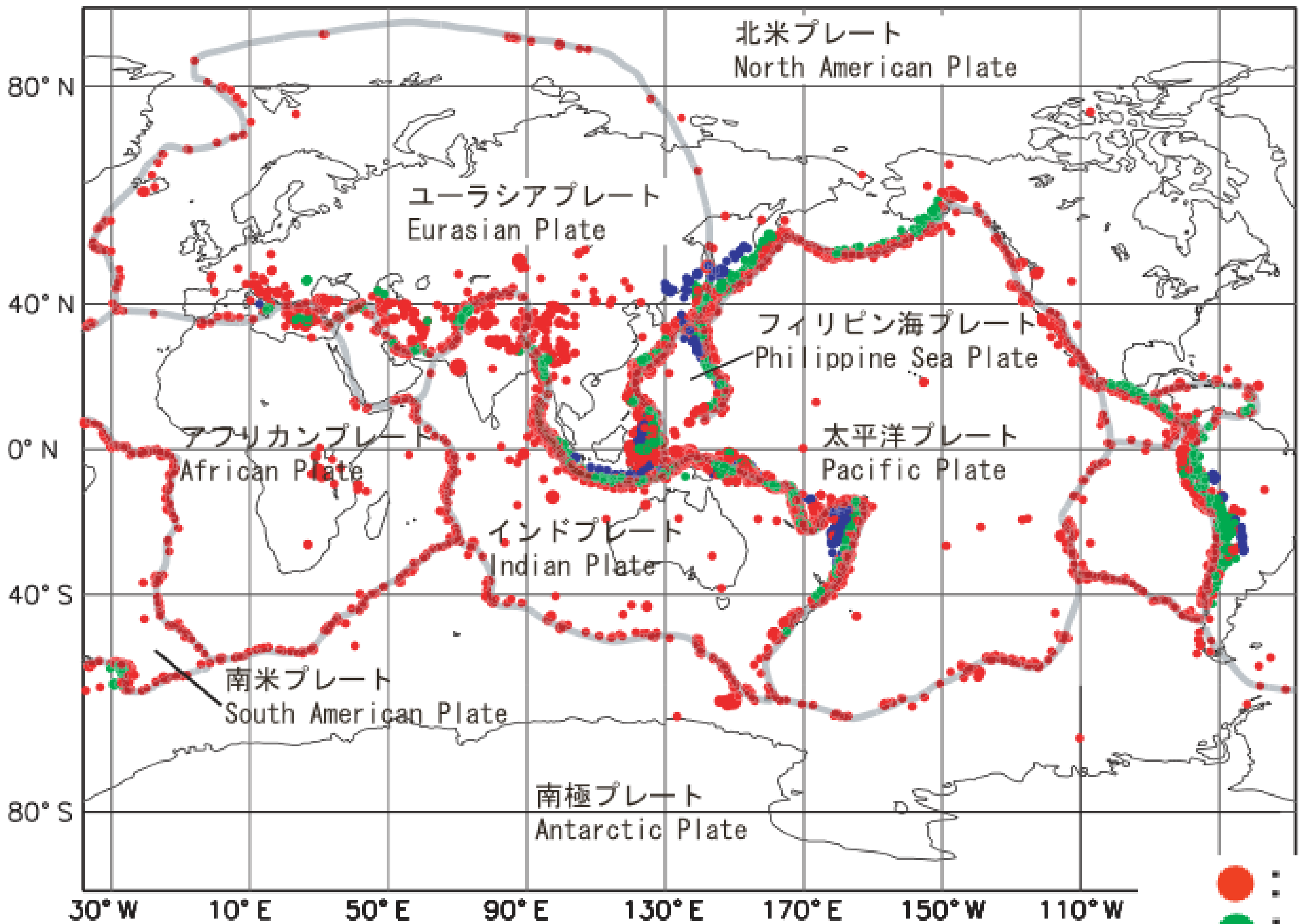
Distribution of Hypocenters



Depth

- : 0～ 60km
- : 60～300km
- : 300～700km

1996～2005, Magnitude ≧ 5.0

Reference: Disaster Management in Japan, Cabinet Office, Government of Japan

History of Disasters and Codes

Year	Disaster or upgrade	Deaths and Missing
1923	**Great Kanto Earthquake(M7.9)**	**about 105 000**
1924	Upgrade in Rules Seismic design became mandatory (0.1)	
1948	**Fukui Earthquake(M7.1)**	**3 769**
1950	Promulgation of the Building Standard of Law Seismic load (0.2), Seismic design for timber structure	
1968	**Tokachi-oki Earthquake (M7.9)**	**52**
1971	Upgrade in Rules of BSL Rules for RC structure became more strict	
1978	**Miyagi-ken-oki Earthquake (M7.4)**	**28**
1981	Upgrade in Rules of BSL Equivalent lateral force procedure was introduced	

History of Disasters and Codes

Year	Disaster	Deaths and Missing
1995	**Great Hanshin-Awaji Earthquake (M7.3)**	**6437**
2000	Additional design procedure was included in BSL	
	Promulgation of " The Calculation Method of Response and Limit Strength "	
2005	Additional design procedure was included in Notification	
	Promulgation of "Energy Balance Based Seismic Resistance Design procedure "	

General (Structural Design)

Ministry of Land Infrastructure, Transport and Tourism (MLIT)

- **Building Standard of Law in JAPAN (BSL)**
  - **Notification** (similar to Law)

Architectural Institute of Japan (AIJ)

- Design Standard for Steel Structures -Based on Allowable Stress Concept-
- Recommendation for Limit State Design of Steel Structures
- Recommendations for the Plastic Design of Steel Structures
- Recommendation for Design of Connections in Steel Structures
- Recommendations for Stability Design of Steel Structures

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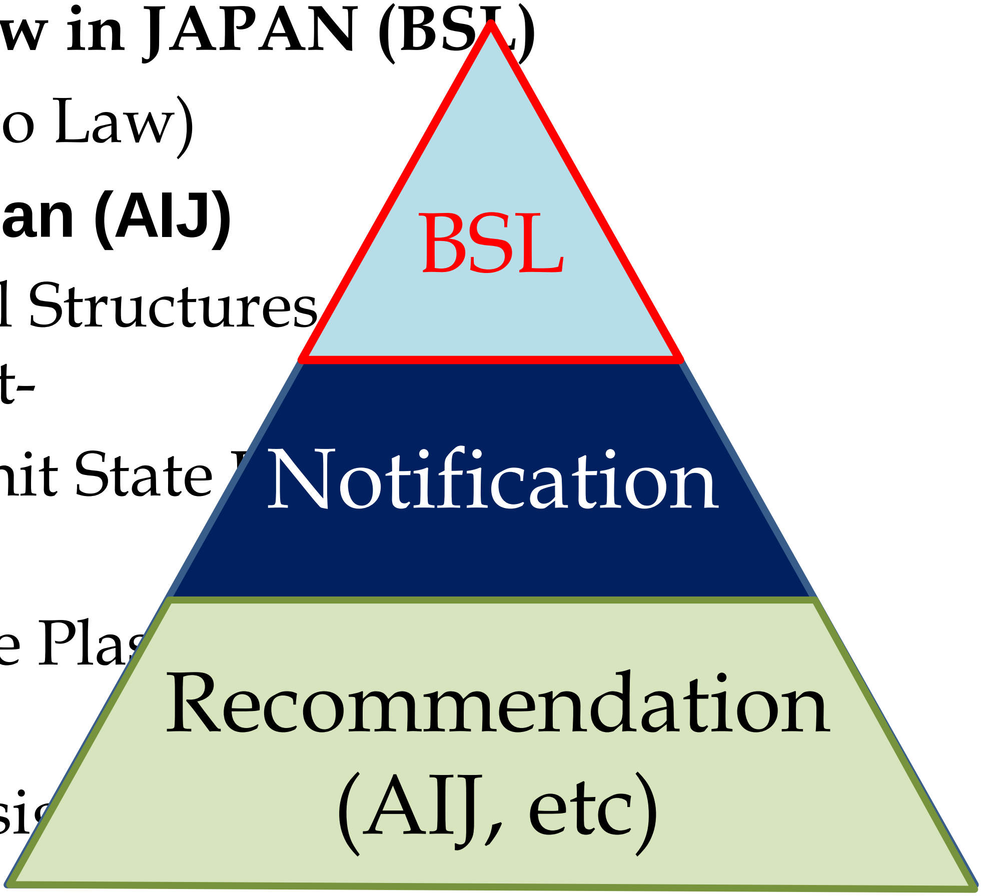


AIJ publications

Allowable Stress Concept

Plastic Design Concept

LRFD Concept

AIJ publications

Joints (Connections)

Stability

General (Structural Design)

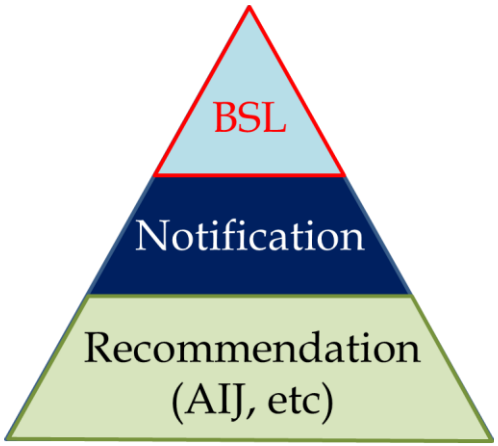
Ministry of Land Infrastructure, Transport and Tourism (MLIT)

- Building Standard of Law in JAPAN (BSL)
  - Notification (similar to Law)

-**Concept** of Design

-Load (Action)

-Resistance (allowable stress)



If needed information are not provided in the Law or Notifications, structure designer will use the Recommendations published by AIJ.

AIJ Recommendations are often referred to compute the Resistance or Limitation for ULS.

General (Structural Design)

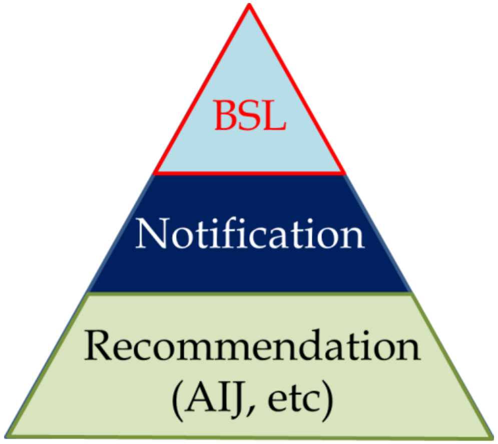
Architectural Institute of Japan (AIJ)

- Design Standard for Steel Structures -Based on Allowable Stress Concept-
- Recommendation for Limit State Design of Steel Structures (LSD, LRFD concept)
- Recommendations for the Plastic Design of Steel Structures (PD, Plastic design concept)
- Recommendation for Design of Connections in Steel Structures (Joints)
- Recommendations for Stability Design of Steel Structures

Resistance (ULS, and SLS)

Introduction of LRFD concept (not used in practice)

Building Standard of Law (BSL)



Building Standard of Law in Japan (BSL)

- Building height greater than 60m

→ **Nonlinear dynamic response time-history analysis** should be conducted. Design process should get an endorsement from the scientific committee.

- Height less than or equal to 60m

→ Standard Procedure can be used.

"Equivalent Lateral Force Procedure"

  - Validity was proved though Kobe Earthquake (1995) and Tohoku Earthquake (2011)

Seismic Design Procedures (BSL)

Equivalent Lateral Force procedure (1981)

The Calculation Method of Response and Limit Strength (2000)

Energy Balance Based Seismic Resistance Design procedure (2005)

Seismic Design Procedures (BSL)

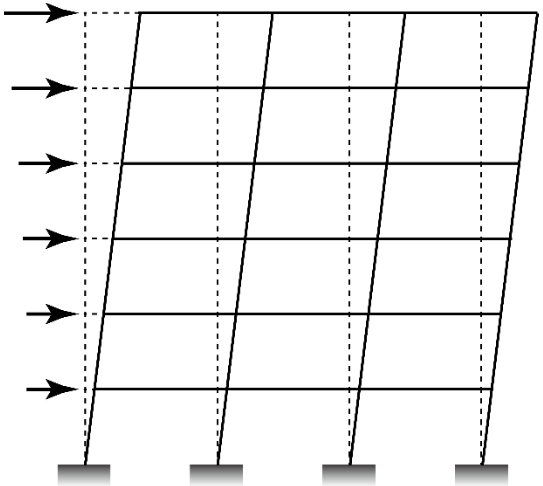
Equivalent Lateral Force procedure (1981)

The Calculation Method of Response and Limit Strength (2000)

Performance Based Design

Energy Balance Based Seismic Resistance Design procedure (2005)

Equivalent Lateral Force Procedure (1981)



Equivalent Lateral Force procedure (1981)

- Three types of design procedure, So called "Route" is stipulated in BSL.
  - Route 3, Route 2, and Route 1

Sophisticated (Default)

Simplified

- Height limitation;
- Size limitation;
- etc…

Design Procedure so called "**Route 3**"

- Can be applied to all size of structures. Building height greater than 31m and less than or equal to 60m should follow this procedure ($31 < H \leqq 60$m).
- Two phases of design should be conducted.
  - Phase 1 : **Allowable Stress Design**
    - Service and Damage Limitation requirements
  - Phase 2 : **Ultimate Strength Design**
    - No-collapse requirement

“Route 3” Phase 1 –Allowable Stress Design-

- Long term and short term should be checked

$$\sigma \leq f$$

allowable stress (BSL) — f

design stress — σ

- Return period of seismic event is about 50 years.

  (about 20% exceedance probability in 10 years)

Example of Allowable Stress (Steel)

Allowable stress	Long term	Short term
Tensile stress	$F/1.5$	$1.5 \times$ (long term values)
Shear stress	$F/(1.5\sqrt{3})$	

“Route 3” Phase 1 –Allowable Stress Design-

Load Combination

<table>
<tr><td rowspan="2">Duration of Force
Long term</td><td rowspan="2">Condition
Regular</td><td colspan="2">Combination</td></tr>
<tr><td>Standard Region</td><td>Heavy Snow Region</td></tr>
<tr><td rowspan="2">Long term (SLS)</td><td>Regular</td><td rowspan="2">G+P</td><td>G+P</td></tr>
<tr><td>Regular + Snow</td><td>G+P+0.7S</td></tr>
<tr><td rowspan="4">Short term (DLS)</td><td>Regular + Snow</td><td>G+P+S</td><td>G+P+S</td></tr>
<tr><td rowspan="2">Regular + wind</td><td rowspan="2">G+P+W</td><td>G+P+W</td></tr>
<tr><td>G+P+0.35S+W</td></tr>
<tr><td>Regular + Earthquake</td><td>G+P+K</td><td>G+P+0.35S+K</td></tr>
</table>

G is Dead load effects, P is live load effects, S is Snow load effects, W is wind load effects, and K is seismic load effects

"Route 3" –Story Drift check-

- Return period of seismic event is about 50 years.

  (about 20% exceedance probability in 10 years)

Based on this seismic action, story drift ratio at

i story should be satisfied.

$$SDR_i \leq \frac{1}{200}$$

This value can be relaxed to 1/120 (0.0083) when the non-structural components are not affected.

"Route 3" –Story Drift check-

- Return period of seismic event is

(about 20% exceedance probabili

Based on this seismic action, stor

i story should be satisf

$$SDR_i \leq \frac{1}{200}$$

EN 1998-1 4.4.3.2
Limitation of interstorey drift

a) non-structural elements of brittle materials attached to the structure:

$$d_r \nu / h \leq 0.005$$

b) Ductile non-structural elements:

$$d_r \nu / h \leq 0.0075$$

c) Without non-structural elements:

$$d_r \nu / h \leq 0.01$$

This value can be relaxed to 1/120 (0.0083) when the non-structural components are not affected.

"Route 3" Phase 2 –Ultimate Strength Design-

- Structural Safety should be confirmed.
- Return period of seismic event is about 500 years. (about 10% exceedance probability in 50 years)
- Horizontal load-carrying capacity should be greater than or equal to the required strength.

$$Q_{un,i} \leq Q_{u,i}$$

Horizontal load-carrying capacity

Required Horizontal load-carrying capacity (BSL)

"Route 3" Phase 2 –Ultimate Strength Design-

- Required horizontal load-carrying capacity, $Q_{un,i}$

$$Q_{un,i} = D_{s,i} \cdot F_{es,i} \cdot Q_{ud,i}$$

Load action determined by linear elastic response

Shape factor

Ductility Reduction Factor

- Strong Column Weak Beam Philosophy
  - for column steel grade BCR and BCP (at Floor Level)

$$\sum M_{pc} = \sum \min\left\{1.5M_{pb},\ 1.3M_{pp}\right\}$$

  - for column steel grade STKR (at All Joints)

$$\sum M_{pc} = 1.5 \sum M_{pb}$$

"Route 3" Phase 2 –Ultimate Strength Design-

- Required horizontal load-carrying cap

$$Q_{un,i} = D_{s,i} \cdot F_{es,i} \cdot Q_{ud,i}$$

Load a
by line

Shape

Ductil

- Strong Column Weak Beam Philosoph
  - for column steel grade BCR and BC

$$\sum M_{pc} = \sum \min\{1.5M_{pb},\ 1.3M_{pp}\}$$

  - for column steel grade STKR (at Al

$$\sum M_{pc} = 1.5 \sum M_{pb}$$

EN 1998-1 4.4.2.3
Global and local ductility condition

(3)P In multi-storey buildings formation of a soft storey plastic mechanism shall be prevented.

(4) To satisfy the requirements of (3)P, following conditions should be satisfied at all joints.

$$\sum M_{Rc} = 1.3 \sum M_{Rb}$$

"Route 3" Phase 2 –Ultimate Strength Design-

- Load action determined by linear elastic response, $Q_{ud,i}$

$$Q_{un,i} = D_{s,i} \cdot F_{es,i} \cdot \boxed{Q_{ud,i}}$$

$$Q_{ud,i} = C_i \cdot W_i$$

W_i: Total weight supported at *i* story $= \sum_{j=i}^{n} w_i$

C_i: Seismic story shear (force) coefficient at *i* story

$$C_i = Z \cdot R_t \cdot A_i \cdot C_0$$

C_0: Intensity (≧1.0 for phase 2, ≧0.2 for phase 1)

A_i: Lateral force distribution (≧1.0)

R_t: Normalized elastic response acceleration (≦1.0)

Z: Region coefficient (0.7 to 1.0)

"Route 3" Phase 2 –Ultimate Strength Design-

- Load action determined by linear elastic response, $Q_{ud,i}$

$$Q_{un,i} = D_{s,i} \cdot F_{es,i} \cdot \boxed{Q_{ud,i}}$$

$$Q_{ud,i} = C_i \cdot W_i$$

W_i: Total weight supported at *i* story $= \sum_{j=i}^{n} w_i$

EN 1998-1 3.2.2.2
Horizontal elastic response spectrum

C_i: Seismic story shear (force) coefficient at *i* story

$$C_i = \boxed{Z \cdot R_t} \cdot A_i \cdot C_0$$

C_0: Intensity (≧1.0 for phase 2, ≧0.2 for phase 1)

A_i: Lateral force distribution (≧1.0)

R_t: Normalized elastic response acceleration (≦1.0)

Z: Region coefficient (0.7 to 1.0)

"Route 3" Phase 2 –Ultimate Strength Design-

- Load action determined by linear elastic response, $Q_{ud,i}$

$$Q_{un,i} = D_{s,i} \cdot F_{es,i} \cdot \boxed{Q_{ud,i}}$$

$$Q_{ud,i} = C_i \cdot W_i$$

Total weight supported at i story $= \sum_{j=i}^{n} w_i$

EN 1998-1 4.3.3.3
Modal response spectrum analysis

Seismic story shear (force) coefficient at i story

$$C_i = Z \cdot R_t \cdot \boxed{A_i} \cdot C_0$$

Intensity (≧1.0 for phase 2, ≧0.2 for phase 1)

Lateral force distribution (≧1.0)

Normalized elastic response acceleration (≦1.0)

Region coefficient (0.7 to 1.0)

“Route 3” Phase 2 –Ultimate Strength Design-

- Shape factor, $F_{es,i}$

$$Q_{un,i} = D_{s,i} \cdot \boxed{F_{es,i}} \cdot Q_{ud,i}$$

$$F_{es,i} = F_{e,i} \cdot F_{s,i}$$

Penalty factor to consider irregularity in elevation (1.0 to 2.0)

Penalty factor to consider irregularity in plan (1 to 1.5)

Shape factor will range from 1.0 to 3.0

"Route 3" Phase 2 –Ultimate Strength Design-

- Shape factor, $F_{es,i}$

$$Q_{un,i} = D_{s,i} \cdot \boxed{F_{es,i}} \cdot Q_{ud,i}$$

$$F_{es,i} = \boxed{F_{e,i}} \cdot F_{s,i}$$

Penalty factor to consider irregularity in elevation (1.0 to 2.0)

Penalty factor to consider irregularity in plan (1 to 1.5)

EN 1998-1 4.2.3.2
Criteria for regularity in plan
NOTE: Irregular building should be computed by Special model.

1.0 to 3.0

"Route 3" Phase 2 –Ultimate Strength Design-

- Shape factor, $F_{es,i}$

$$Q_{un,i} = D_{s,i} \cdot \boxed{F_{es,i}} \cdot Q_{ud,i}$$

$$F_{es,i} = F_{e,i} \cdot \boxed{F_{s,i}}$$

BSL is based on story stiffness.

Penalty factor to consider irregularity in elevation (1.0 to 2.0)

Pen[illegible] [illegible]rity in plar[illegible]

EN 1998-1 4.2.3.3
Criteria for regularity in elevation
NOTE: irregularity is judged by configuration only.
Irregular building should be computed by Modal analysis results.

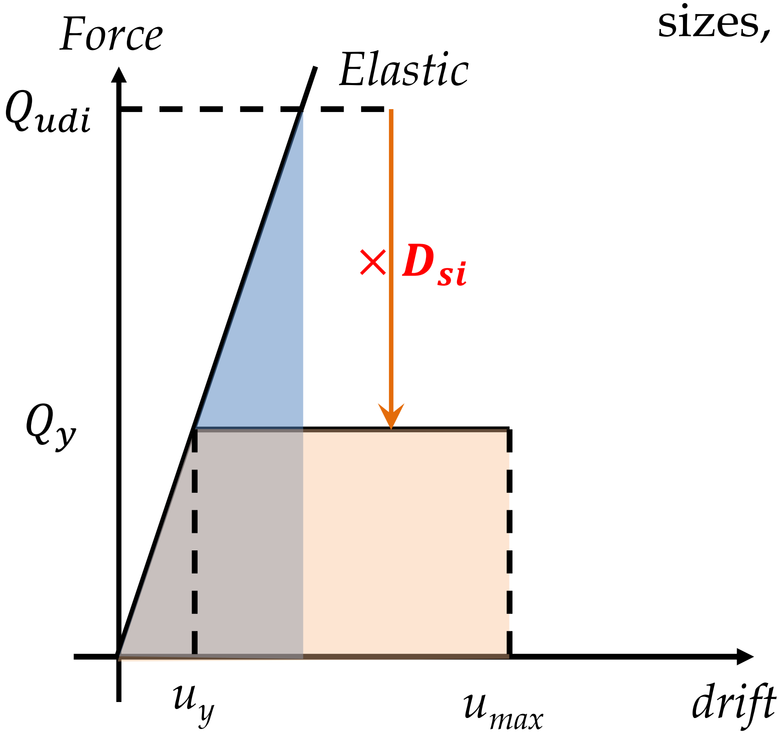
Shape factor[illegible]

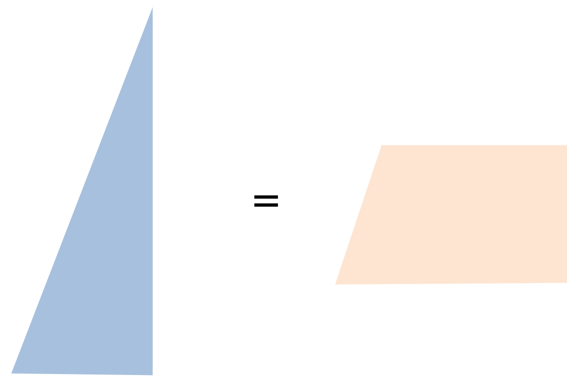
"Route 3" Phase 2 –Ultimate Strength Design-

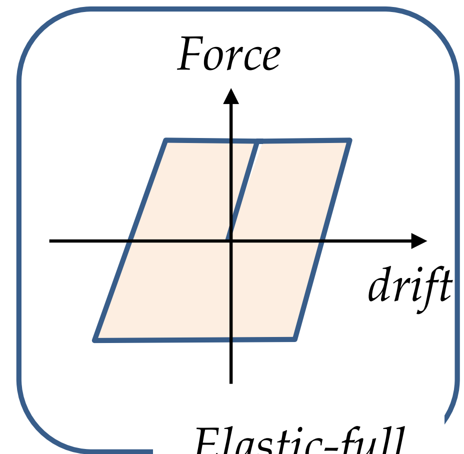
- Ductility Reduction Factor, $D_{s,i}$

$$Q_{un,i} = \boxed{D_{s,i}} \cdot F_{es,i} \cdot Q_{ud,i}$$

Basically, **Newmark Rule** is applied
This value is determined from the member sizes, i.e. compactness(0.25 to 0.50)







Elastic-full plastic Behaviour

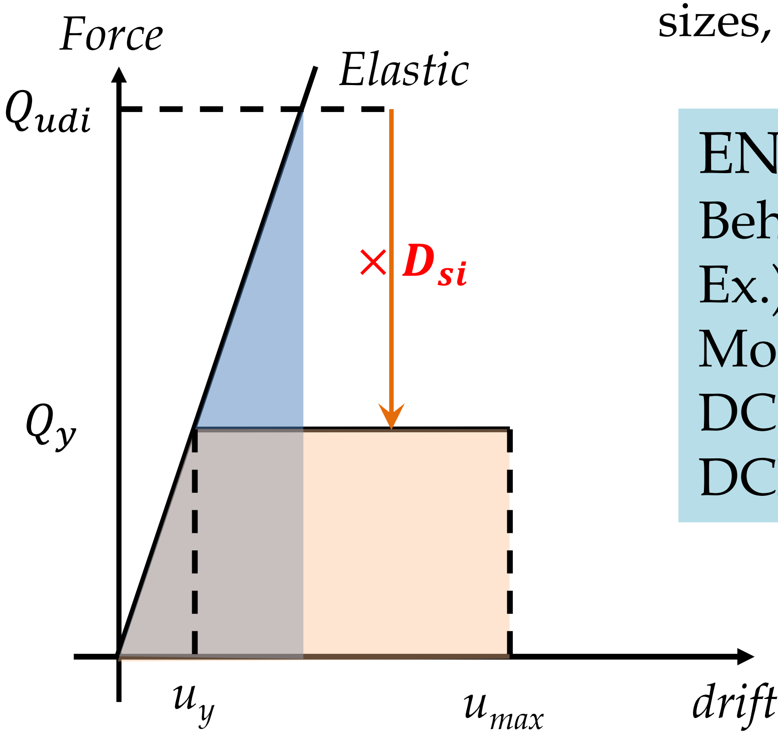
$$D_{s,i} = \frac{Q_y}{Q_{udi}} = \frac{1}{\sqrt{2\frac{u_{max}}{u_y} - 1}} = \frac{1}{\sqrt{2\mu - 1}}$$

"Route 3" Phase 2 –Ultimate Strength Design-

- Ductility Reduction Factor, $D_{s,i}$

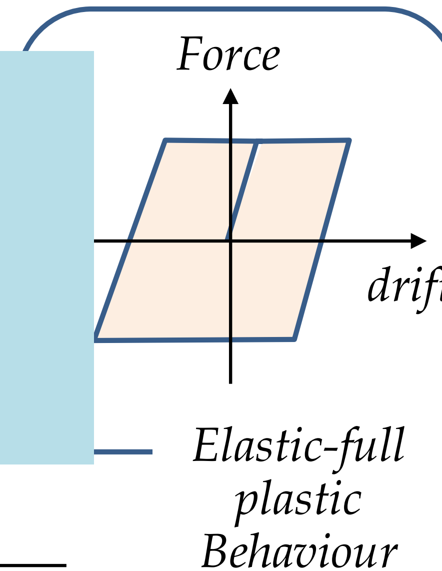
$$Q_{un,i} = \boxed{D_{s,i}} \cdot F_{es,i} \cdot Q_{ud,i}$$

Basically, **Newmark Rule** is applied
This value is determined from the member sizes, i.e. compactness(0.25 to 0.50)



EN 1998-1 6.3.2
Behaviour factors q
Ex.)
Moment Resisting Frames
DCM q=4
DCH $q=5\alpha_u/\alpha_1$

$$= \frac{1}{\sqrt{2\frac{u_{max}}{u_y} - 1}} = \frac{1}{\sqrt{2\mu - 1}}$$



“Route 3” Phase 2 –Ultimate Strength Design-

- Ductility Reduction Factor, $D_{s,i}$

Ds values			Classification of Group of Beam and Column			
A or bu = 0			A	B	C	D
Classification of Group of Braces	A or $\beta_u = 0$		0.25	0.3	0.35	0.4
	B	$\beta_u \leqq 0.3$	0.25	0.3	0.35	0.4
		$0.3<\beta_u \leqq 0.7$	0.3	0.3	0.35	0.45
		$\beta_u > 0.7$	0.35	0.35	0.4	0.5
	C	$\beta_u \leqq 0.3$	0.3	0.3	0.35	0.4
		$0.3<\beta_u \leqq 0.7$	0.35	0.35	0.4	0.45
		$\beta_u > 0.7$	0.4	0.4	0.45	0.5

“Route 3” Phase 2 –Ultir

- Ductility Reduction Factor, $D_{s,}$

Ds values		Classi			
A or bu = 0		A			
	A or β_u = 0	0.25			
		0.25			
		0.3			
		0.35			
		0.3	0.3	0.35	0.4
		0.35	0.35	0.4	0.45
		0.4	0.4	0.45	0.5

EN 1998-1 6.5.3
Design rules for dissipative elements in compression or bending

Required cross-sectional class are tabulated in tabulated 6.3.
Ex.) DCH q>4 class 1

NOTE:
ONLY cross-sectional class

AIJ has a additional requirement for beam and column to avoid strength degradation and guarantee member ductility.

<Beam>

- Distance of lateral support.
- Required strength and stiffness for lateral supports

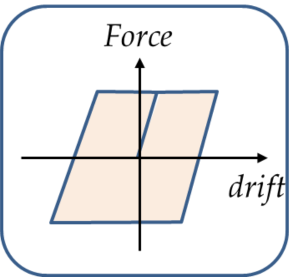
<Column>

- Limitation for compressive axial force with bending moment ratio.

Action and Resistance

Action

- BSL
  - Seismic Action
    - Ductility Reduction Factor



- ➢ Newmark's Rule
- ➢ Dissipative Zones at Members (Beams)
- ➢ **Rigid Joint**
- ➢ **Semi-Rigid Joint**

NO DISCRIPTION

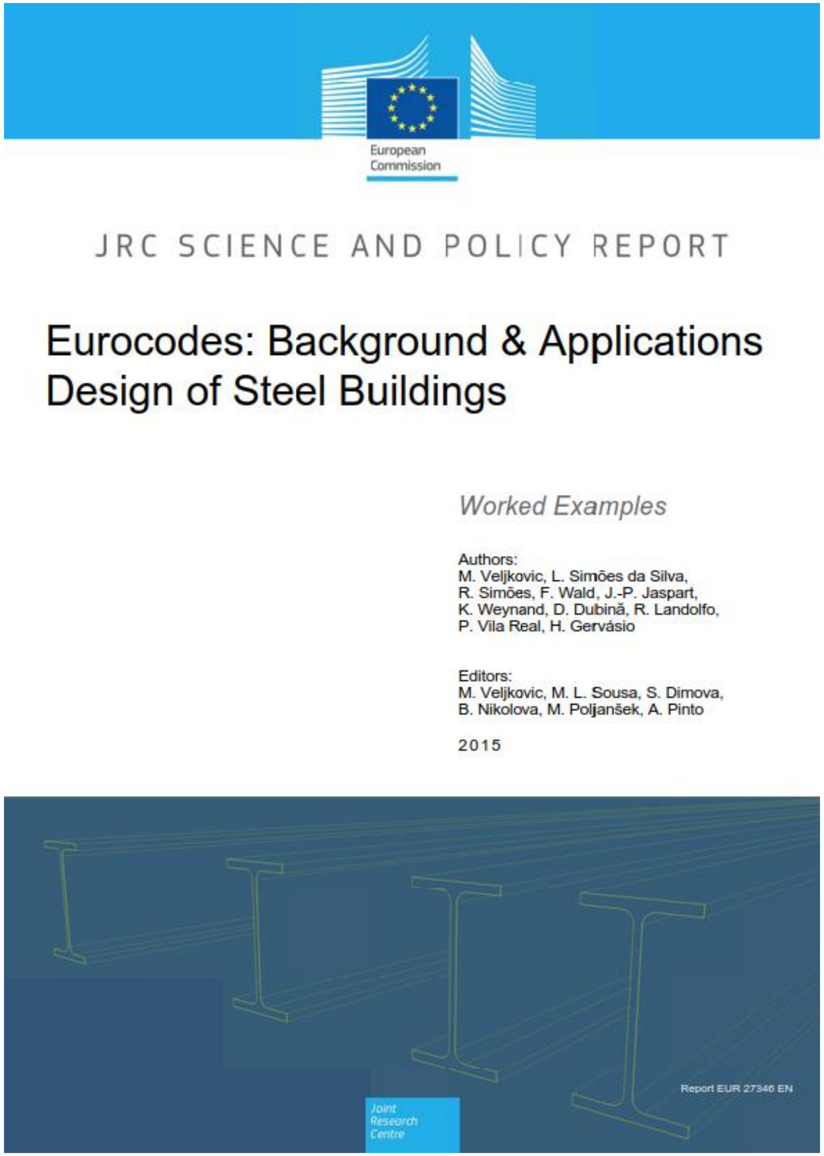
- EuroCode 1 and 8
  - Seismic Action
    - Behaviour Factor *q* (Depending of Ductile Class)

?

Resistance

- BSL (Allowable Stress)
- AIJ (Ultimate Limit State)
  - Capacity Design
  - Detail for **Rigid Joint**
  - Semi-rigid joints are shown but not used in practice.
- Eurocode 3
  - Capacity Design
  - Rigid or Semi-rigid Joints
- Eurocode 8
  - Rigid or Semi-rigid Joints

Numerical Simulation (Example Study)



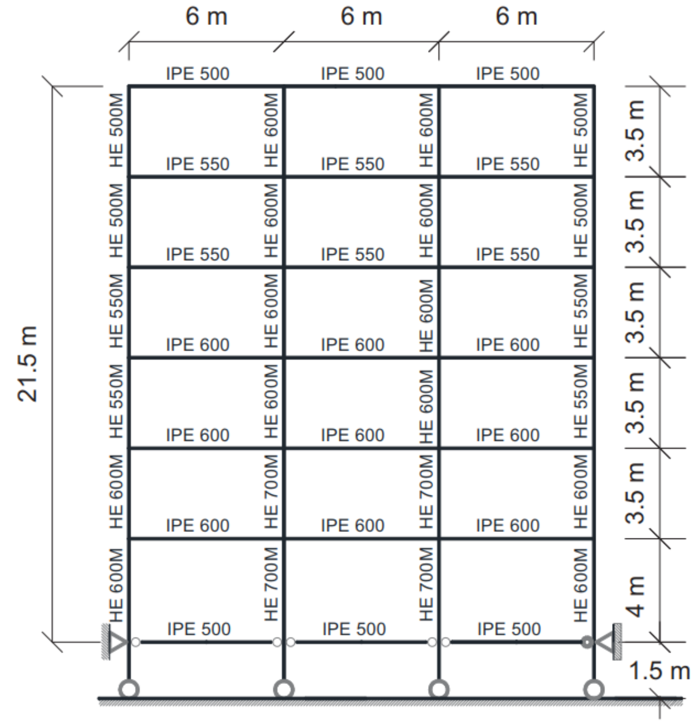


Figure 5.14 Configuration of the designed MRFs.

- Selected Beam Joints
  - Rigid
  - Rigid Lower Bound ($=30EI_b/L$)
  - Semi-Rigid ($=10EI_b/L$)
  - Full and Partial Strength
- Non-Linear Analysis (Material and Geometrical)

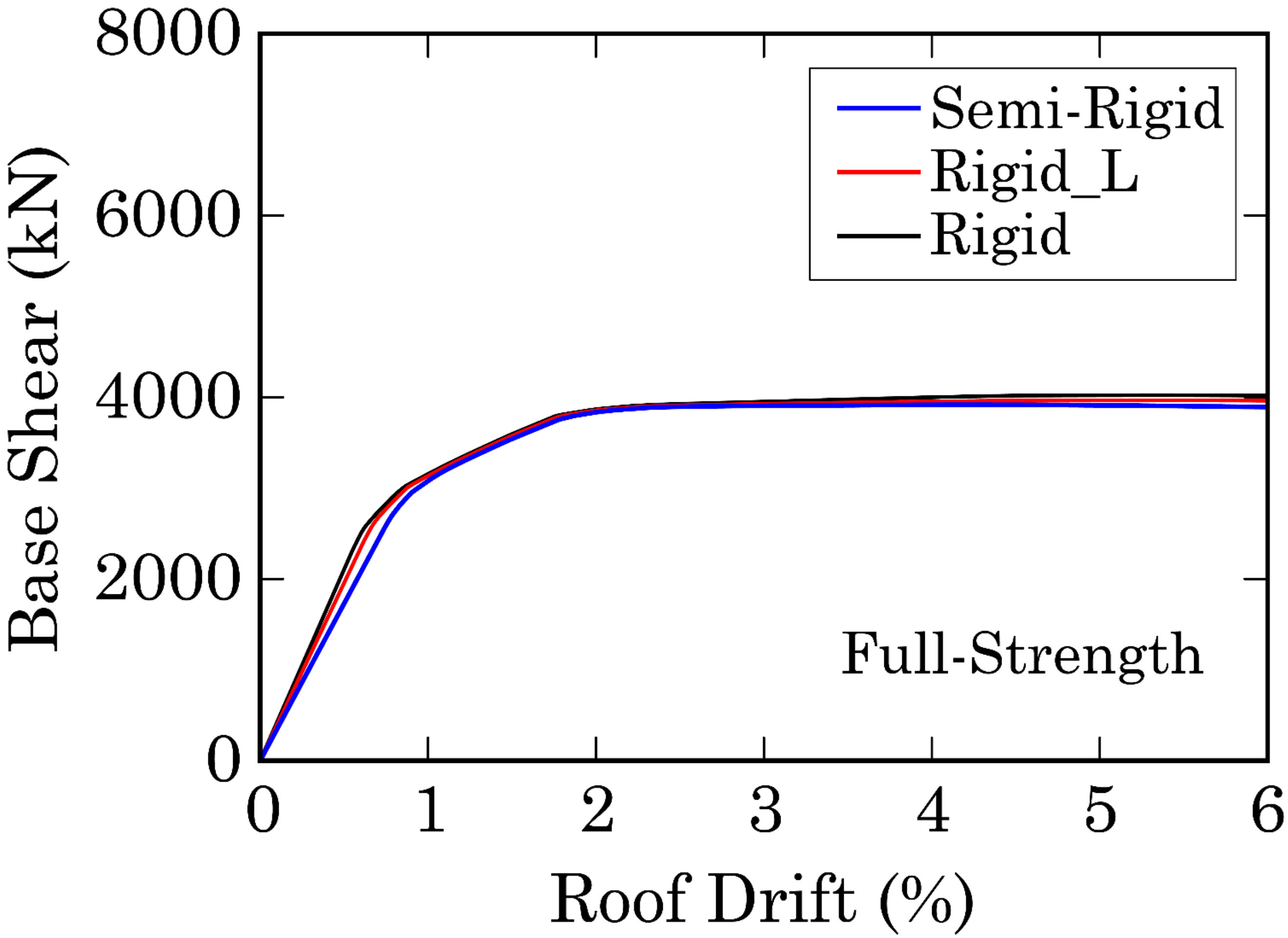
Type of Beam Joint model

Structural Analysis Model Matrix

Joint models		Joint Stiffness		
		Rigid	Rigid (Lower bound)	Semi-Rigid
Full-Strength	Without pinching	×	×	×
	With pinching	-	×	×
Partial-Strength	With pinching	-	×	×

※ For Partial strength, Beam strengths were increased 1.25 times.

Pushover Analysis



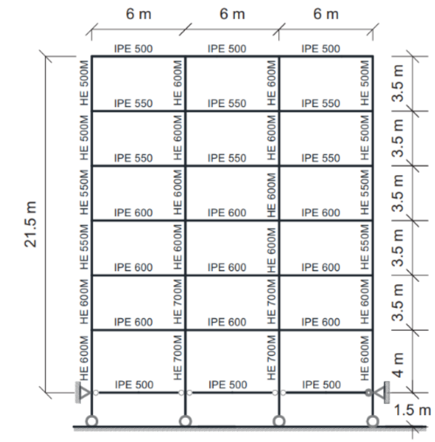
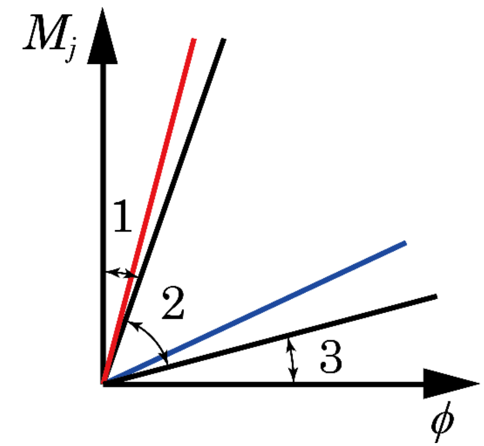


Figure 5.14 Configuration of the designed MRFs.

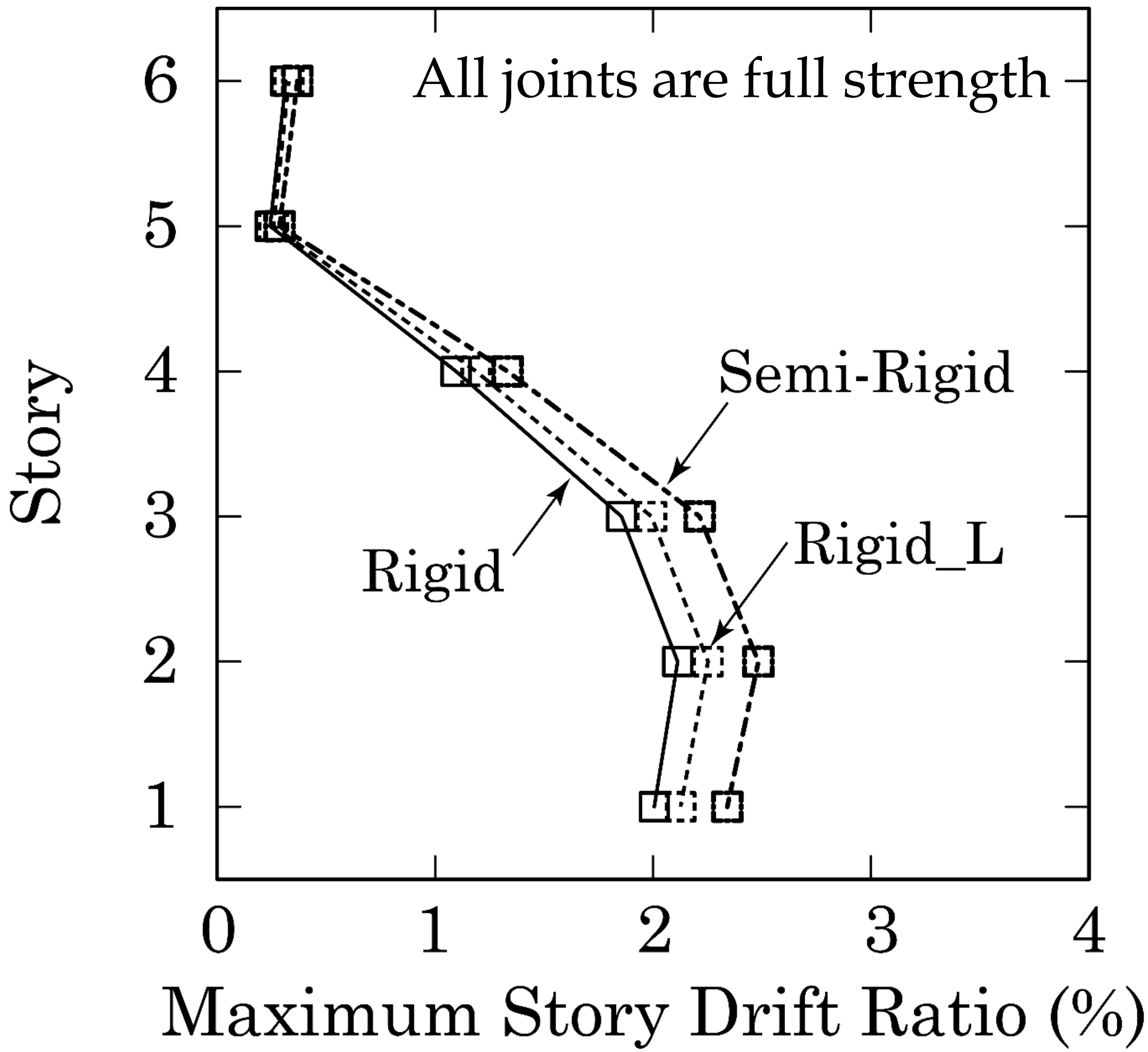


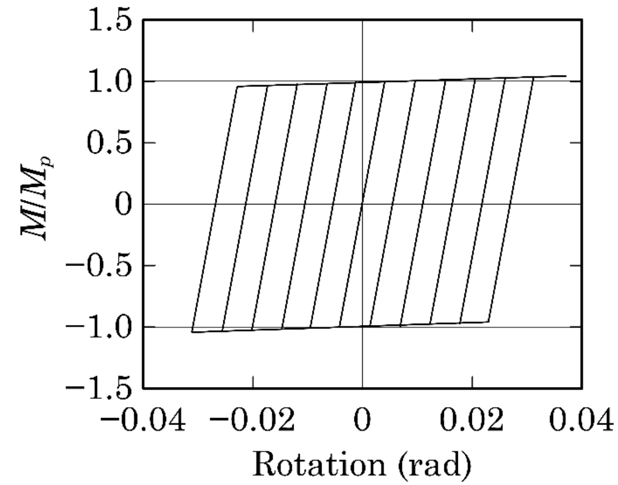
Classification of joints by stiffness

Non-Linear Time History Analysis

Joint models		Joint Stiffness		
		Rigid	Rigid (Lower bound)	Semi-Rigid
Full-Strength	Without pinching	×	×	×
	With pinching	-	×	×
Partial-Strength	With pinching	-	×	×

- 44 Ground Motion Records per **FEMA P 695**



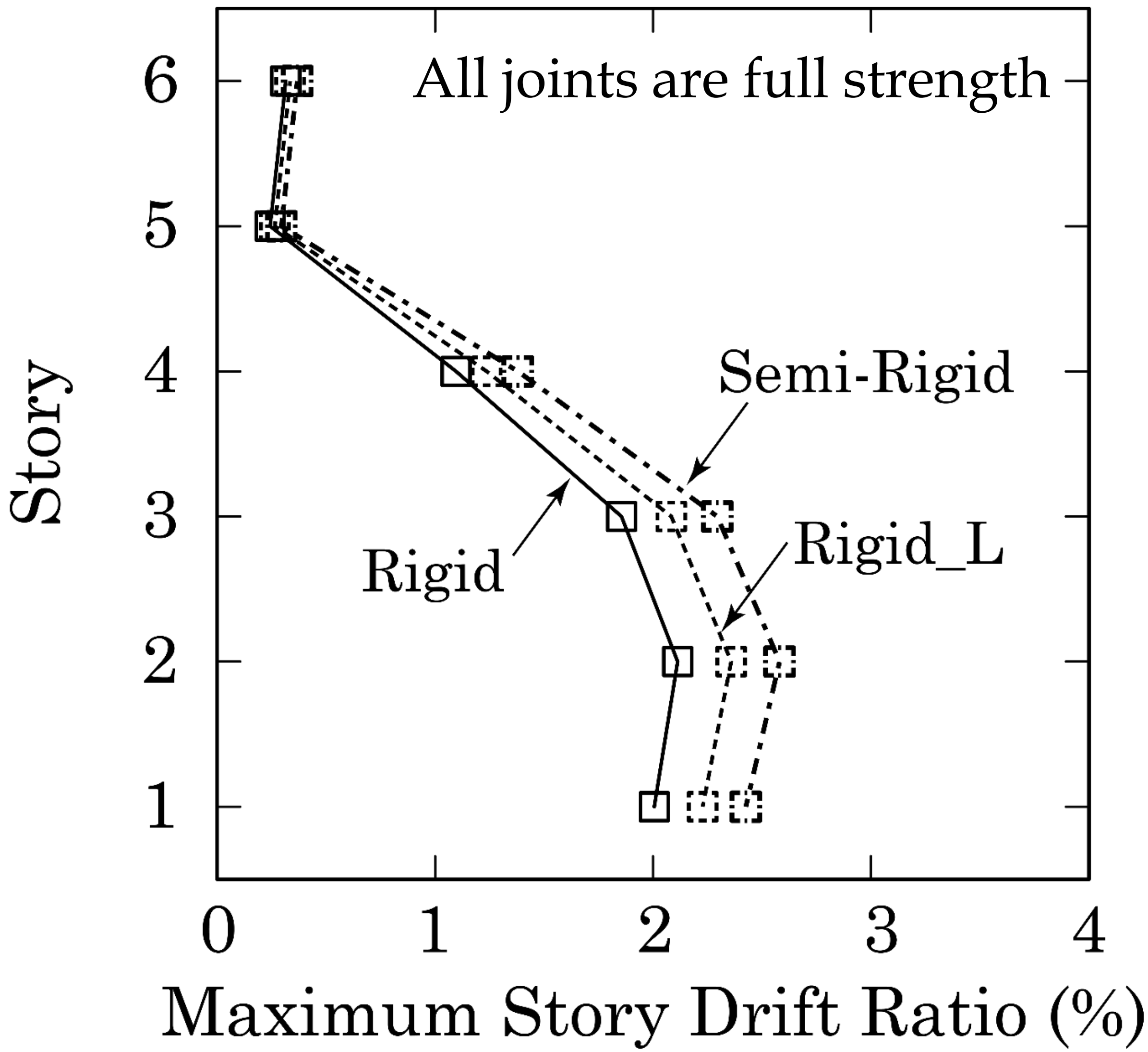


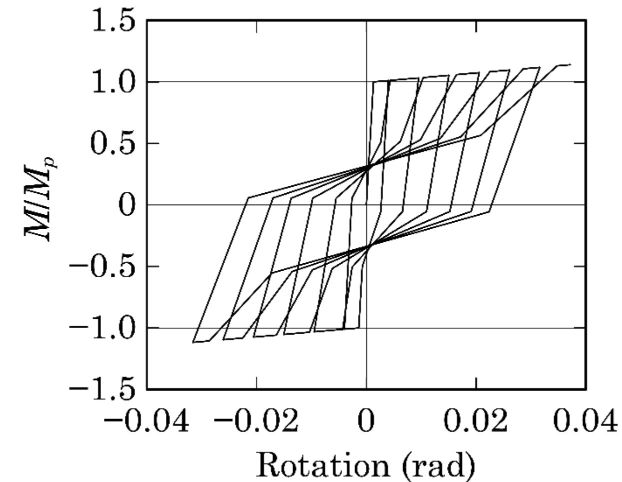
Assumed joint behavior
Without Pinching

Non-Linear Time History Analysis

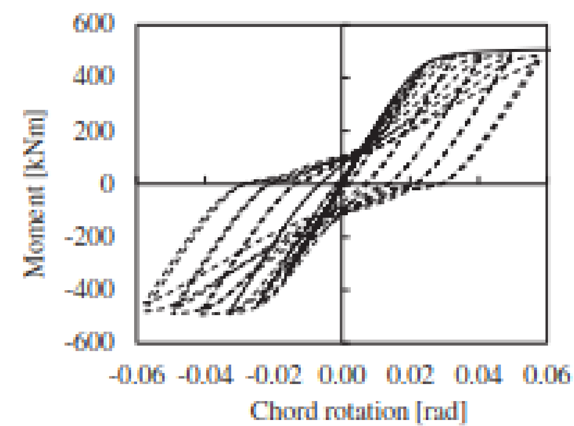
- 44 Ground Motion Records per **FEMA P 695**

Joint models		Joint Stiffness: Rigid	Rigid (Lower bound)	Semi-Rigid
Full-Strength	Without pinching	×	×	×
	With pinching	-	×	×
Partial-Strength	With pinching	-	×	×





Assumed joint behavior
With Pinching

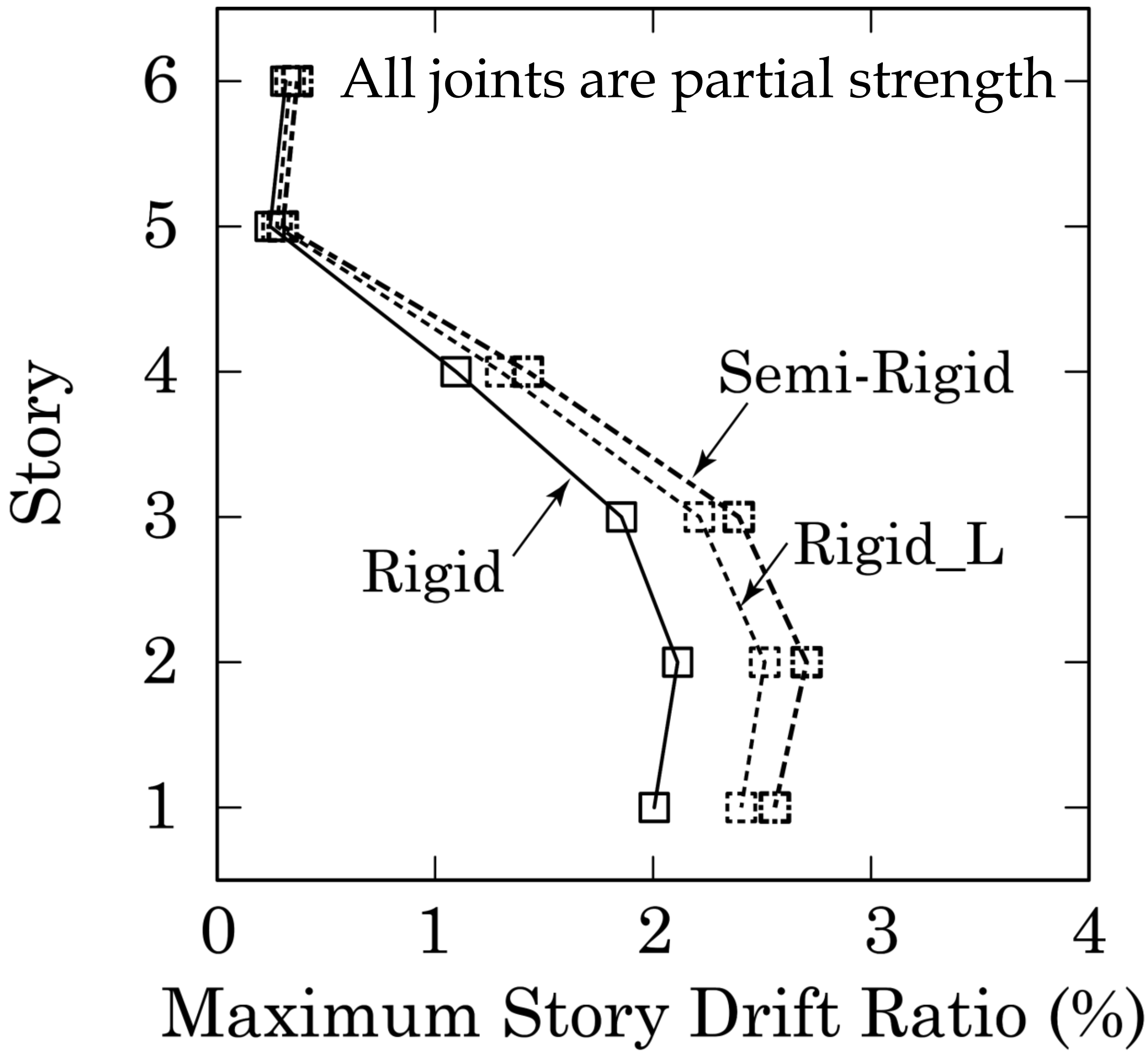


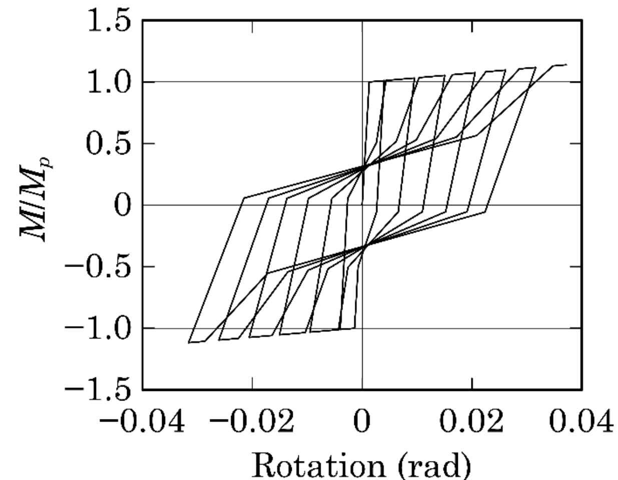
Mario D'Aniello et al. [2017]

Non-Linear Time History Analysis

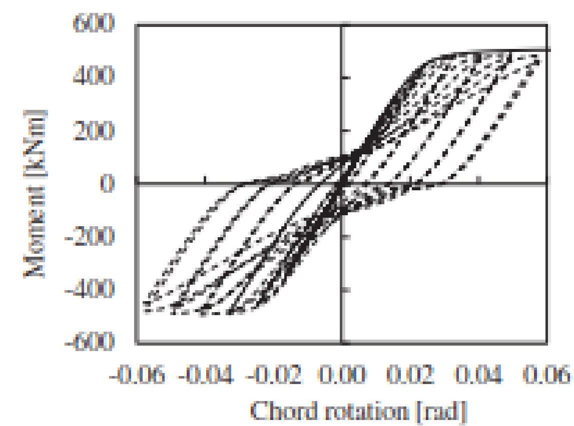
Joint models		Joint Stiffness		
		Rigid	Rigid (Lower bound)	Semi-Rigid
Full-Strength	Without pinching	×	×	×
	With pinching	-	×	×
Partial-Strength	With pinching	-	×	×

- 44 Ground Motion Records per **FEMA P 695**





Assumed joint behavior
With Pinching

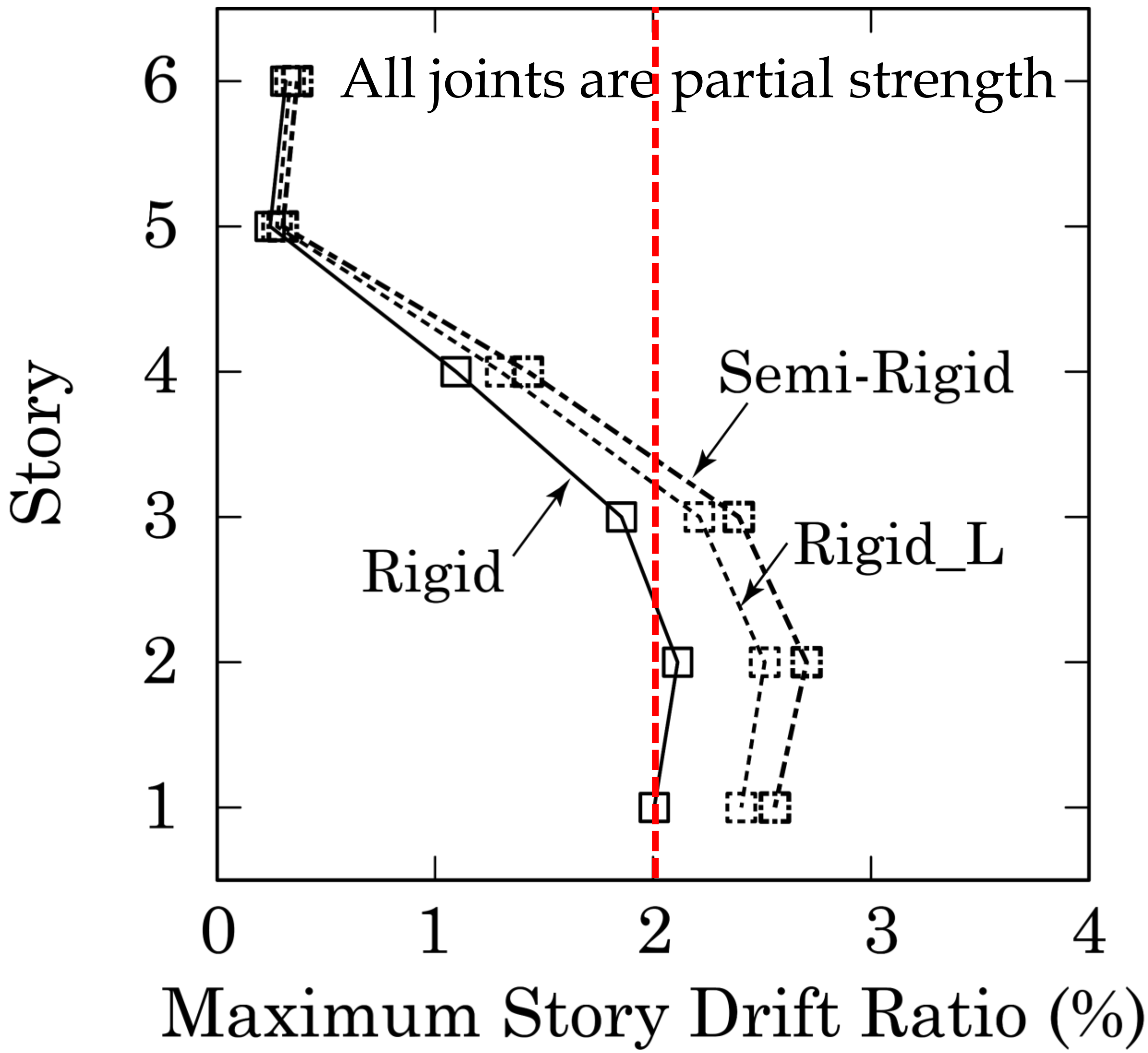


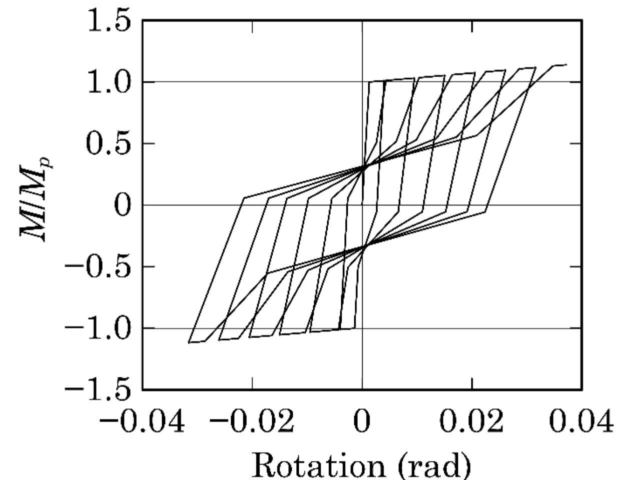
Mario D'Aniello et al. [2017]

Non-Linear Time History Analysis

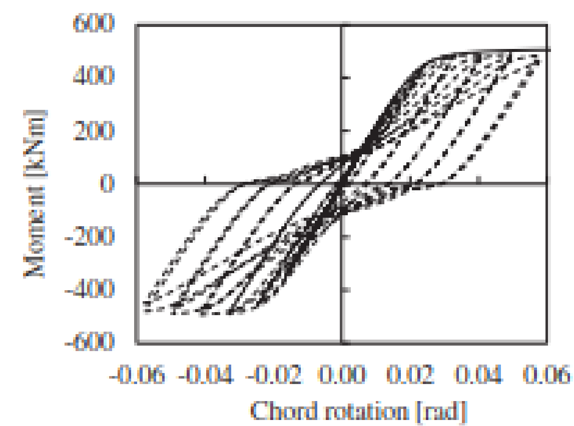
Joint models		Joint Stiffness: Rigid	Rigid (Lower bound)	Semi-Rigid
Full-Strength	Without pinching	×	×	×
	With pinching	-	×	×
Partial-Strength	With pinching	-	×	×

ASCE 7-10 LIMITATION





Assumed joint behavior
With Pinching



Mario D'Aniello et al. [2017]

Summary of Time History Analysis

Average Maximum Story Drift Ratio under 44 Ground Motions

Joint models		Joint Stiffness		
		Rigid	Rigid (Lower bound)	Semi-Rigid
Full-Strength	Without pinching	2.11(%) (1.00)	2.25(%) (1.07)	2.48(%) (1.18)
	With pinching	-	2.39(%) (1.13)	2.58(%) (1.22)
Partial-Strength	With pinching	-	2.51(%) (1.19)	2.70(%) (1.28)

※ For Partial strength, Beam strengths were increased 1.25 times.

Results from Time History Analysis

Average Maximum Story Drift Ratio under 44 Ground Motions

Joint models		Joint Stiffness		
		Rigid	Rigid (Lower bound)	Semi-Rigid
Full-Strength	Without pinching	2.11(%) (1.00)	2.25(%) (1.07)	2.48(%) (1.18)
	p			
Partial-Strength	p			

※ For Partial s

Can we use same "Behaviour Factor *q*" for different Beam Joints?

→ For Semi-rigid and Partial-Strength joints, *q* value should be greater than the value used for rigid joints.

Can be a future research topic?

After Kobe Earthquake (after 1995)

BSL is a minimum requirement; protection of the human life is the main objective.

- Damage is allowed in Ultimate Limit State, and after a severe seismic action it should be demolished and do a reconstruction.

However, in current social system does not allow this concept. Level of damage due to severe earthquake should be controlled by the designer.

Performance Based Design became a high demand

Not only protecting the human life but also maintain the function of the buildings

Seismic Design Procedures (BLS)

- Equivalent Lateral Force procedure (1981)
- The Calculation Method of Response and Limit Strength (2000)
- Energy Balance Based Seismic Resistance Design procedure (2005)

Next Time!

Installation of Damper (Oil Damper)

- Example

Reference: KYB https://www.kyb-ksm.co.jp/products/

Installation of Damper (Steel Damper)

- Examples




Reference: NSENGI https://www.nsec-steelstructures.jp/

Recommendation from AIJ

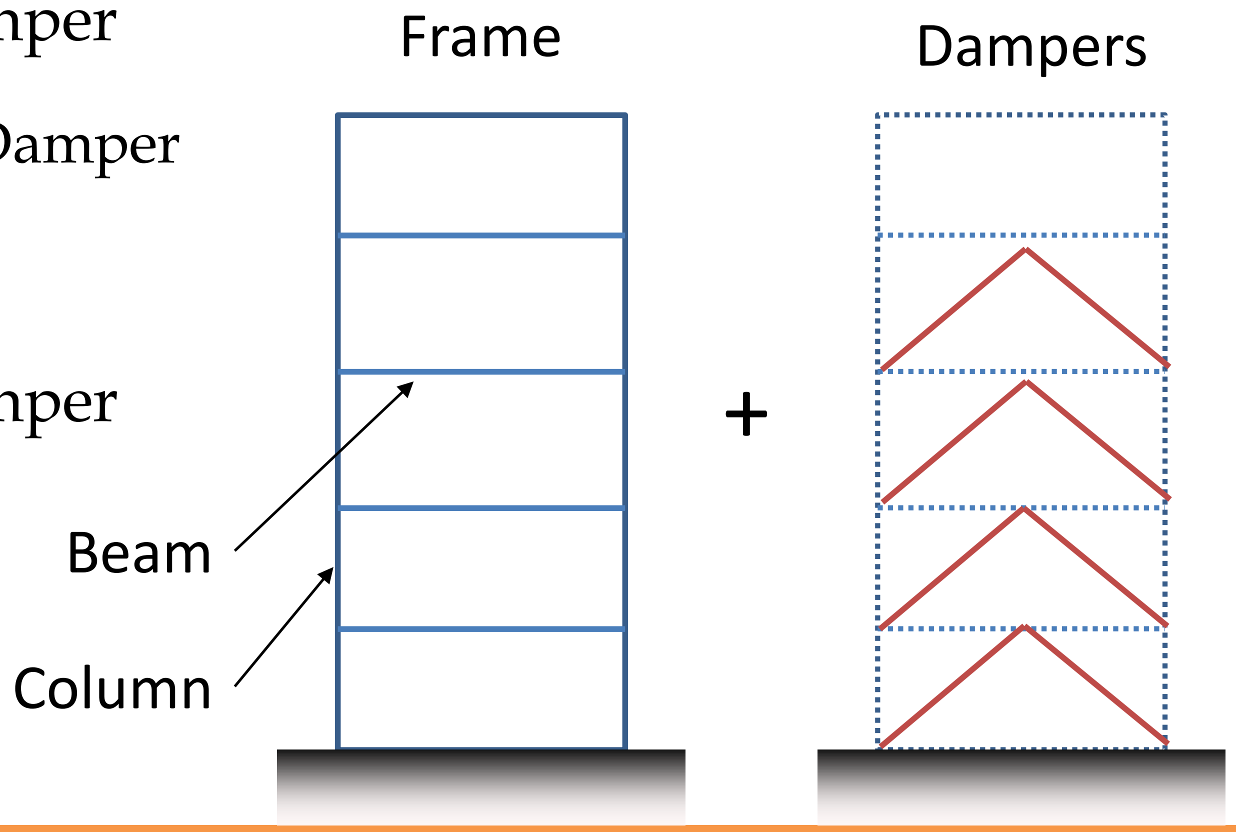
- Recommended Provisions for Seismic Damping Systems applied to Steel Structures (2014)

Recommendation from JSSI

- Design of Passive Damping (2005, 2013)

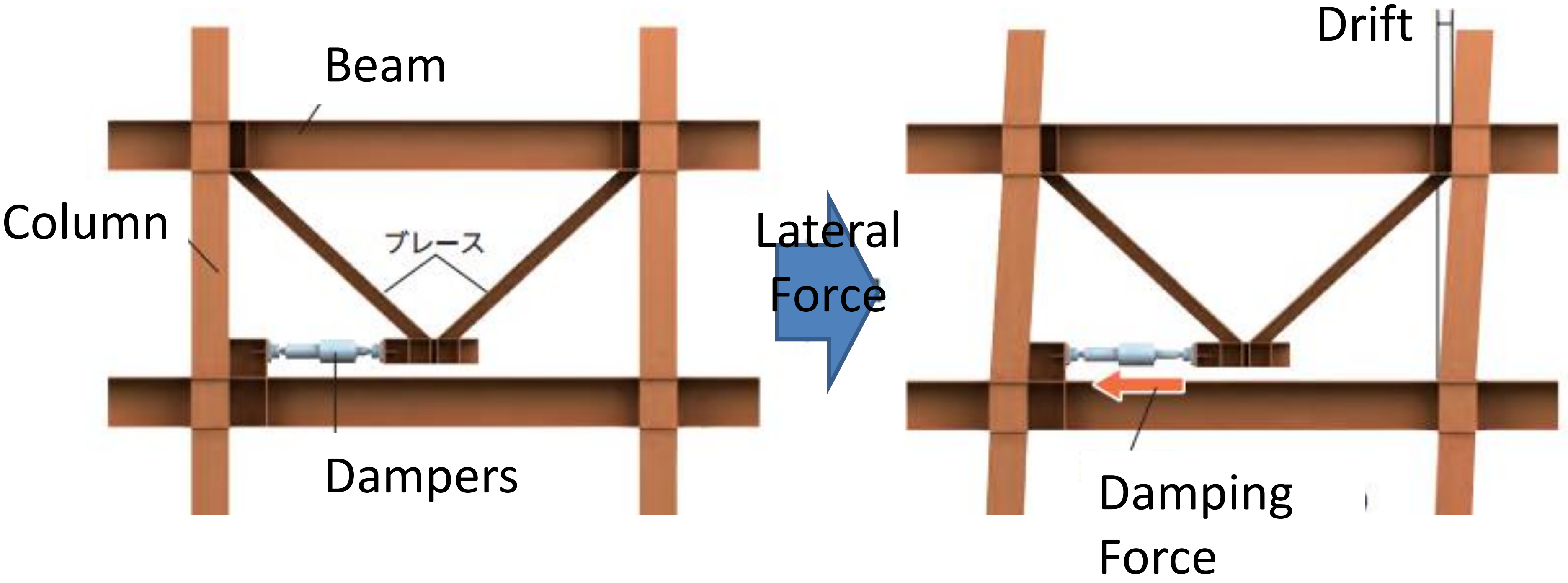
  Design Procure for following dampers are shown.

  - Steel Damper
  - Friction Damper
  - Viscoelastic Damper
  - Oil Damper
  - Viscous Damper



Installation of Damper (Oil Damper)

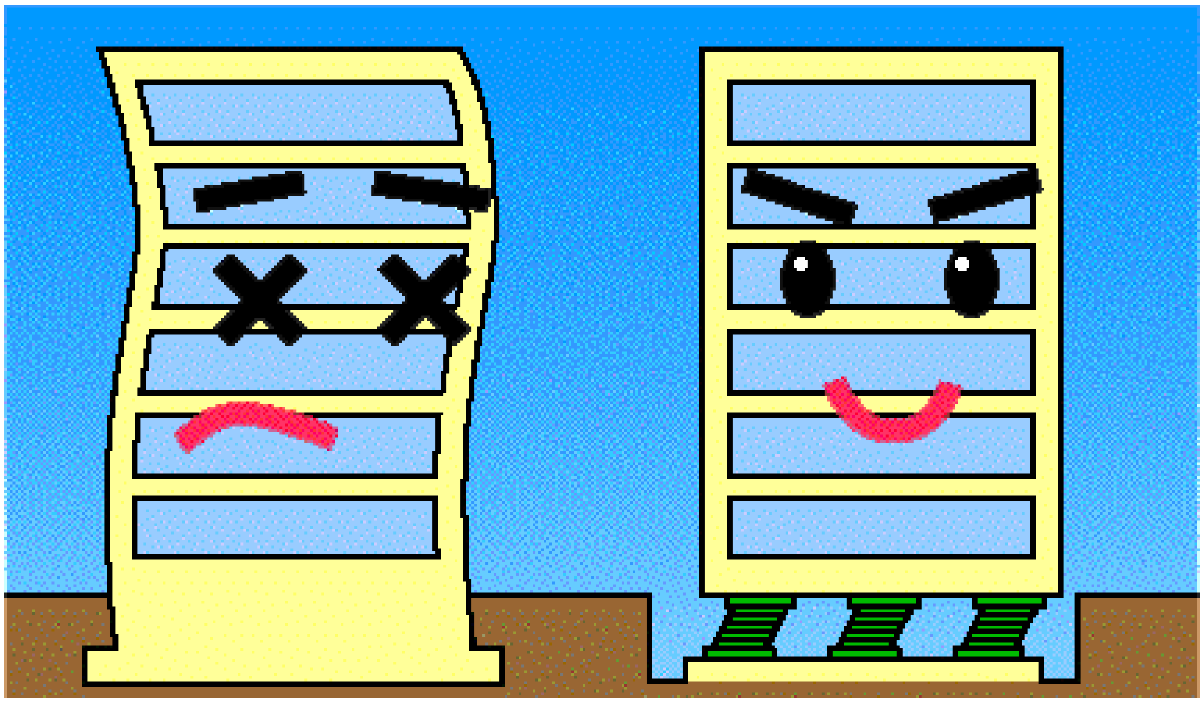
- Example



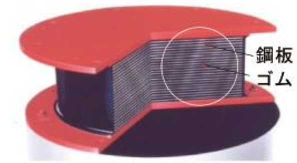
Reference: SENQCIA https://www.senqcia.co.jp/products/kz/damper/

Base Isolated Structure

- Concept of this structure



Force Resisting Structure Base Isolated Structure



Isolator

Steel Damper

Oil Damper

Reference: JSSI http://www.jssi.or.jp/menshin/m_kenchiku.html

Base Isolated Structure

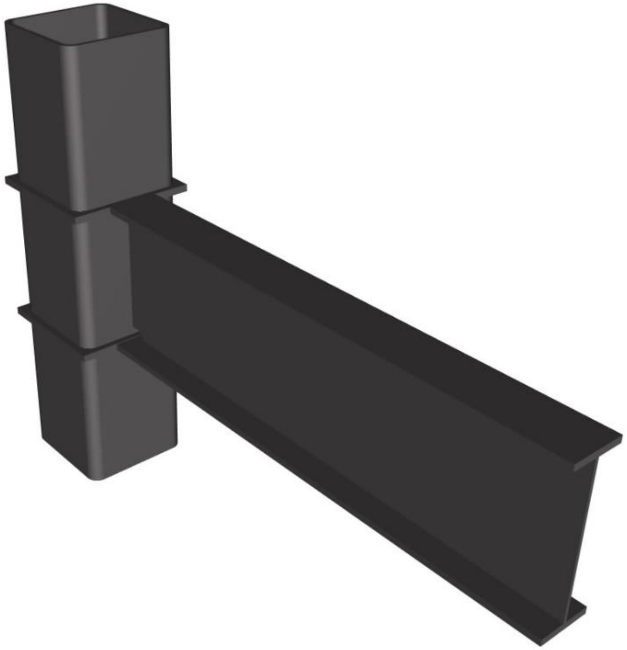
- Examples

Reference: NSENGI https://www.nsec-steelstructures.jp/

Topics -Steel Structure-

- Design Procedure (General)
  - History and Concept
- Design of Beam-to-Column Connection
- Design of Column
- Recent Research Topics in my Group
  - Column
  - Beam-to-Column Connection

Design of
Beam-to-Column Connection



Beam Joint Design

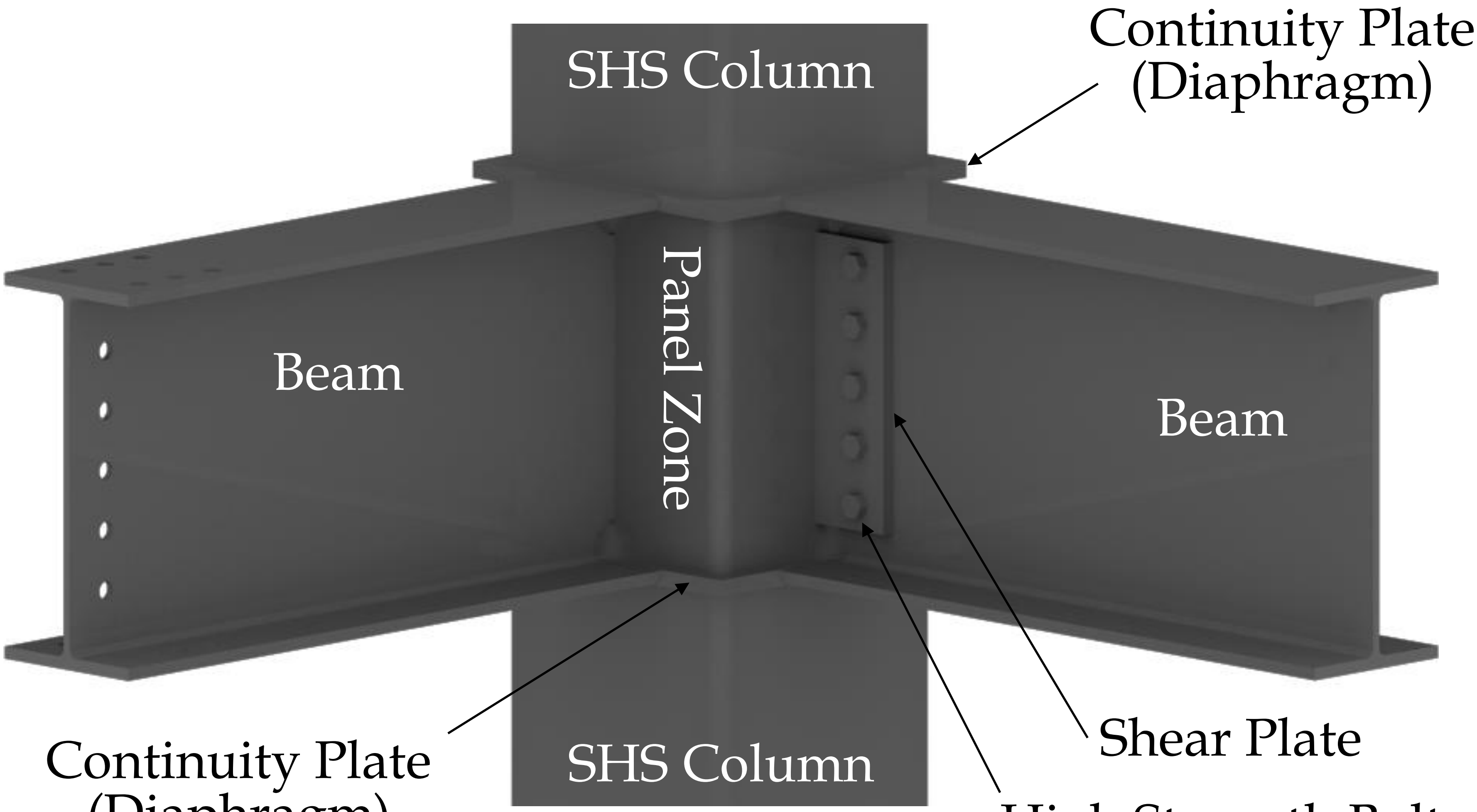
- **Rigid Joint**
  - Dissipative Zones are assumed at beam ends (BSL)
    - Strong Column weak beam philosophy
  - D_s factor (Behaviour factor q in EC8) is given base on this assumption.
  - To achieve rigid joints with Square Hollow Section (SHS) columns, Japan has unique detail.
    - Avoid local deformation at the joints
      - Continuity plates (diaphragm) are the must.

We Love Welding!

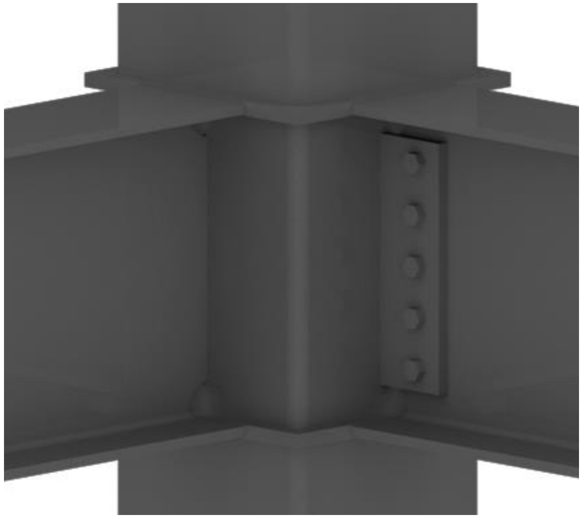
Typical Beam-to-Column Connection

Shop Welded Detail

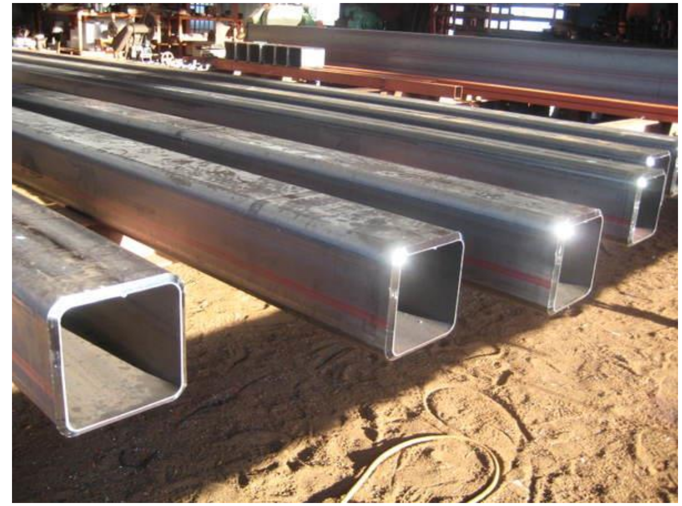
Field Welded Detail



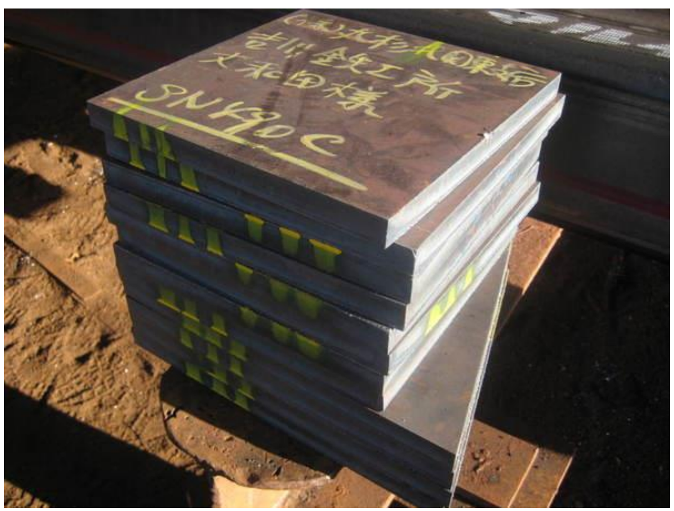
Typical Beam-to-Column Connection



Panel Zone



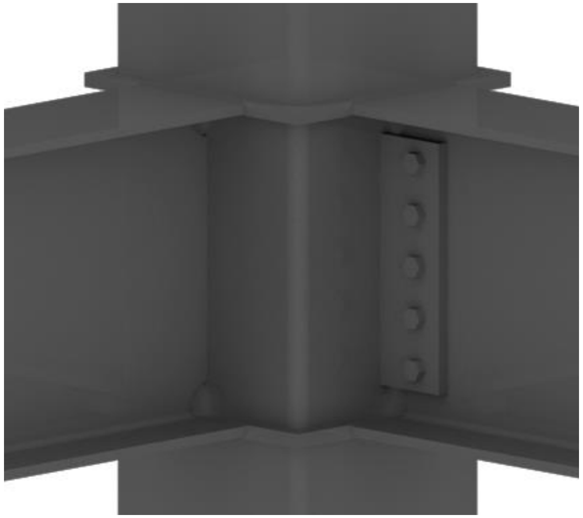
Square Hollow Section Column



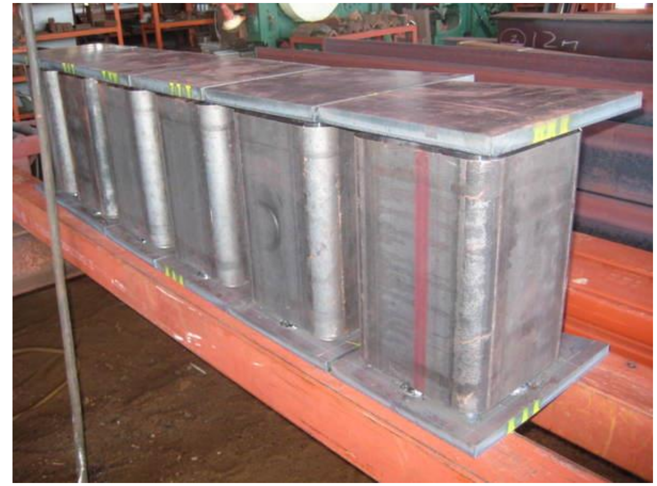
Continuity Plates

SHS for Panel Zone

Typical Beam-to-Column Connection



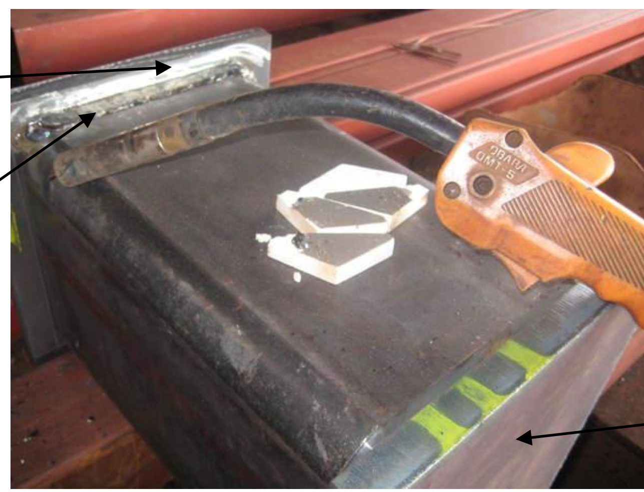
Panel Zone



Pre-Assembled Panel Zones

Continuity Plate (Diaphragm)

CJP



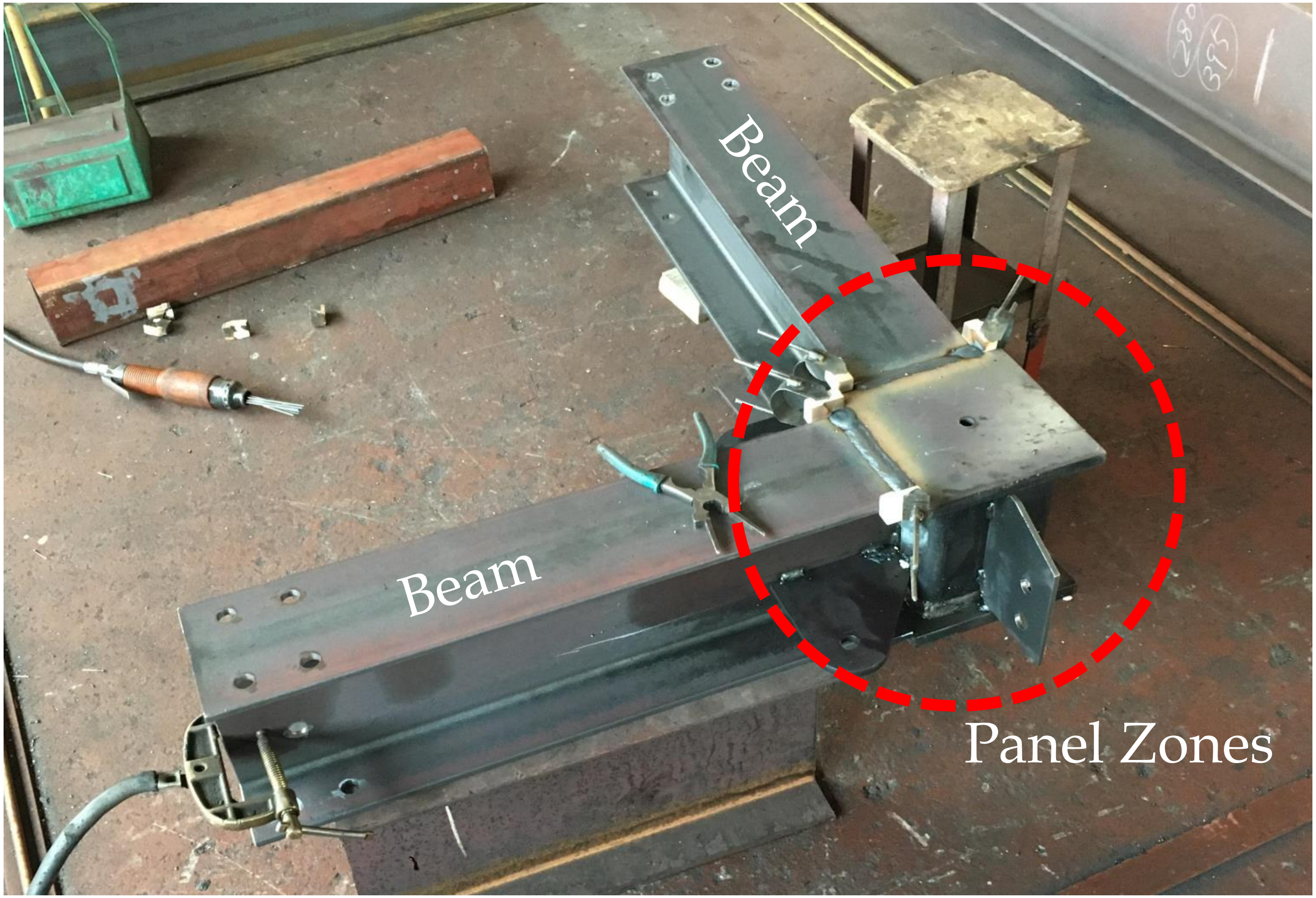
Continuity Plate (Diaphragm)

Panel Zone Assemblage (Dice)

- Complete Joint Penetration (CJP) is used.

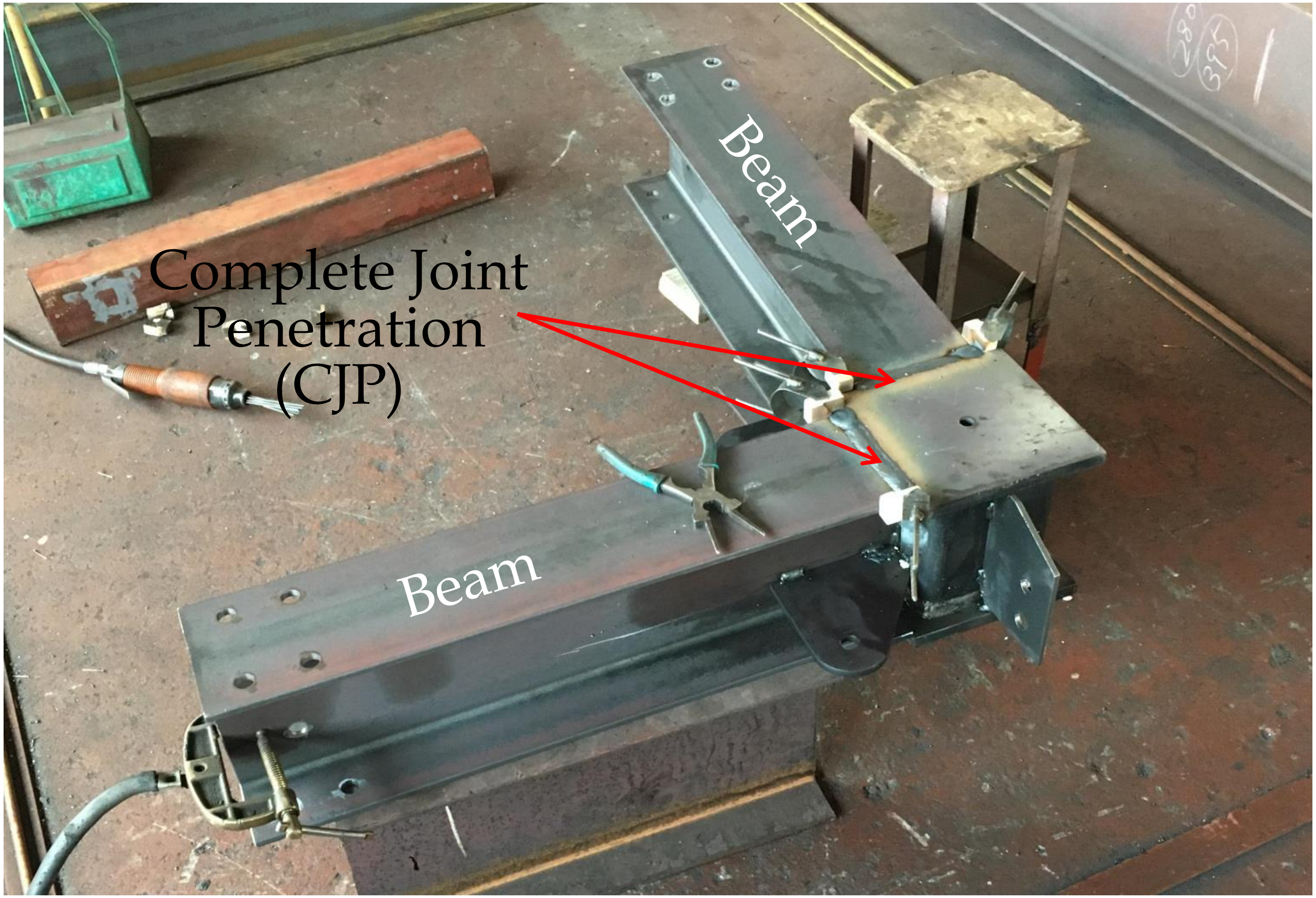
Typical Beam-to-Column Connection

- Shop Welded Detail (Beam-to-Column Connection)



Typical Beam-to-Column Connection

- Shop Welded Detail (Beam-to-Column Connection)



Typical Beam-to-Column Connection

Shop Welded Detail
- ✓ Column Tree
- ✓ CJP
- ✓ UT inspection

Ultrasonic Test Inspection

Typical Beam-to-Column Connection

Transport to Site

High strength bolts are used for Beam splice joints

Assemble Moment Frame

Beam-to-Column Connections

- Beam Joints
  - Assumed to be rigid and beam is expected to be the dissipative zones at Ultimate limit State (ULS).
    → Consistent with BSL
  - Capacity design. Following should be satisfied.

$$\alpha \cdot {}_b M_p \leq {}_j M_u$$

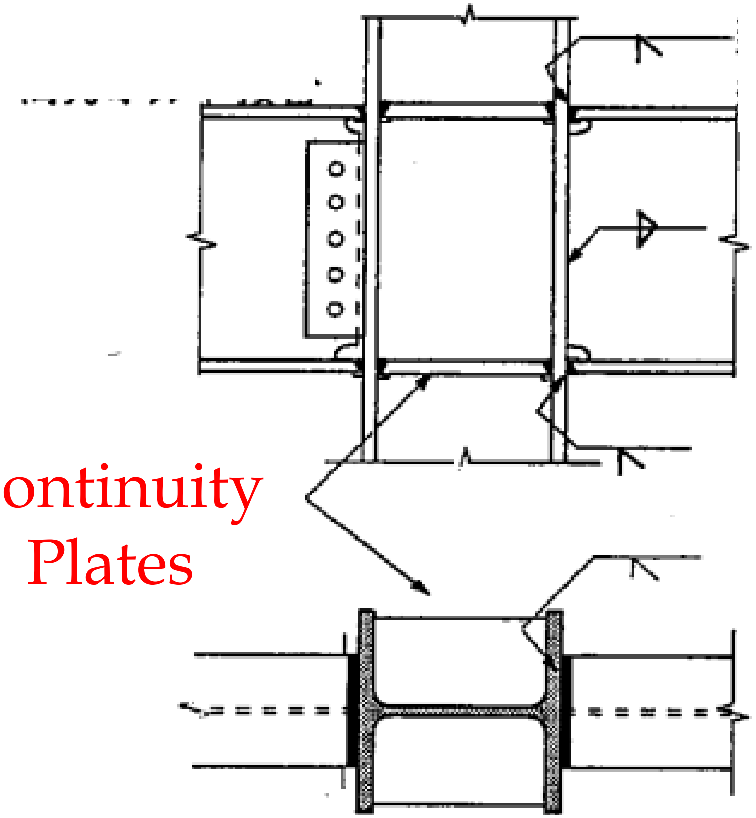
Maximum Strength of the Beam Joint

Full Plastic moment of the Beam

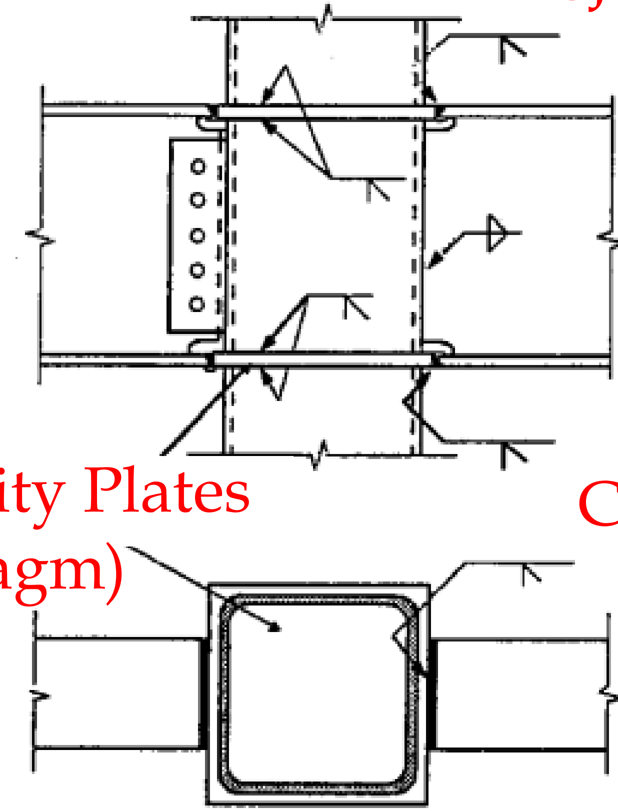
Beam Joint Coefficient. Considering Hardening and Strength Randomness. Depending on Steel Grades.

α: SS400 1.40, SM490 1.35, SN400B 1.30, and SN490B 1.25

Rigid Joints (to be consistent with BSL)

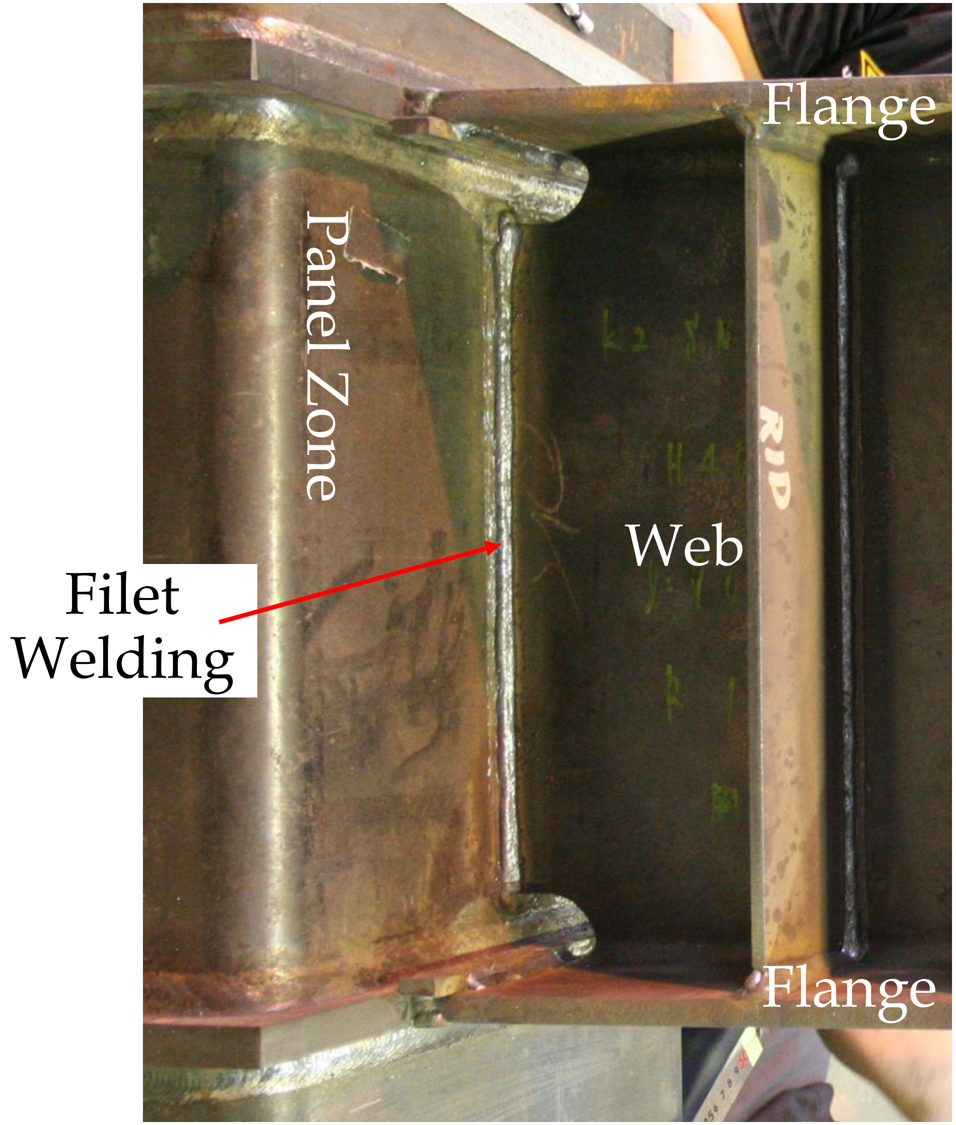


H section Column
<not common>

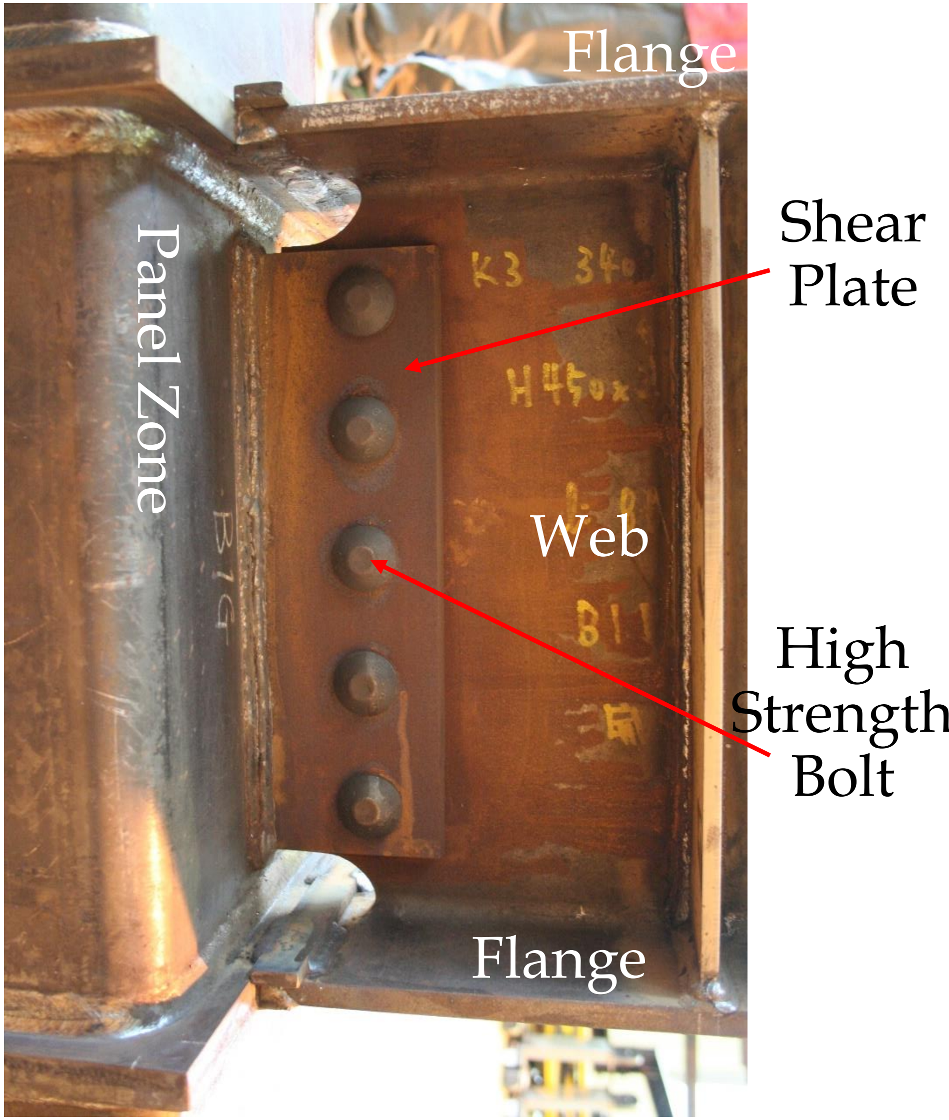


Hollow section Column
<Typical>

Difference Between Shop and Field



Shop Welding



Field Welding

Maximum Strength of the Beam Joint

- For Rigid Joints at ULS

$$\alpha \cdot {}_bM_p \leq {}_jM_u$$

<Beam>

$$\alpha \cdot {}_bM_p = \alpha \cdot Z_p \cdot F_{by}$$

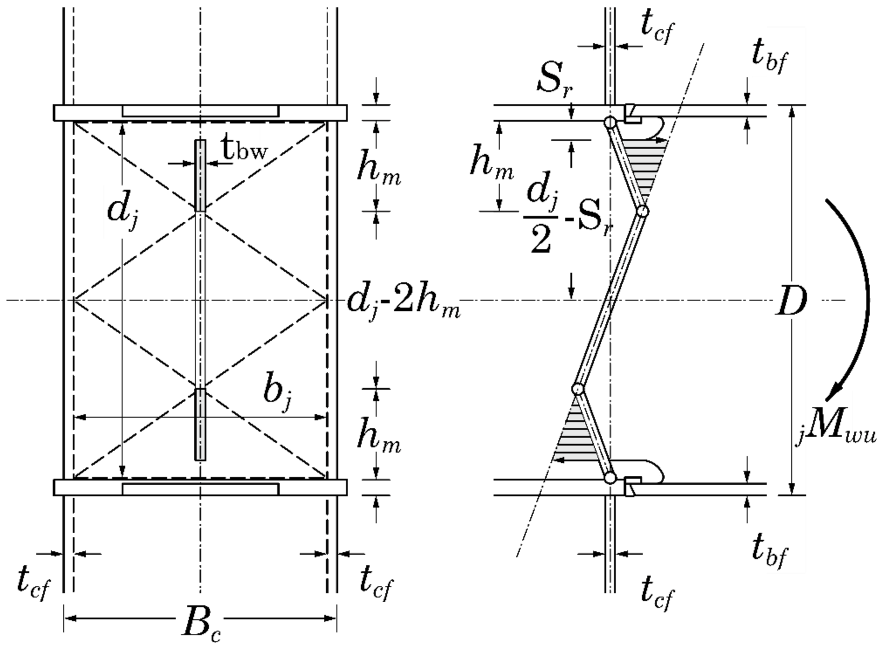
<Beam Joint Strength>

$${}_jM_u = {}_jM_{ju} + {}_jM_{wu}$$

<flange> $${}_jM_{fu} = A_f \cdot d_b \cdot F_{bu}$$

<web> $${}_jM_{wu} = m \cdot Z_{wpe} \cdot F_{by}$$

<web joint efficiency> $$m = \min\left\{1,\ 4\frac{t_{cf}}{d_j}\sqrt{\frac{b_j}{t_{bw}}\frac{F_{cy}}{F_{by}}}\right\}$$



<effective area of beam web>

Recommendation from AIJ

- Semi-rigid joints can be found.
  - Joint should have sufficient rotation capacity.
  - The performance should be predictable.
  - Local deformation at the joint should be considered in the design, e.g. spring model.

  → Numerical model which is used to compute the member forces will be complicated. Structural designers try to avoid complexities.

  ↔ No description in BLS, i.e. seismic action for semi-rigid are not specified. Therefore, designer are not active to use.

- Joints strength can be computed.

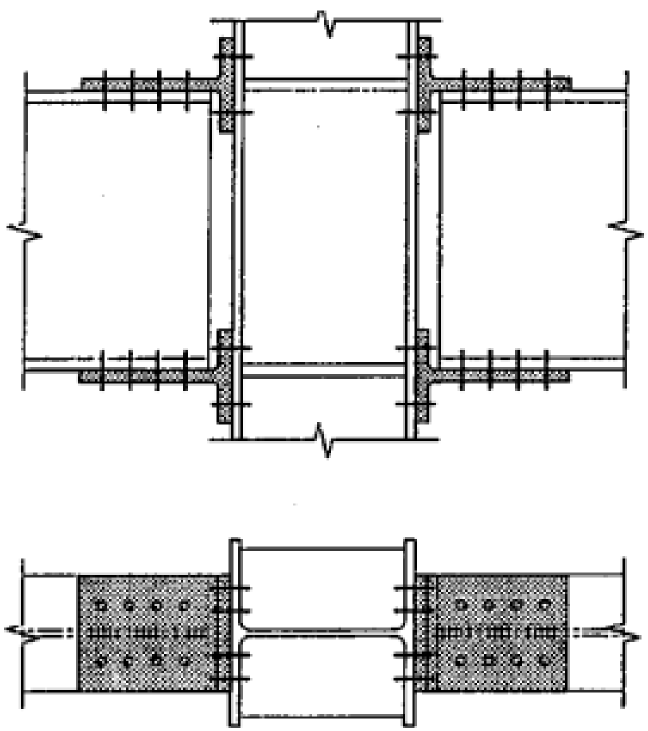
  ↔ Joints are not allowed to be the dissipative zones (BSL)

Recommendation from AIJ

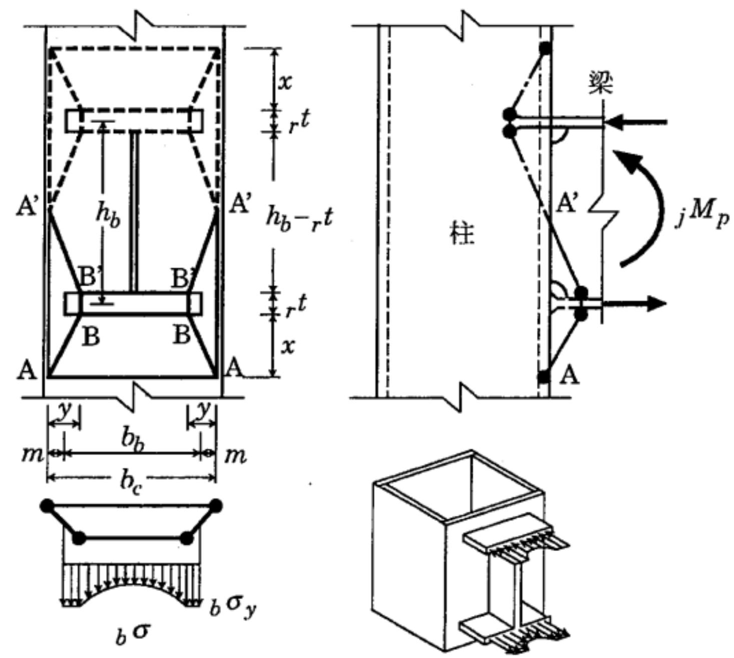
- Joints where the local deformation should be evaluated.

  (in Japan so called Semi-rigid joints. Classification for the joint do not exist)

- Following design formulae are shown
  - Yield Strength, ${}_jM_y$
  - Maximum Strength, ${}_jM_u$
  - Initial Stiffness, K



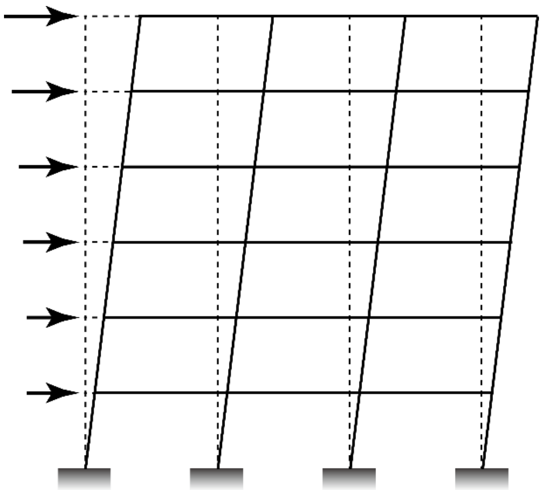
T-stub Joints



Topics -Steel Structure-

- Design Procedure (General)
  - History and Concept
- Design of Beam-to-Column Connection
- Design of Column
- Recent Research Topics in my Group
  - Column
  - Beam-to-Column Connection

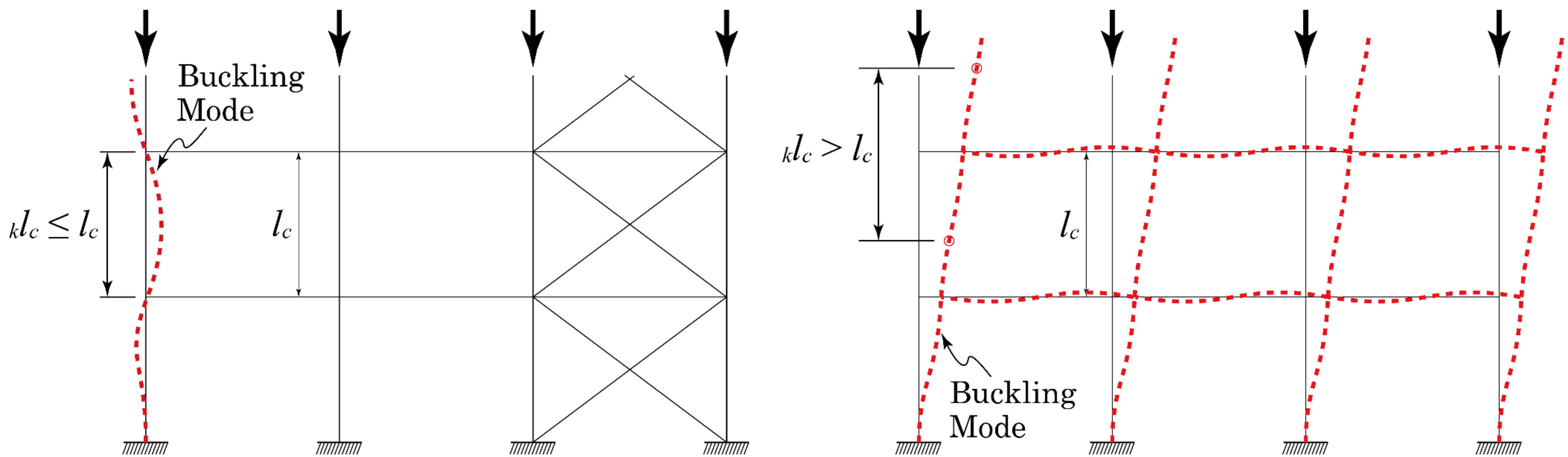
Design of Column



Flexural Buckling Length ${}_kl_c$

- Eigenvalue Analysis;
- Calculation Method base on relevant member stiffness.

【Under **Gravity Load Condition**】



(a) Without Sway (b) With Sway

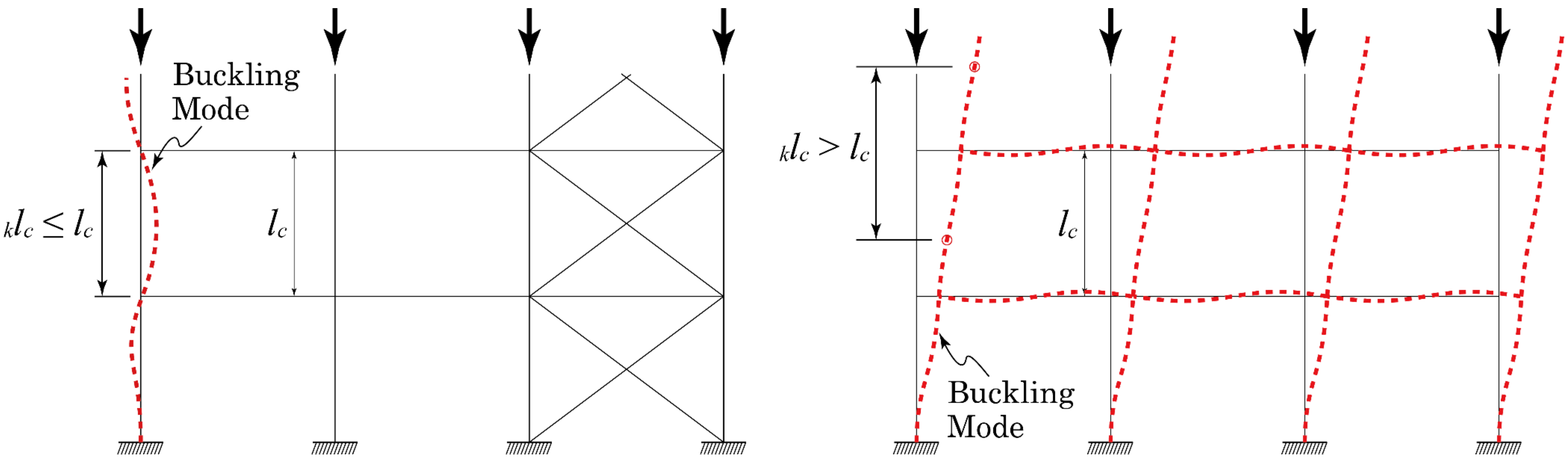
$${}_kl_c \leq l_c \qquad {}_kl_c > l_c$$

Column Length

Flexural Buckling Length $_kl_c$

- Calculation Method base on relevant member stiffness

【Under **Gravity Load Condition**】



(a) Without Sway (b) With Sway

$$_kl_c \leq \underline{l_c} \qquad\qquad _kl_c > \underline{l_c}$$

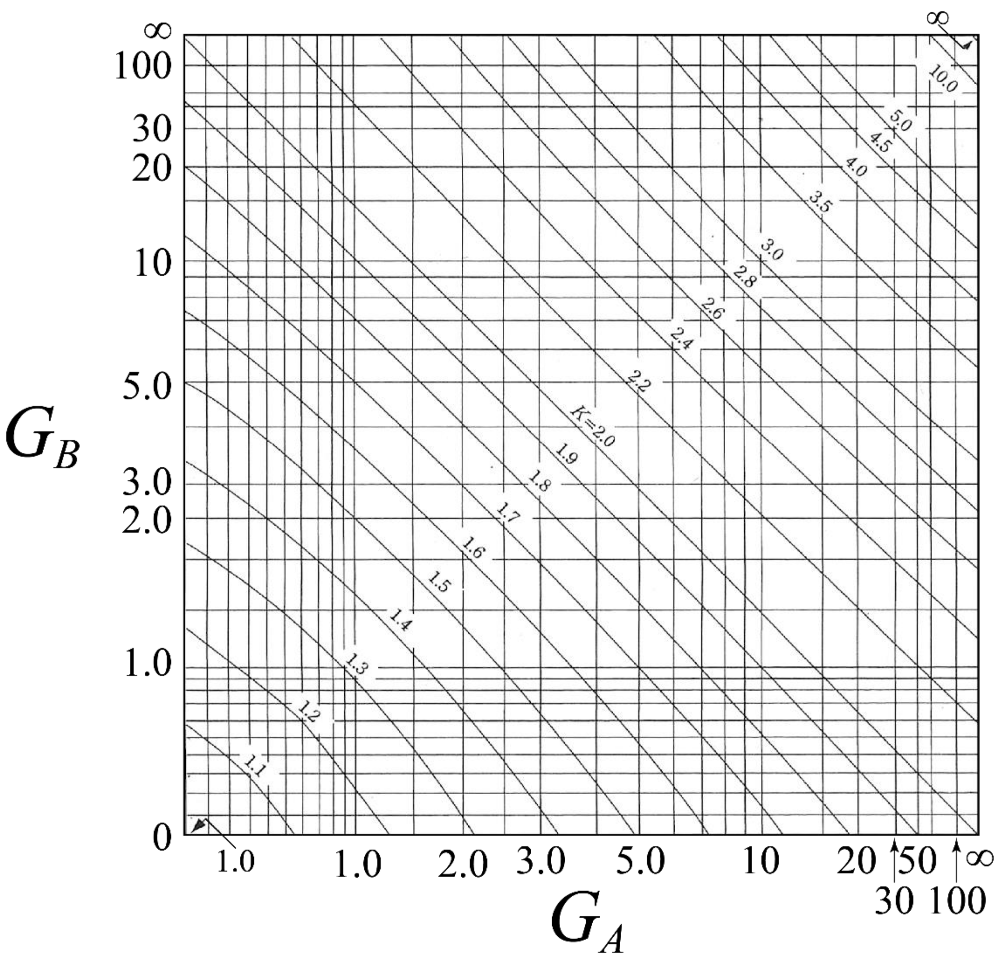
Column Length

【Flexural Buckling Length】 $_kl_c = \underline{k_c} \cdot l_c$

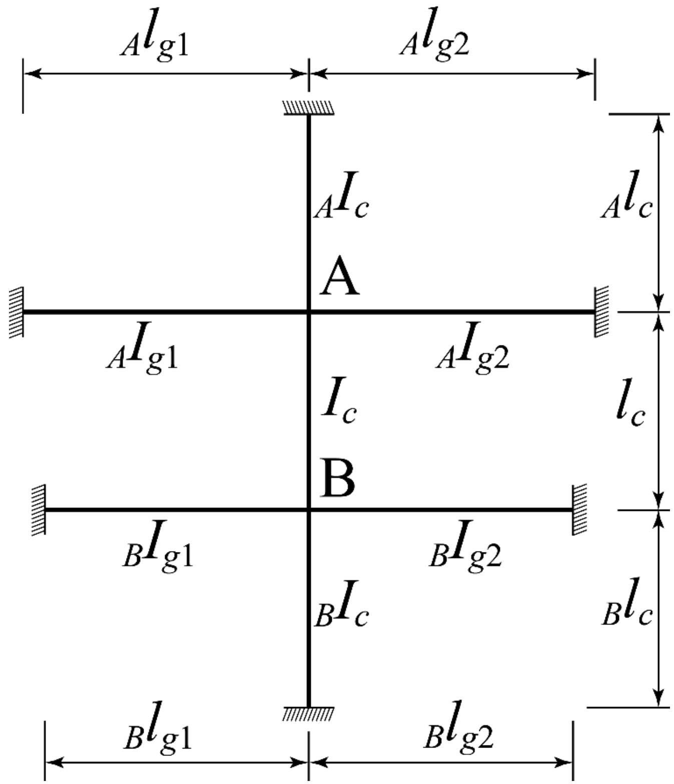
k_c: effective length factor

Flexural Buckling Length $_k l_c$

- Design Table (with sway)



Sway Buckling Mode



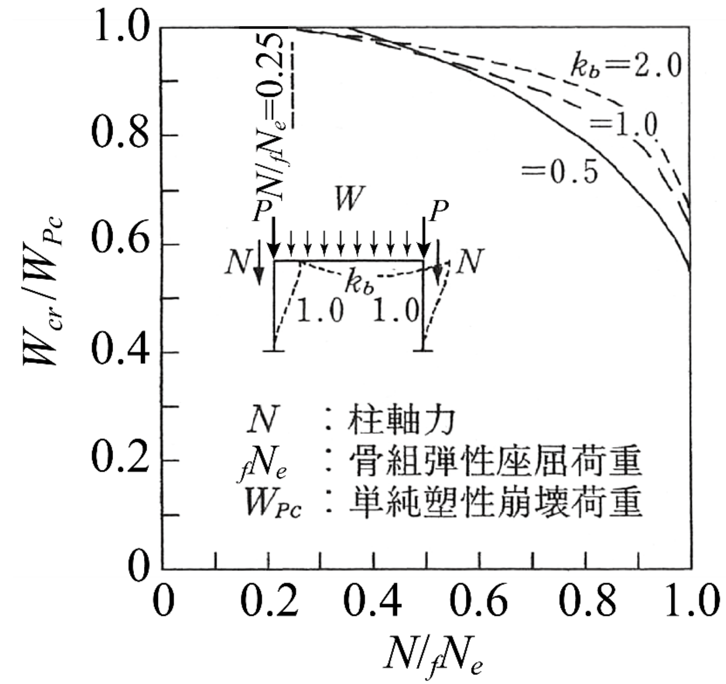
$$G_A = \frac{(I_c/l_c) + ({}_A I_c/{}_A l_c)}{({}_A I_{g1}/{}_A l_{g1}) + ({}_A I_{g2}/{}_A l_{g2})}$$

$$G_B = \frac{(I_c/l_c) + ({}_B I_c/{}_B l_c)}{({}_B I_{g1}/{}_B l_{g1}) + ({}_B I_{g2}/{}_B l_{g2})}$$

Frame Stability

（1）Combination of Compressive axial and slenderness

$$\left(\frac{N}{N_Y}\right)\cdot {}_f\lambda_c^{\,2} \leq 0.25$$



（2）Maximum Compressive Axial force

$$\frac{N}{N_Y} \leq 0.75$$

【Symbol】

In-plane non-dimensional slenderness ratio

$${}_f\lambda_c = \sqrt{N_Y / {}_fN_e}$$

In-plane elastic buckling strength

$${}_fN_e = \frac{\pi^2 \cdot E \cdot I}{{}_kl_c^{\,2}} = \frac{\pi^2 \cdot E \cdot I}{(k_c \cdot l_c)^2}$$

N : Compressive Axial force N_Y : Axial Yield Strength

Column Stability (class 1 cross-section)

Limitation for the column which will form Plastic Hinge

(1) Combination of Compressive axial and slenderness

(a) $-0.5 < \kappa \leq 1.0$

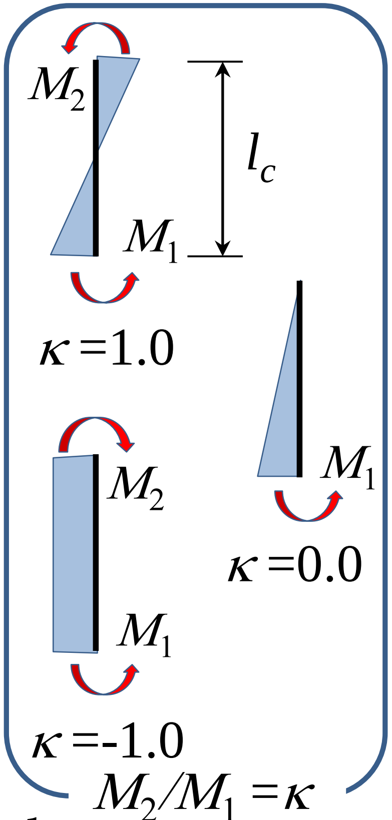
$$\left(\frac{N}{N_Y}\right)\cdot \lambda_{c0}{}^{2} \leq 0.1\cdot(1+\kappa)$$

(b) $-1.0 \leq \kappa \leq -0.5$

$$\left(\frac{N}{N_Y}\right)\cdot \lambda_{c0}{}^{2} \leq 0.05$$

$\kappa = M_2/M_1$

Positive for double curvature bending



【Symbol】

Non-dimensional slenderness ratio

$$\lambda_{c0} = \sqrt{N_Y / N_0}$$

Euler's buckling Strength

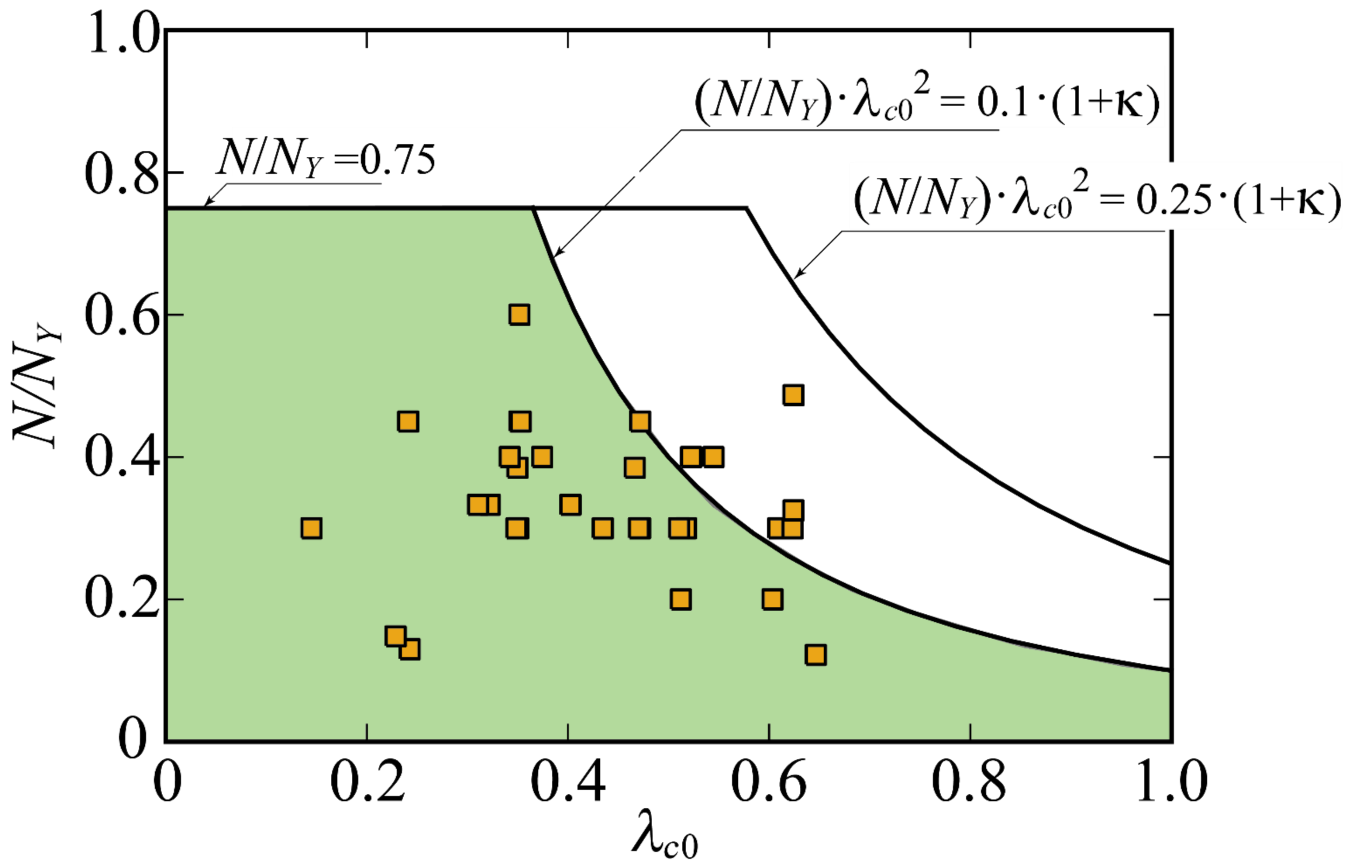
$$N_0 = \frac{\pi^2 \cdot E \cdot I}{l_c{}^2}$$

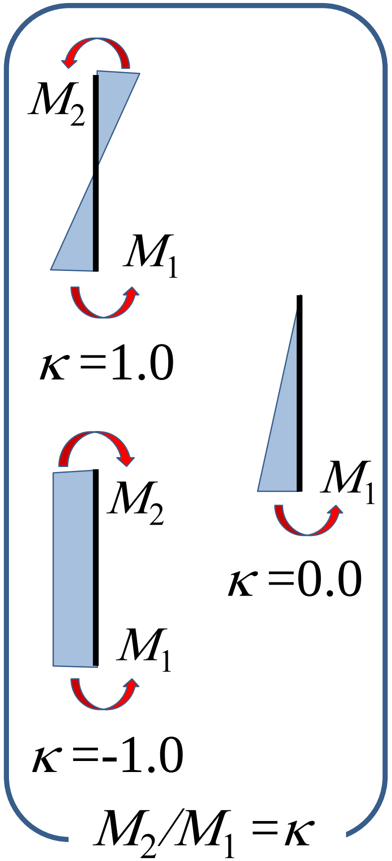
Column Length

Column Stability (class 1 cross-section)

Limitation for the column which will form Plastic Hinge

（1）Combination of Compressive axial and slenderness



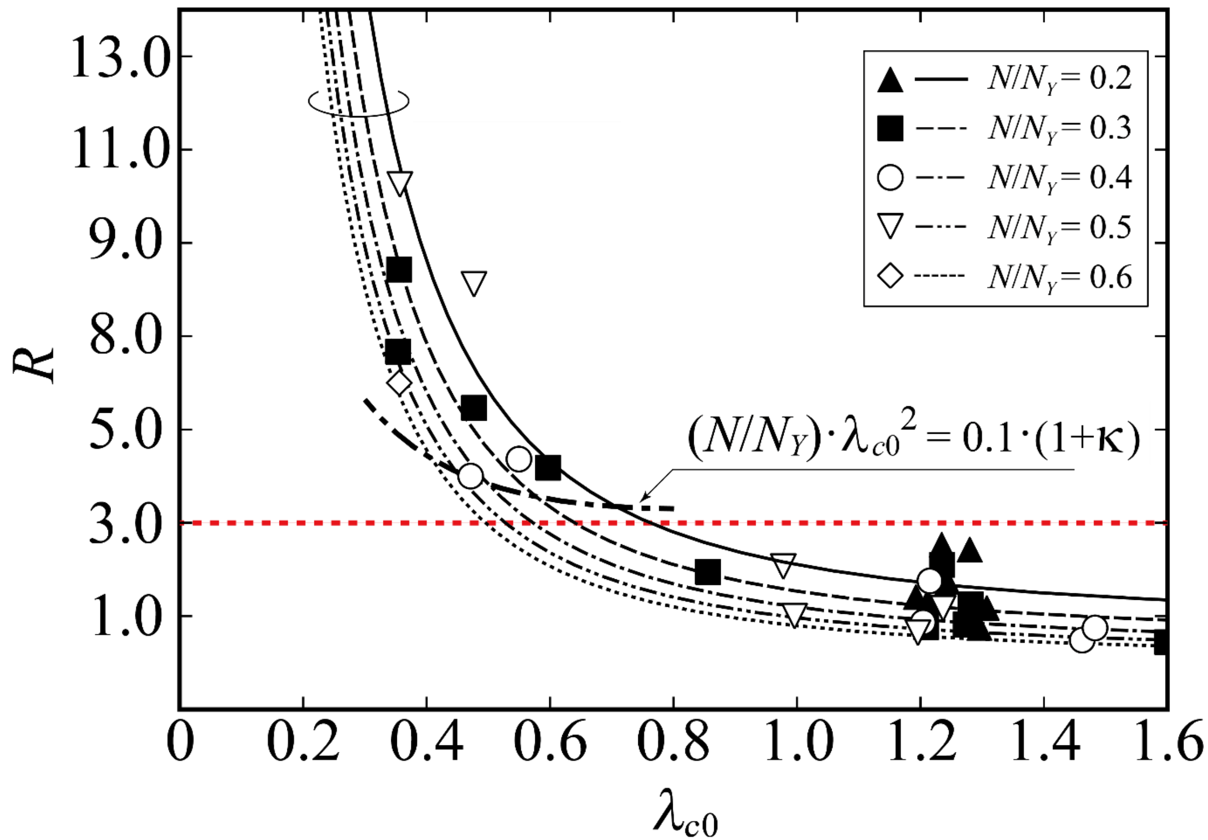


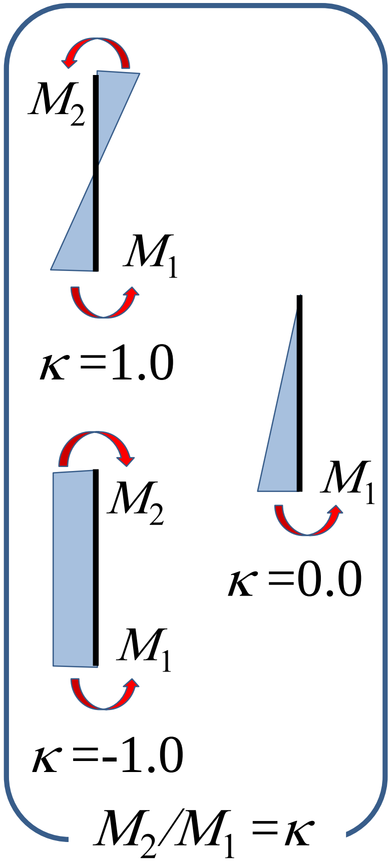
Comparison between test results and limitations (κ=0)

Column Stability (class 1 cross-section)

Limitation for the column which will form Plastic Hinge

（1）Combination of Compressive axial and slenderness





Deformation Capacity (κ=0)

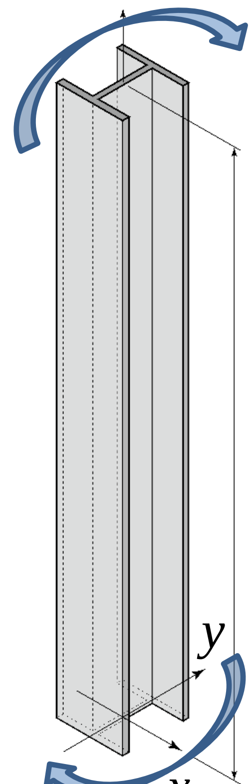
Column Stability (class 1 cross-section)

Limitation for the column which will form Plastic Hinge

(2) Wide Flange Section subjected to strong axis bending

Limitation of torsional-flexural non-dimensional slenderness ratio

$$\lambda_b \le 0.75 \cdot {}_p\lambda_b$$



【Symbol】

λ_b: torsional-flexural non-dimensional slenderness ratio

$$\lambda_b = \sqrt{M_P / M_e}$$

$$M_e = C_b \sqrt{\frac{\pi^2 \cdot E \cdot I_y \cdot G \cdot J_T}{l_c^{\,2}} + \frac{\pi^4 \cdot E \cdot I_y \cdot G \cdot I_W}{{}_k l_c^{\,2}}}$$

$$C_b = 1.75 + 1.05 \cdot \kappa + 0.3 \cdot \kappa^2 \le 2.3$$

${}_p\lambda_b$: Plastic Limit (plateau)

$$ {}_p\lambda_b = 0.6 + 0.3 \cdot \kappa$$

$\kappa = M_2 / M_1$

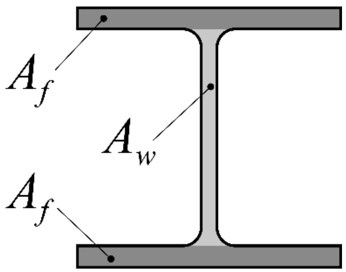
Positive for double curvature

Resistance

- Resistance of Column under combined loading

(1) Wide Flange Section

(a) Under Strong Axis bending



i) Fulfill **Column Stability**→Full Strength (M_{Pc})

$$\frac{N}{N_Y} + \frac{4 \cdot A_f + A_w}{2 \cdot A} \cdot \frac{M}{M_P} = 1.0$$

ii) $\lambda_b \leq {}_p\lambda_b$ (in-plane)

$$\frac{N}{N_{cr}} + \varphi \cdot \frac{4 \cdot A_f + A_w}{2 \cdot A} \cdot \frac{M}{M_P} = 1.0 \quad , \qquad \frac{M}{M_{Pc}} \leq 1.0$$

iii) $\lambda_b \geq {}_p\lambda_b$ (out-of-plane)

$$\frac{N}{N_{cr,y}} + \frac{4 \cdot A_f + A_w}{2 \cdot A} \cdot \frac{M}{M_{cr}} = 1.0 \quad , \qquad \frac{M}{M_{cr}} \leq 1.0$$

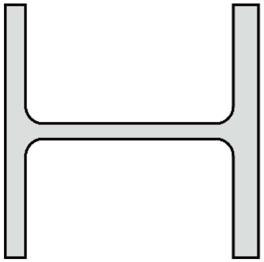
Resistance

- Resistance of Column under combined loading

（1）Wide Flange Section (cont.)

(a) Under Weak Axis bending

i) Fulfill **Column Stability**→Full Strength (M_{Pc})



$$\left(\frac{N-N_{wY}}{N_Y-N_{wY}}\right)^2+\frac{M}{M_P}=1.0$$

ii) others

$$\left(\frac{N-N_{wY}}{N_Y-N_{wY}}\right)^2+\varphi\cdot\frac{M}{M_P}=1.0\ ,\qquad \frac{N}{N_{cr}}\leq 1.0$$

【Symbol】

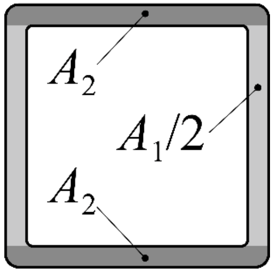
φ: Coefficient to evaluate $P\delta$ effects (Second order effects)

N_{wY}: Yield strength of web

Resistance

(2) Rectangular (Square) Hollow Section

i) Fulfill **Column Stability**→Full Strength (M_{Pc})



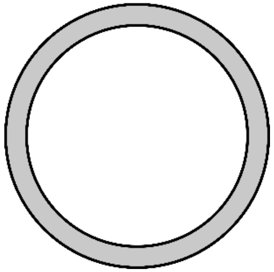
$$\frac{N}{N_Y} + \frac{4 \cdot A_2 + A_1}{2 \cdot A} \cdot \frac{M}{M_P} = 1.0$$

ii) others

$$\frac{N}{N_{cr}} + \varphi \cdot \frac{4 \cdot A_2 + A_1}{2 \cdot A} \cdot \frac{M}{M_P} = 1.0 \quad , \quad \frac{M}{M_{Pc}} \leq 1.0$$

(3) Circular Hollow Section

i) Fulfill **Column Stability**→Full Strength (M_{Pc})



$$\frac{N}{N_Y} + 0.80 \cdot \frac{M}{M_P} = 1.0$$

ii) others

$$\frac{N}{N_{cr}} + \varphi \cdot 0.80 \cdot \frac{M}{M_P} = 1.0 \quad , \quad \frac{M}{M_{Pc}} \leq 1.0$$

Resistance

- Coefficient to evaluate $P\delta$ effects (Second order effects) φ

$$(N/N_Y)\cdot \lambda_{c0}^{\ 2} \le 0.25(1+\kappa)$$

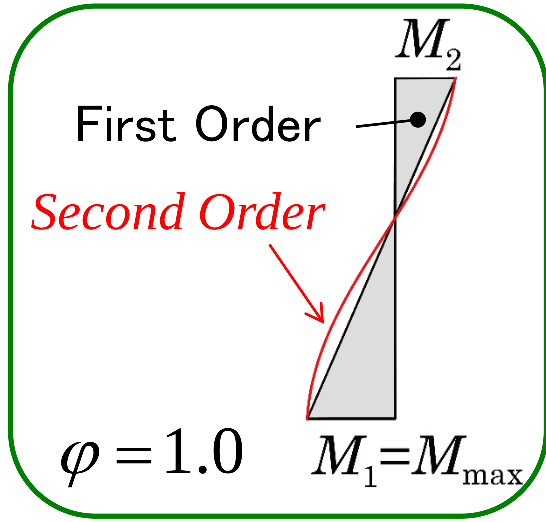
$$\varphi = 1.0$$

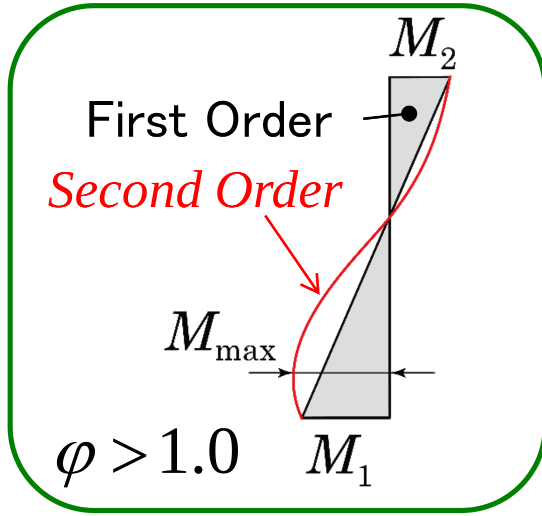
(6.3.6.a)

$$(N/N_Y)\cdot \lambda_{c0}^{\ 2} > 0.25(1+\kappa)$$

$$\varphi = \frac{1-0.5(1+\kappa)\sqrt{N/N_0}}{1-N/N_0} \ge 1.0$$

(6.3.6.b)





【Symbol】

Non-dimensional slenderness ratio

$$\lambda_{c0} = \sqrt{N_Y/N_0}$$

Euler's buckling Strength

$$N_0 = \frac{\pi^2 \cdot E \cdot I}{l_c^{\ 2}}$$

$\kappa = M_2 / M_1$

Positive for Double Curvature bending

Resistance

- Coefficient to evaluate $P\delta$ effects (Second order effects) φ

$$(N/N_Y)\cdot\lambda_{c0}{}^2 \le 0.25(1+\kappa)$$

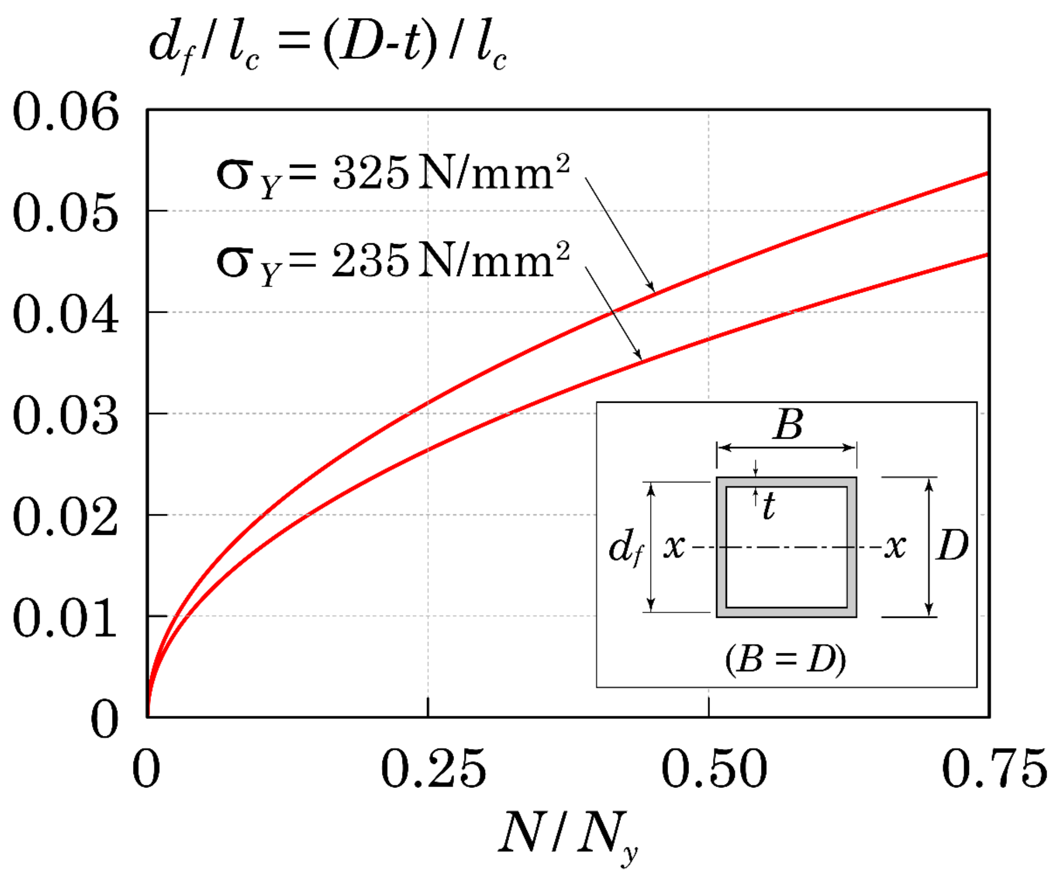
$$\boxed{\kappa = 0}$$

$$\frac{1}{l_c}\sqrt{\frac{I}{A}} \ge \frac{2}{\pi}\sqrt{\frac{N}{N_Y}\cdot\frac{\sigma_Y}{E}}$$

【For Square Hollow Section (approximation)】

$$A = 4\cdot d_f\cdot t$$

$$I_x = I_y = \frac{2}{3}d_f{}^3\cdot t$$



【Symbol】

E:Elastic Modulus, I:Moment of Inertia, A:Area

Topics -Steel Structure-

- Design Procedure (General)
  - History and Concept
- Design of Beam-to-Column Connection
- Design of Column
- Recent Research Topics in my Group
  - Column
  - Beam-to-Column Connection

Research Topics in **Sato Lab.**

- **Steel Structure Research**
  - Seismic Design
    - Heavy Section (Large or Tall Buildings)
    - Light Gauge (Small or Residential Buildings)
  - Structural Member Stability

- **Timber Structure Research**
  - Retrofit of Residential Timber Structure
    - Traditional way
    - Adding two Technique

- **Investigation of New Material Research**
  - Ductile Cast Iron (popular in vehicle Engineering)

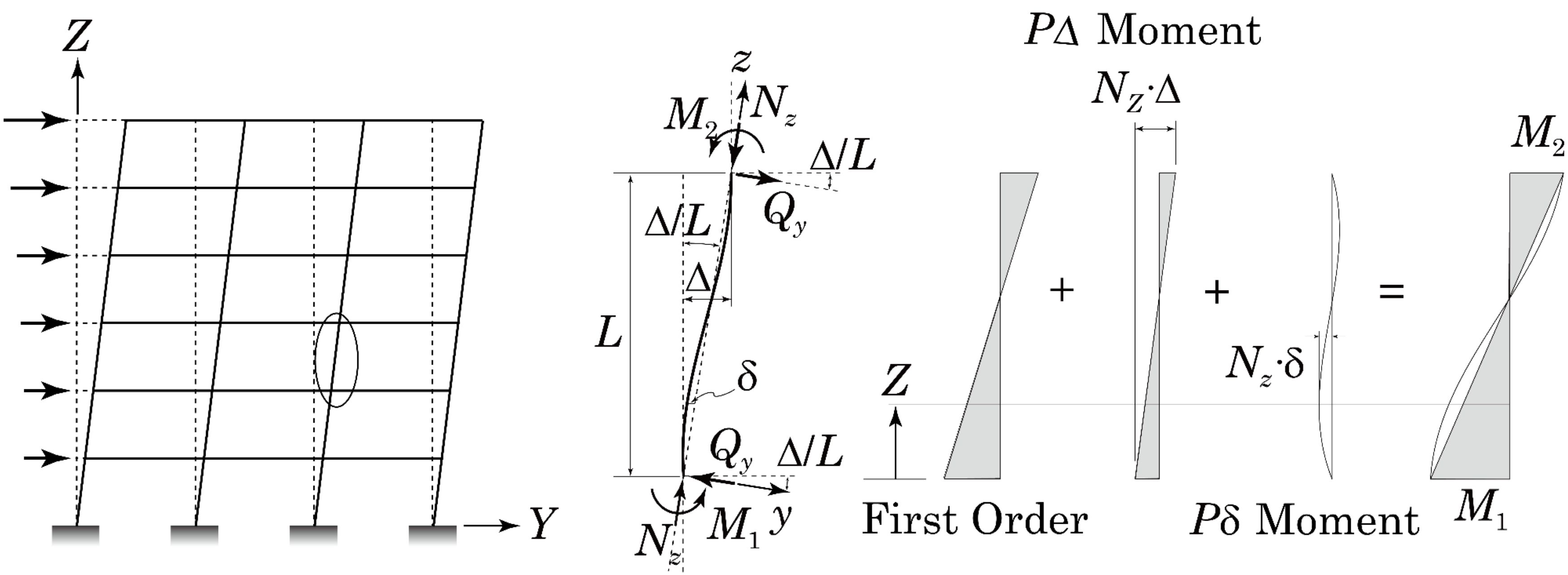
Topics -Steel Structure-

- Design Procedure (General)
  - History and Concept
- Design of Beam-to-Column Connection
- Design of Column
- Recent Research Topics in my Group
  - Column
  - Beam-to-Column Connection

Column
in Steel Structure

Column

- It will support gravity load (**Axial Force ,*N***).
- **Bending Moment** (***M***) will get larger once horizontal force is applied.
- Capacity for Combined Loading (**Axial Force** with **Bending Moment**) is important in a large story drift.



Design Limitation

- *Recommendation for Limit State Design of Steel Structures* (AIJ 2010) specifies the limitation of Axial Force and Slenderness Ratio of the Column to guaranty sufficient ductility.

Maximum Axial Force Limitation

$$\frac{N}{N_y} \le 0.75$$

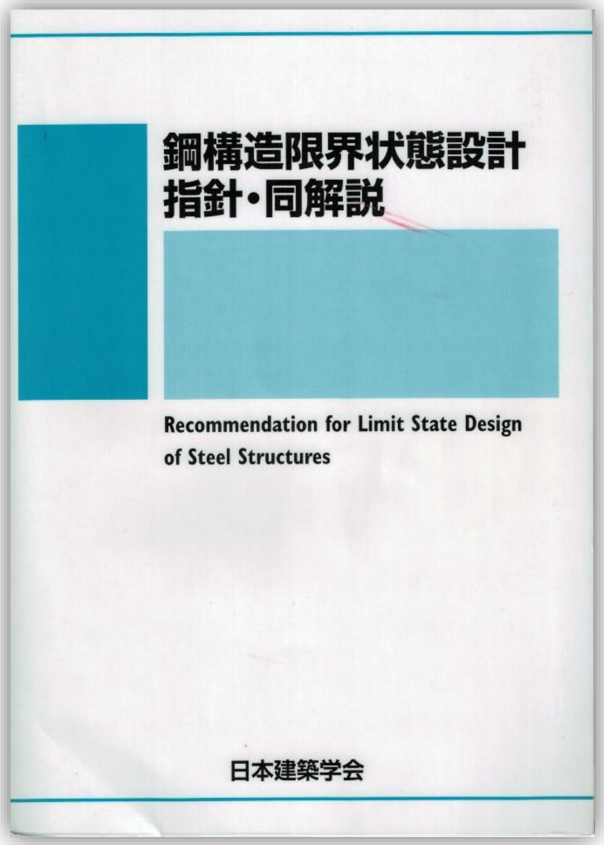
LTB Limitation (only for Wide Flange)

$$\sqrt{\frac{M_p}{M_e}} \le 0.75\left(0.6 + 0.3\frac{M_2}{M_1}\right)$$

Axial Force Ratio

and Slenderness Ratio

$$\frac{N}{N_y}\left(\frac{1}{\pi}\sqrt{\frac{\sigma_y}{E}}\frac{L}{i_x}\right)^2 \le 0.10\left(1 + \frac{M_2}{M_1}\right)$$



Design Limitation

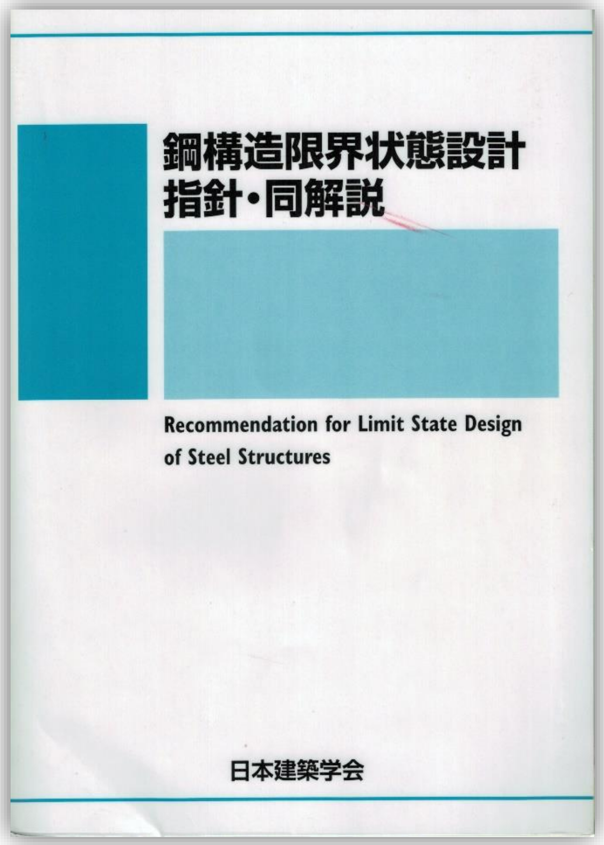
- *Recommendation for Limit State Design of Steel Structures* (AIJ 2010) specifies the limitation of Axial Force and Slenderness Ratio of the Column to guaranty sufficient ductility.

Maximum Moment at the end（elastic derivation）

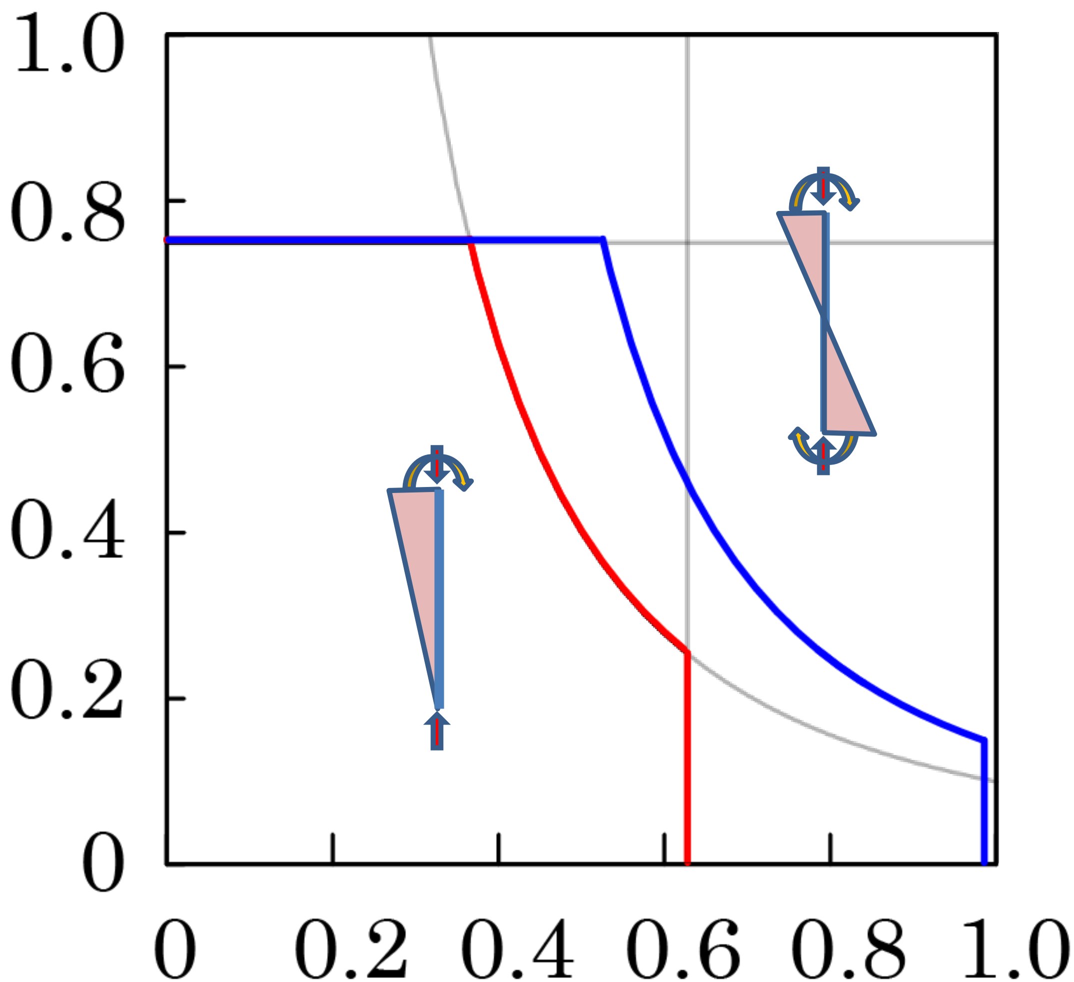
$$\frac{N}{N_y}\left(\frac{1}{\pi}\sqrt{\frac{\sigma_y}{E}}\frac{L}{i_x}\right)^2 \le 0.25\left(1+\frac{M_2}{M_1}\right)$$

Axial Force Ratio and Slenderness Ratio

$$\frac{N}{N_y}\left(\frac{1}{\pi}\sqrt{\frac{\sigma_y}{E}}\frac{L}{i_x}\right)^2 \le 0.10\left(1+\frac{M_2}{M_1}\right)$$

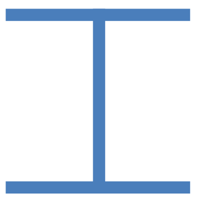


Design Limitation in LSD



$$\frac{N}{N_y}$$

$$\lambda_{c0} = \frac{1}{\pi}\sqrt{\frac{\sigma_y}{E}}\frac{L}{i_x}$$



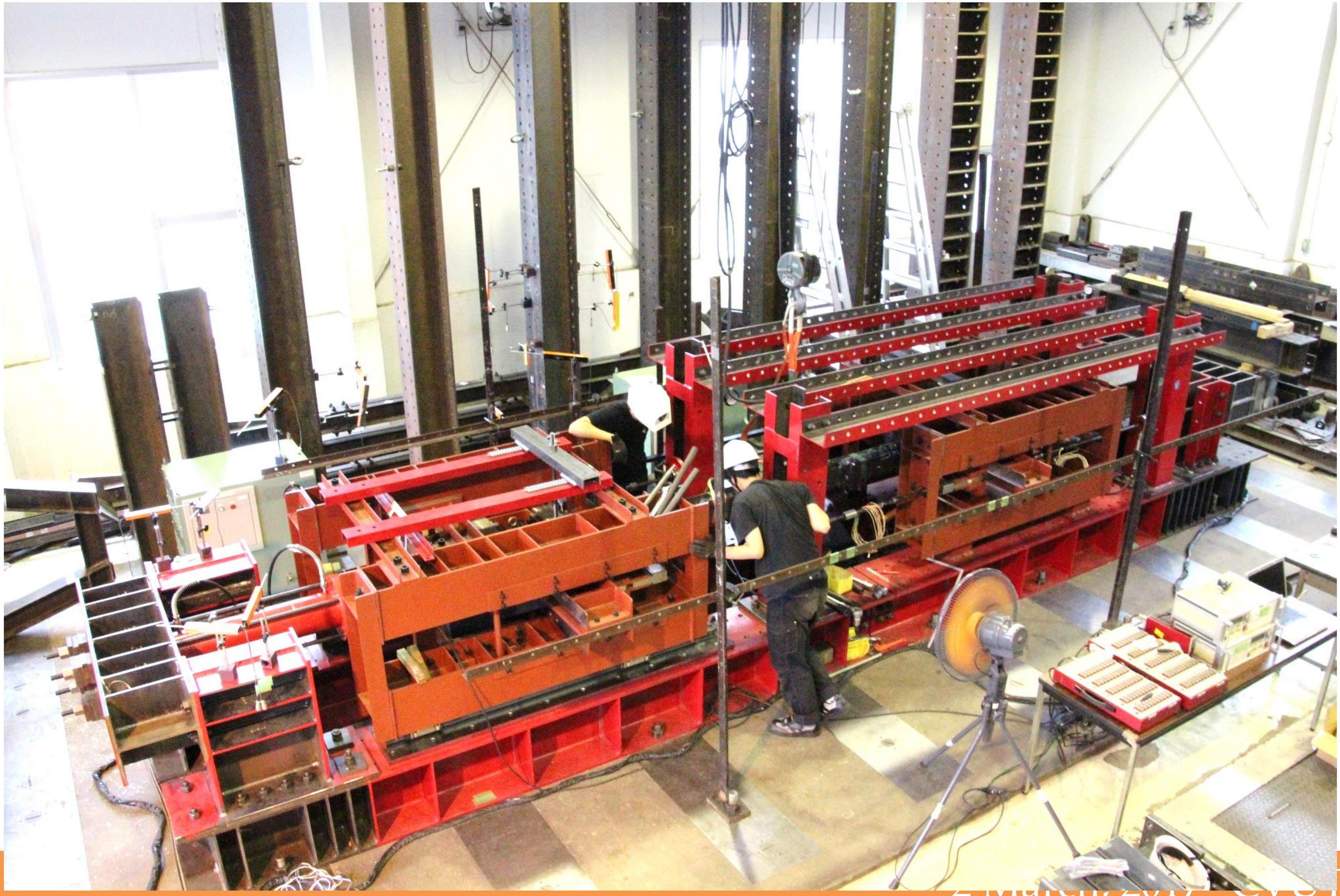
Background TEST DATA

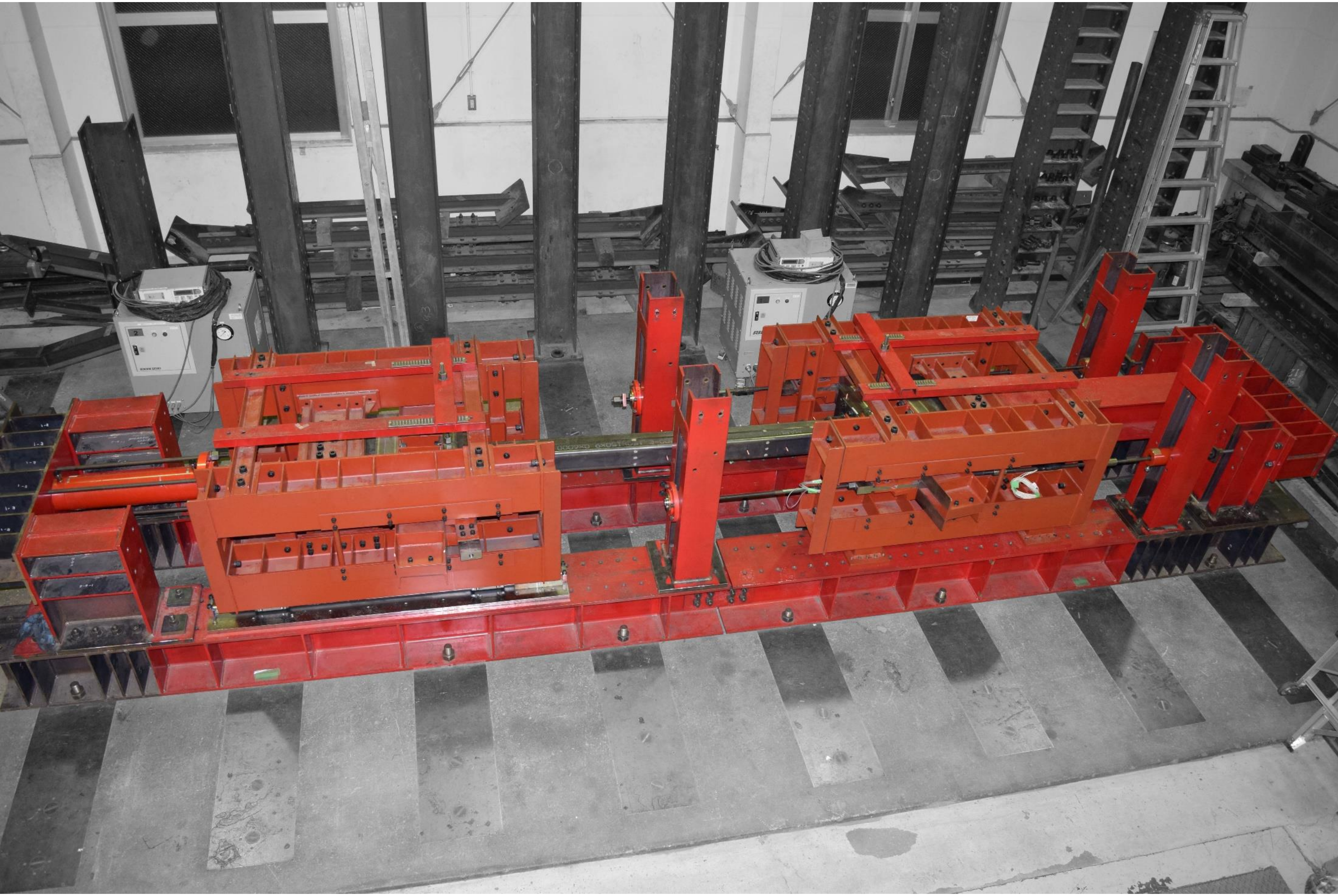
- Most Test data are Single curvature bending moment.
- Wide flange section.
- Monotonic Loading.

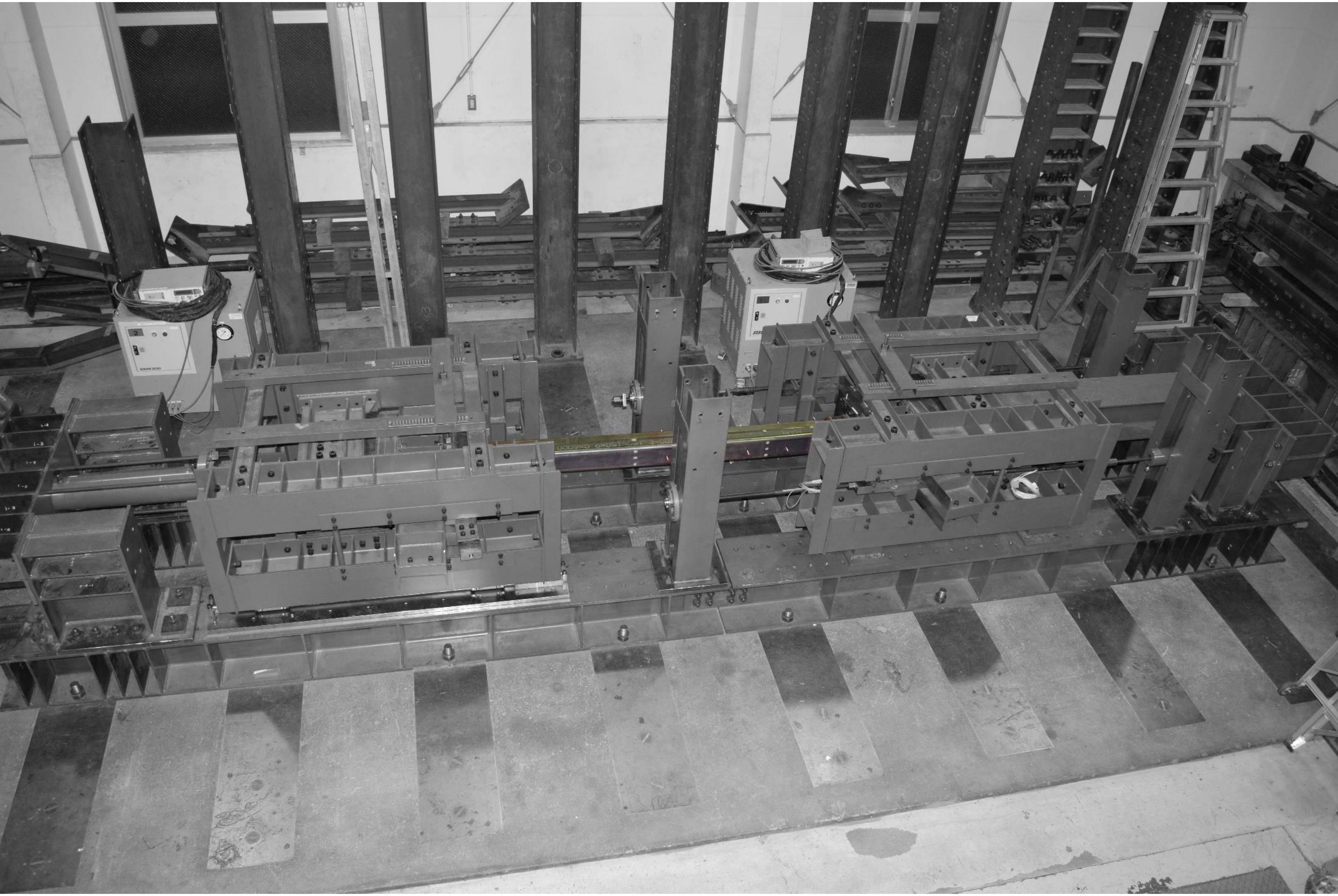
[Not included (Not Considered)]

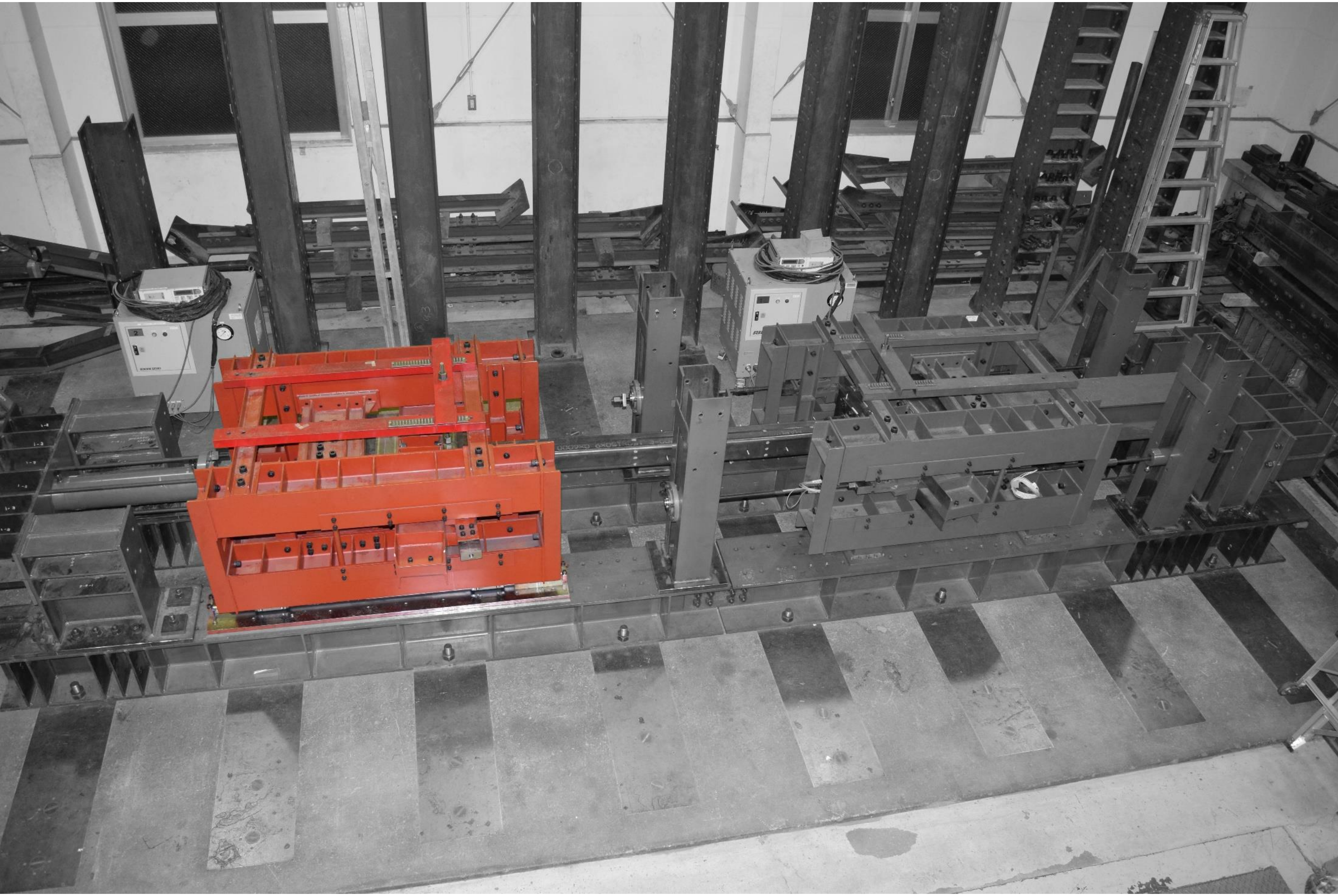
- **Double curvature** bending moment is more realistic.
- **Box section (HSS)** is more popular in Japan.
- Seismic effect will be **Cyclic Loading**.

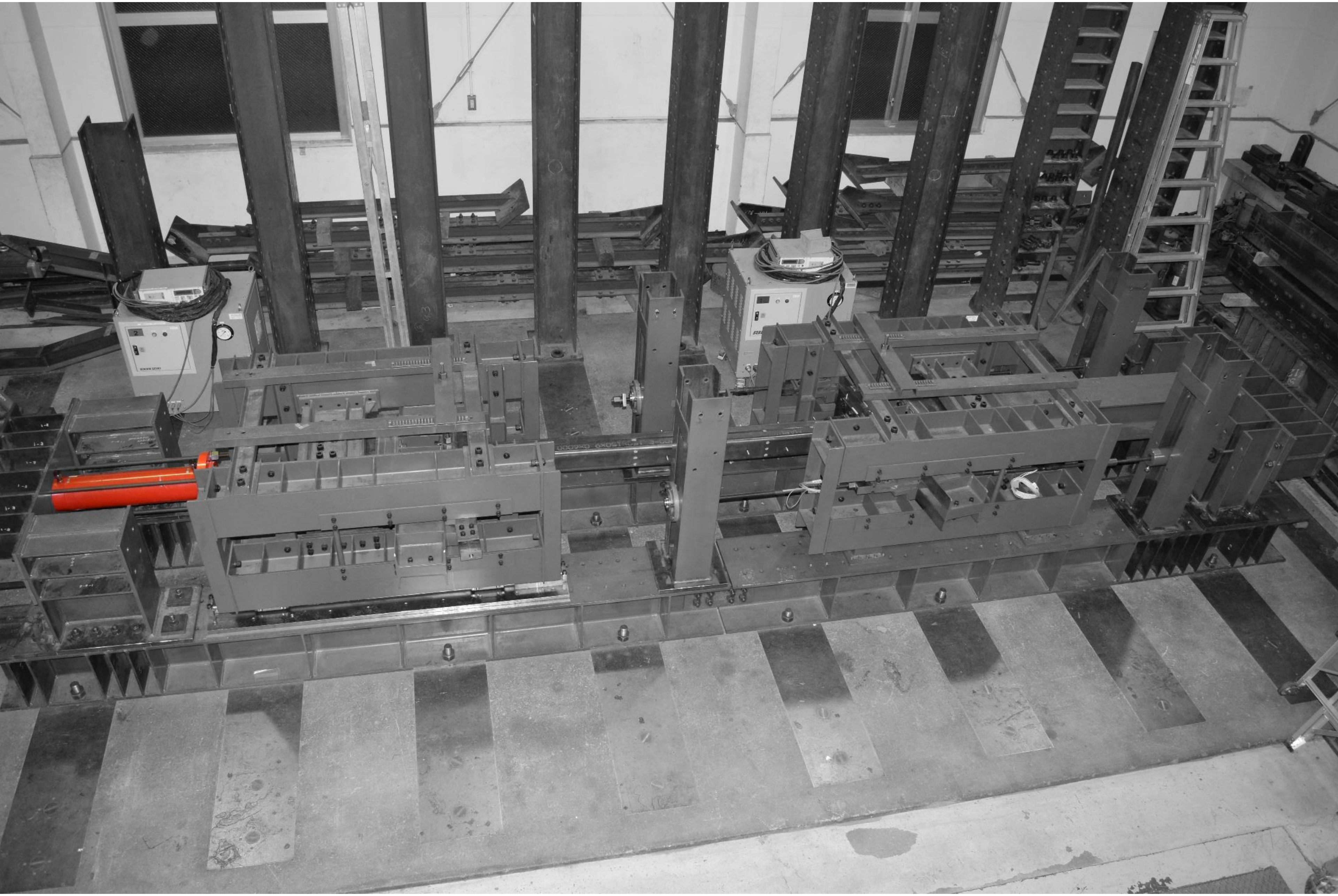
Test Setup (NITech 2015)

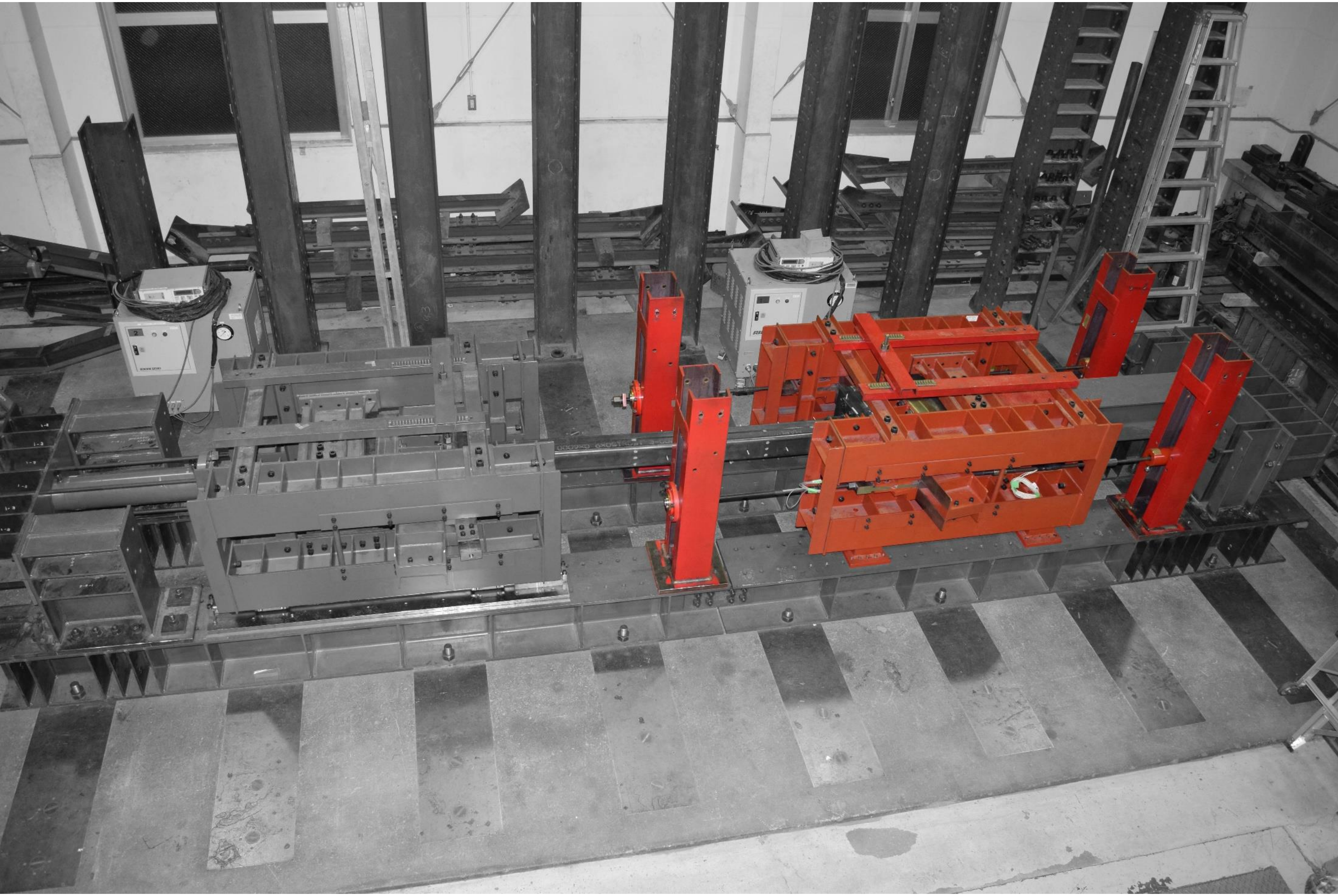


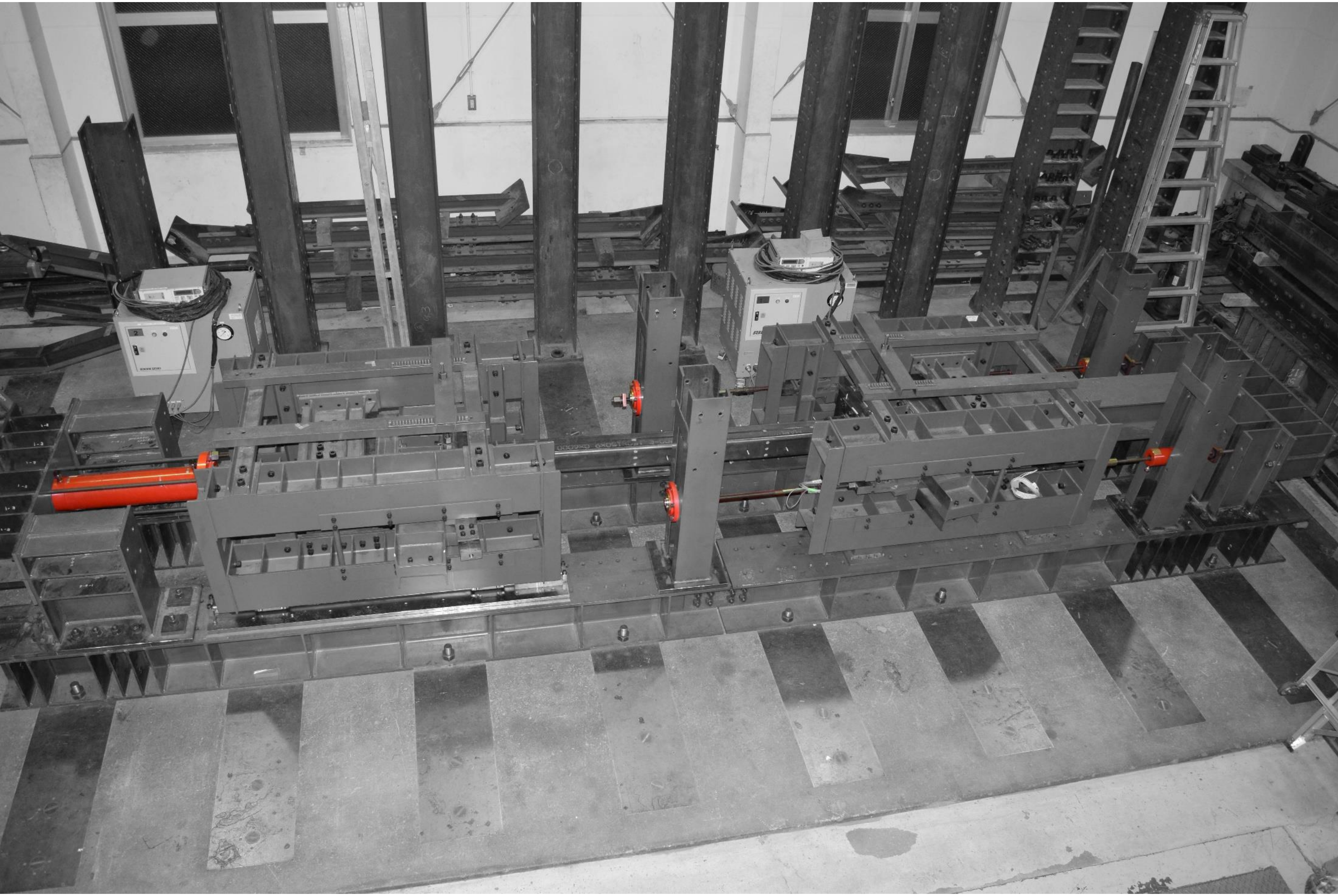


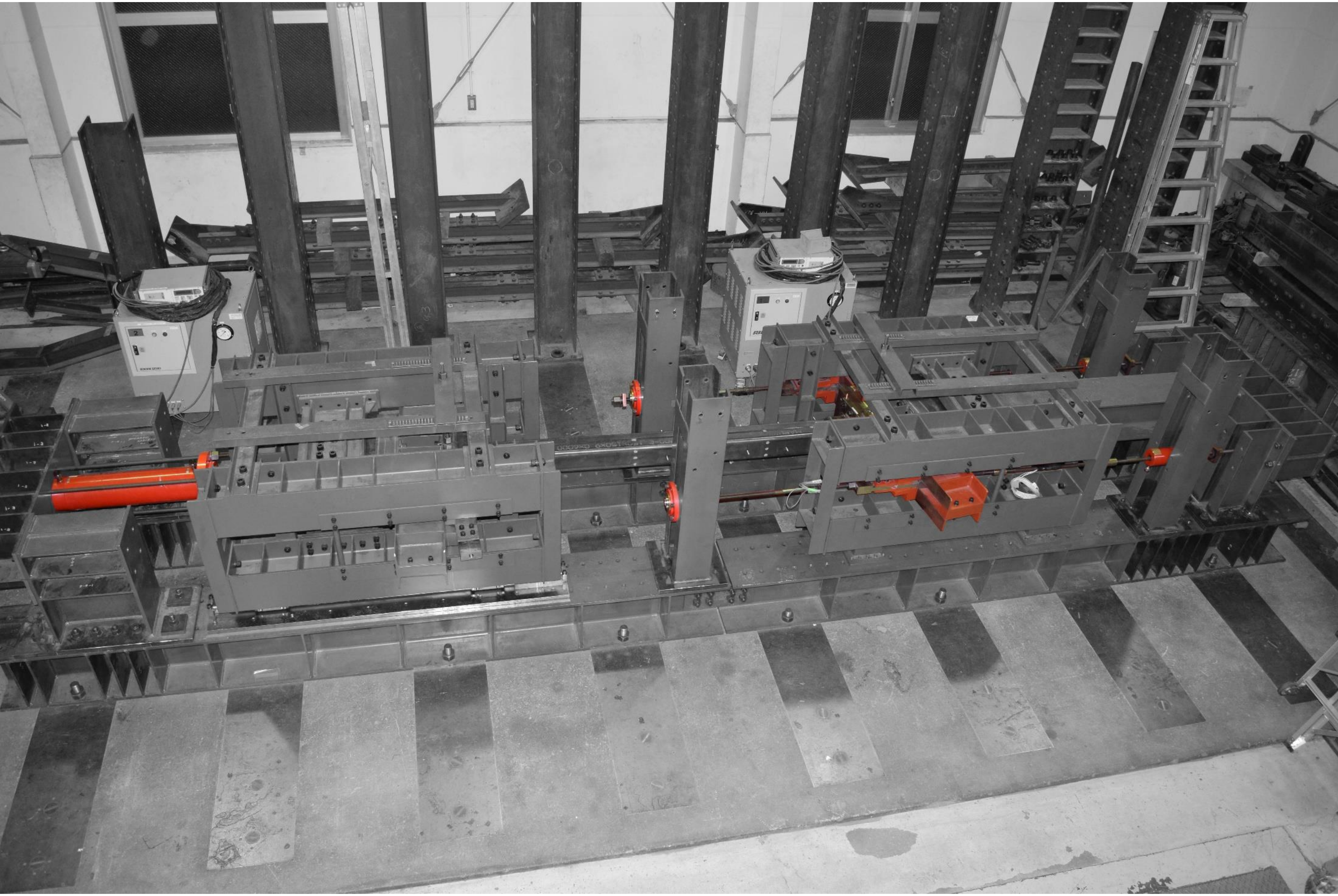


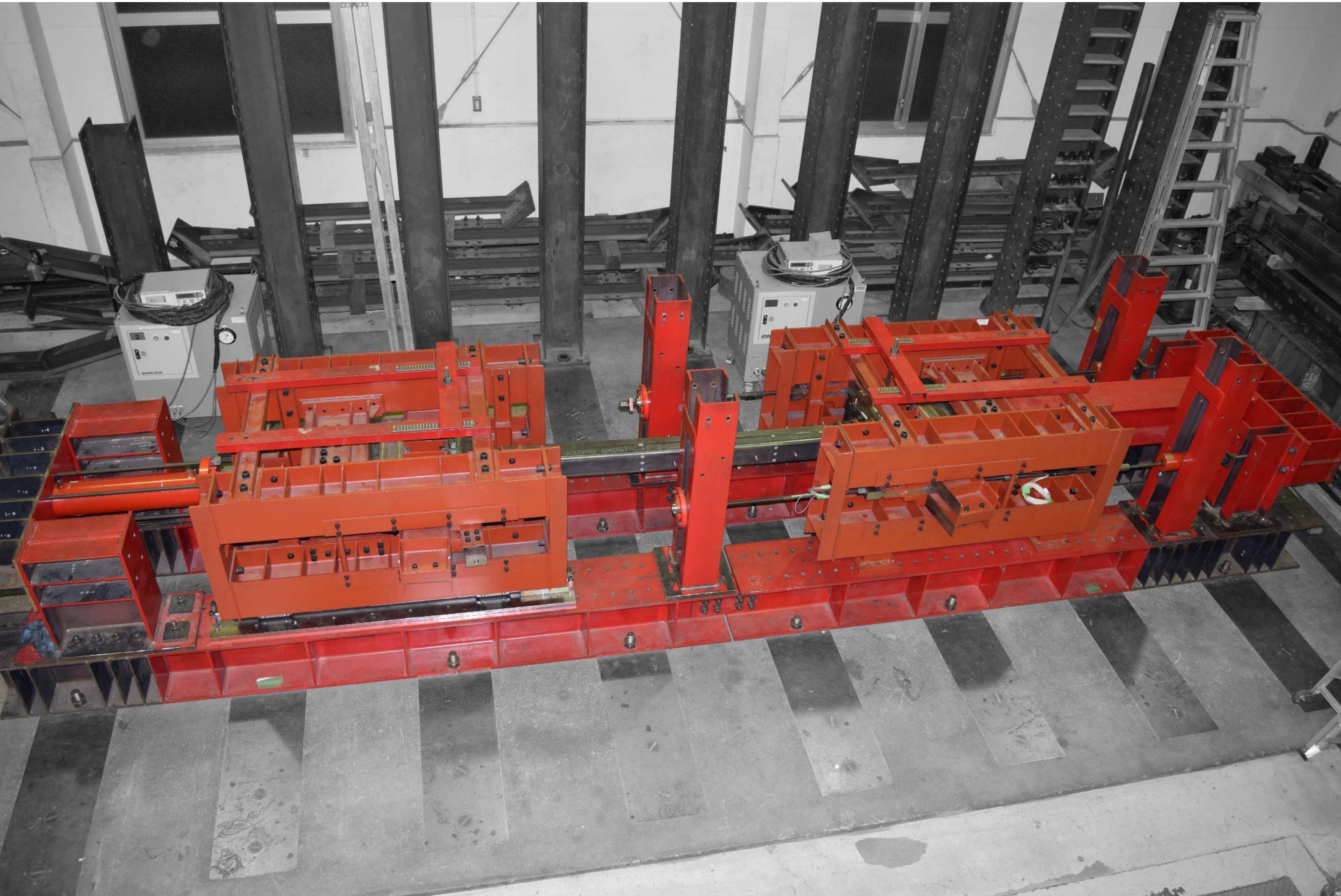




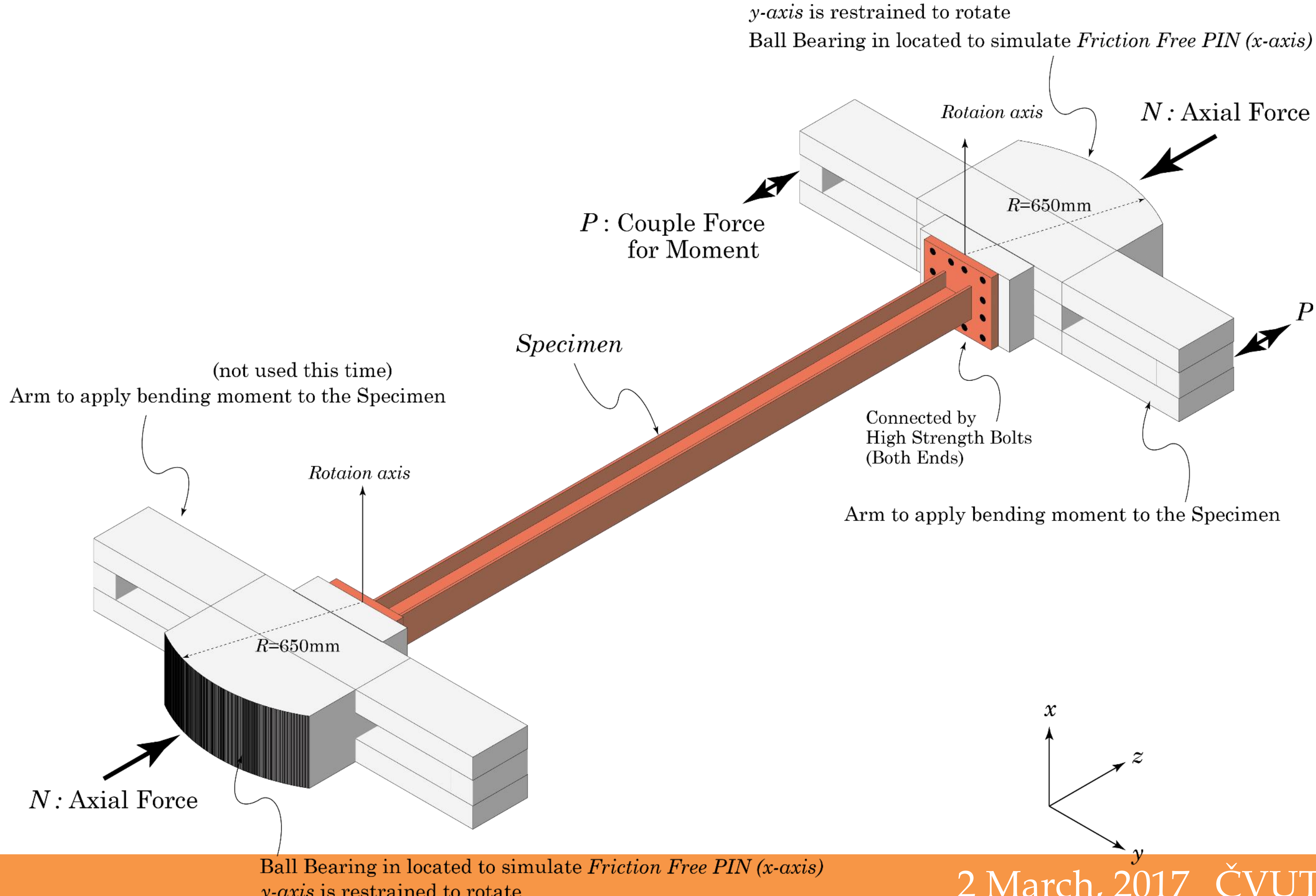






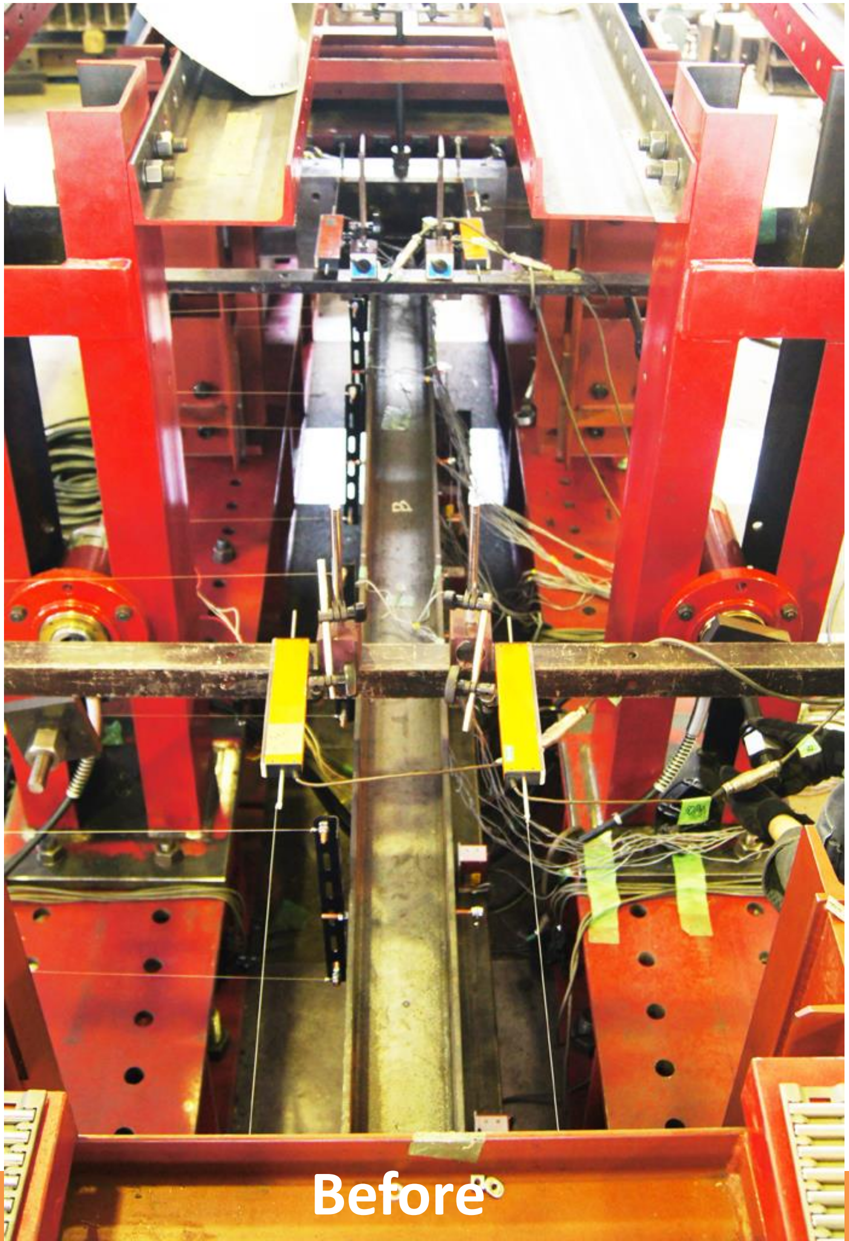


Test Setup

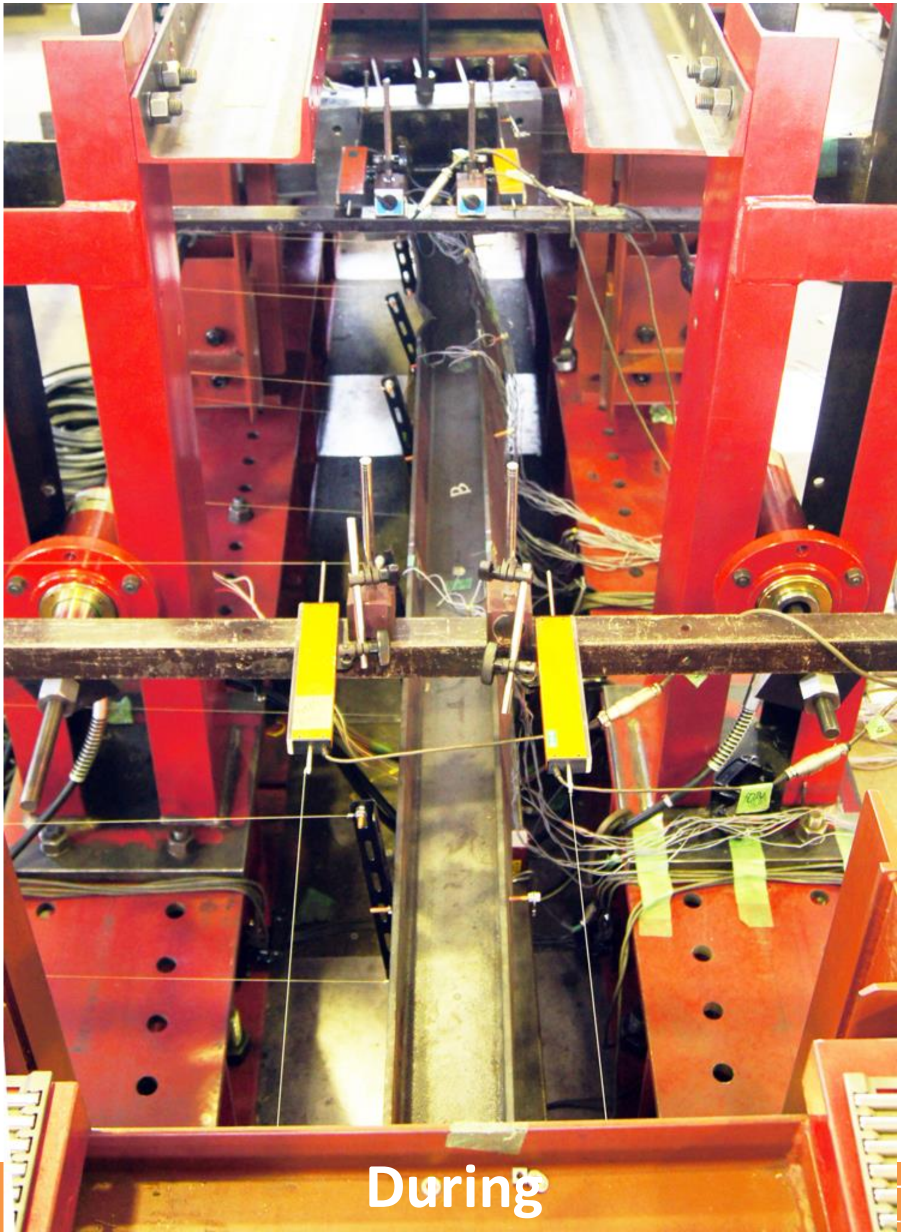


y-axis is restrained to rotate
Ball Bearing in located to simulate *Friction Free PIN (x-axis)*
Rotaion axis
N : Axial Force
R=650mm
P : Couple Force
for Moment
P
Specimen
(not used this time)
Arm to apply bending moment to the Specimen
Connected by
High Strength Bolts
(Both Ends)
Arm to apply bending moment to the Specimen
Rotaion axis
R=650mm
N : Axial Force
x
z
y
Ball Bearing in located to simulate *Friction Free PIN (x-axis)*
y-axis is restrained to rotate

Test Results (H-125x125x6.5x9)

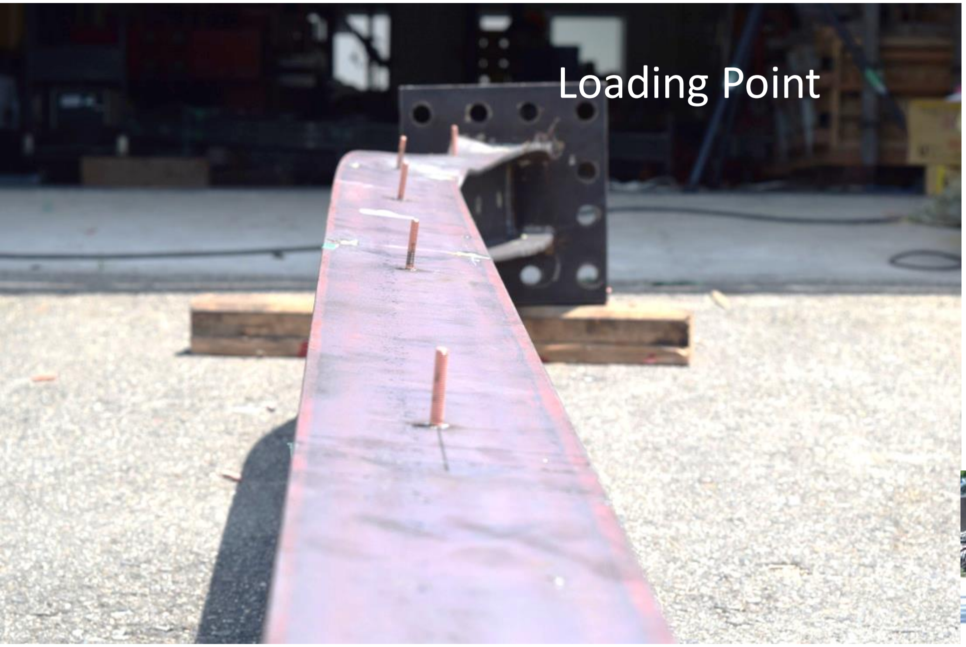


Before



During

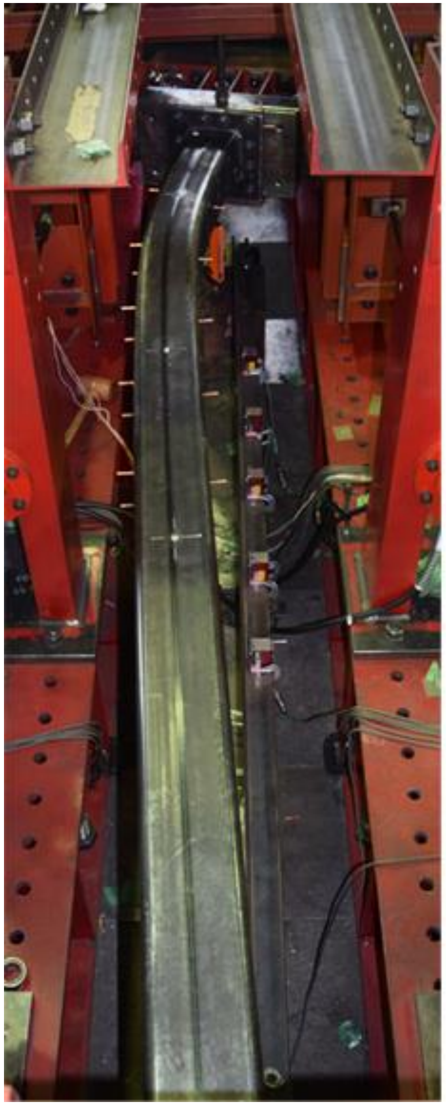
After Testing（H-125x125x6.5x9）



After Testing

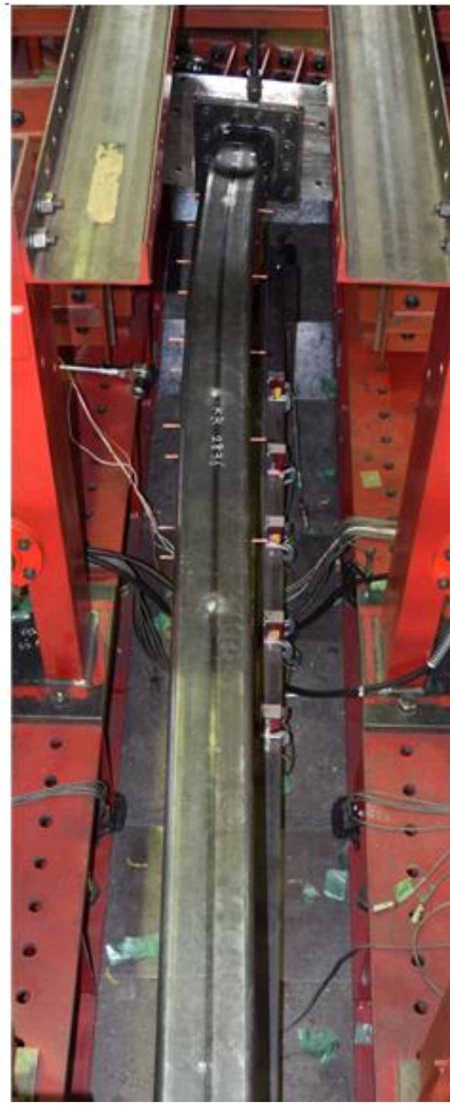
Whole View

Deformed Shape (SHS Column)



n_y=0.2, Mono
STKR400

n_y=0.3, Mono
STKR400

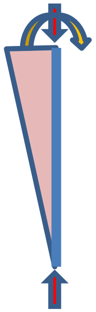


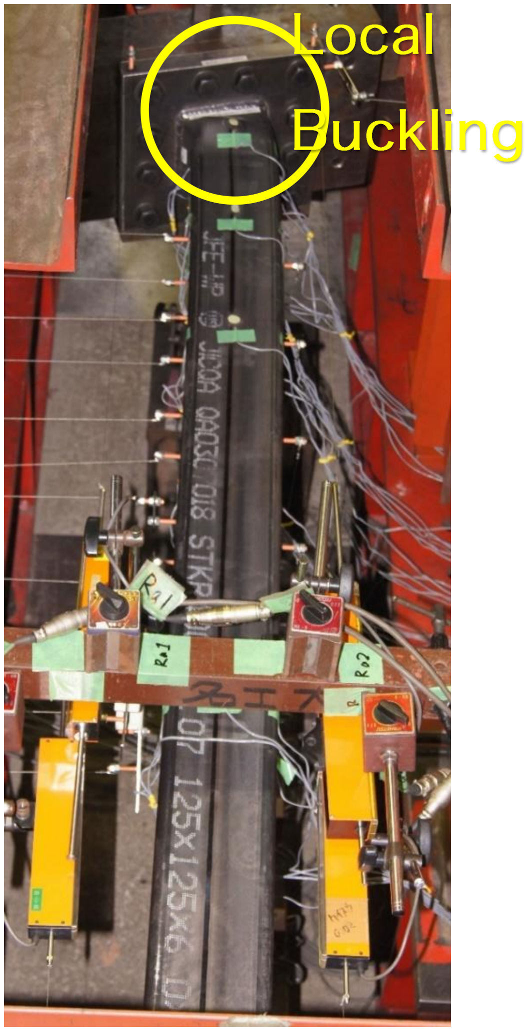
n_y=0.3, **Cyc.**
STKR400

n_y=0.3, Mono
BCR295

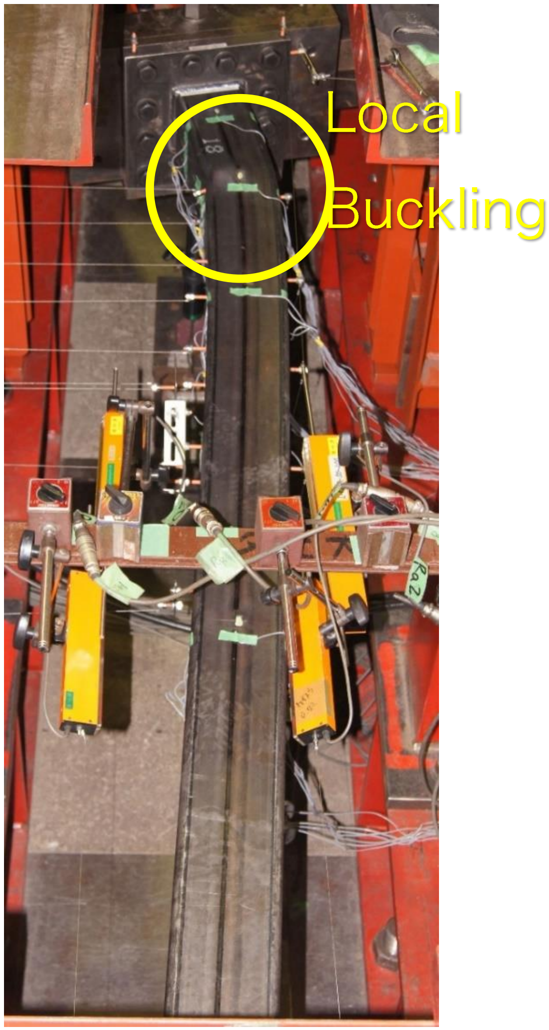
Deformed Shape (under one end moment)

Three types of failure mode were observed





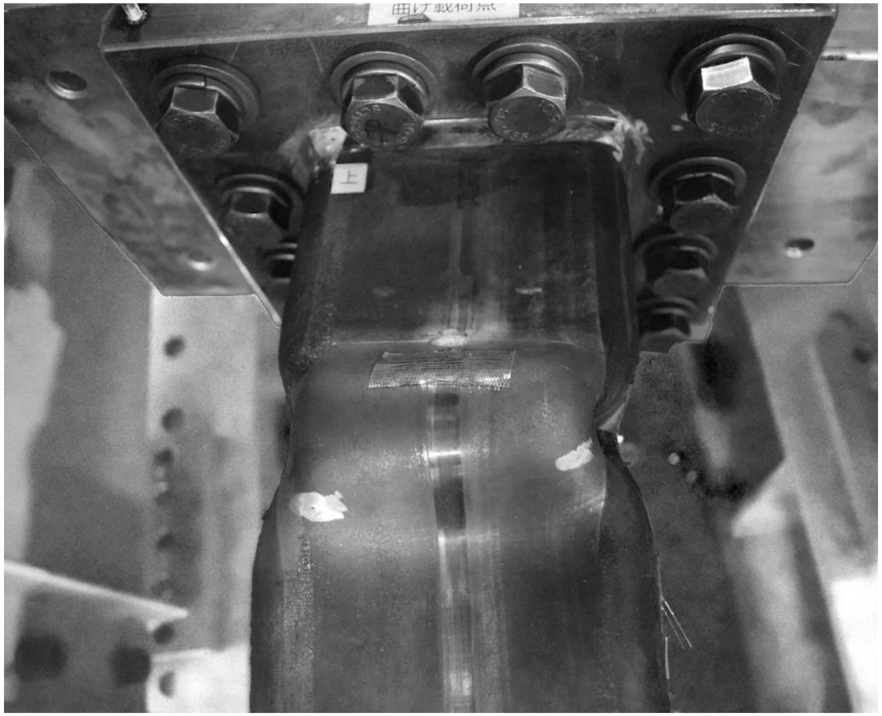
C.M.:L

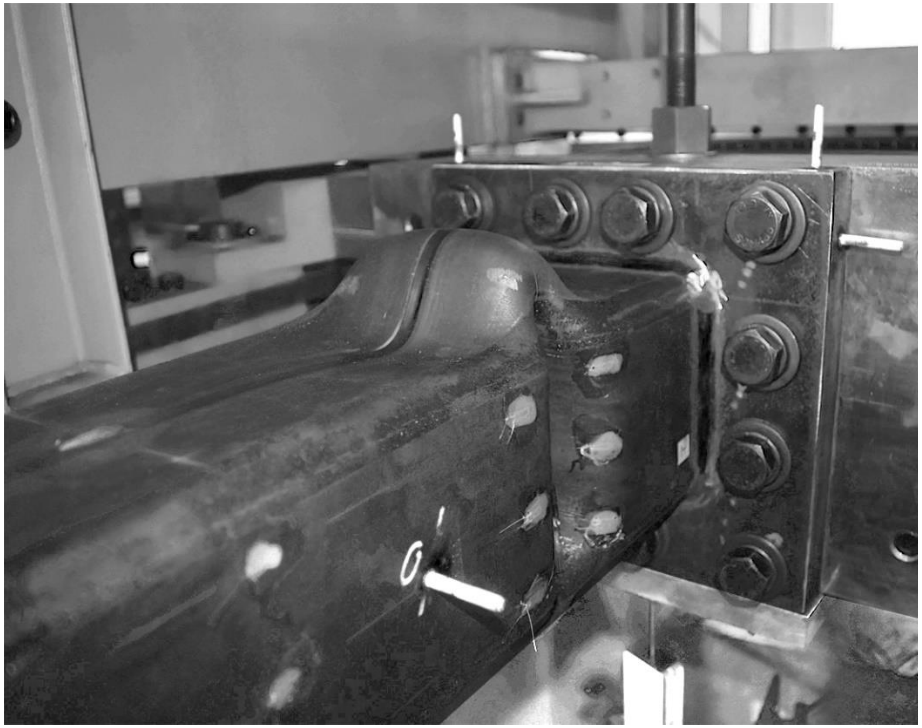


C.M.: $P\delta$ +L

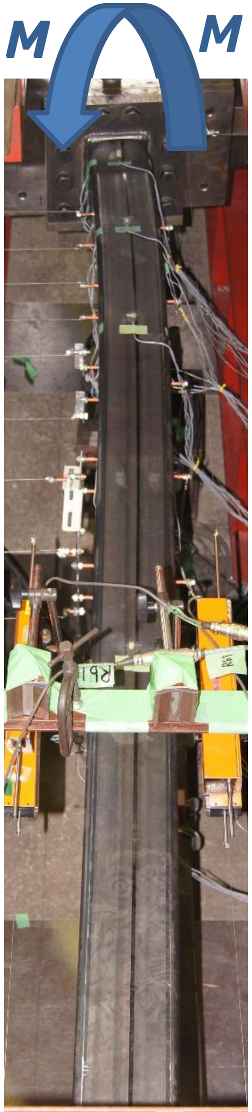
C.M.:$P\delta$

Local Buckling Deformation



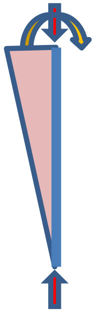


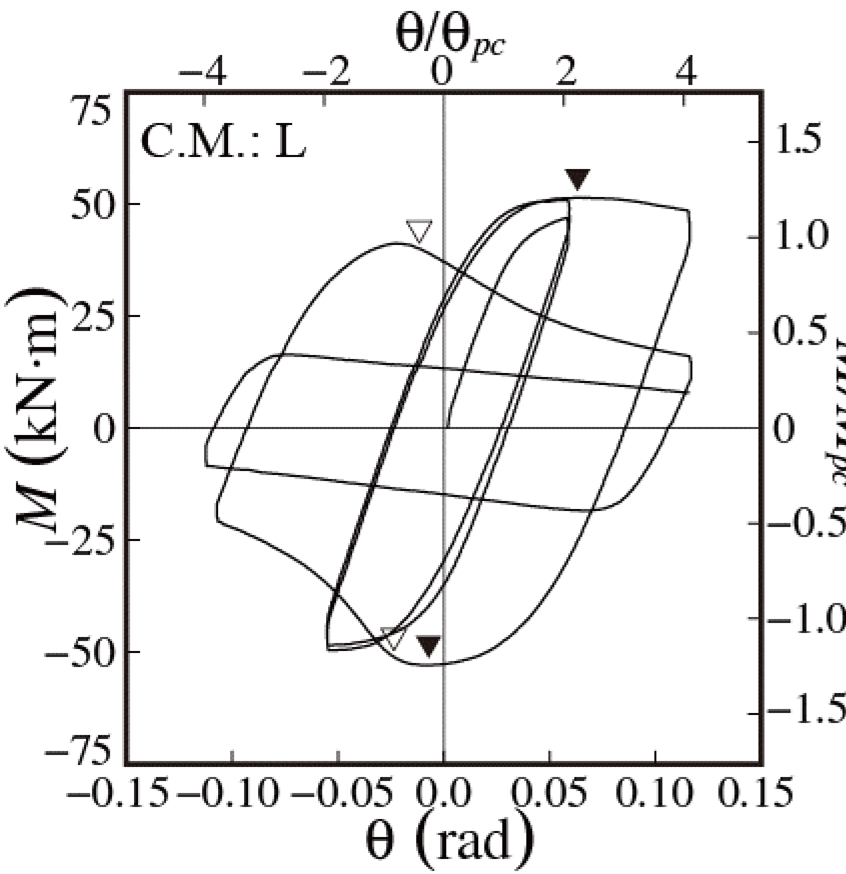
Under Cyclic Loading (one end moment)



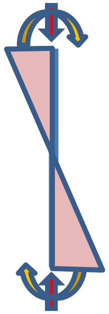
C.M.:L

C.M.: $P\delta$ +L



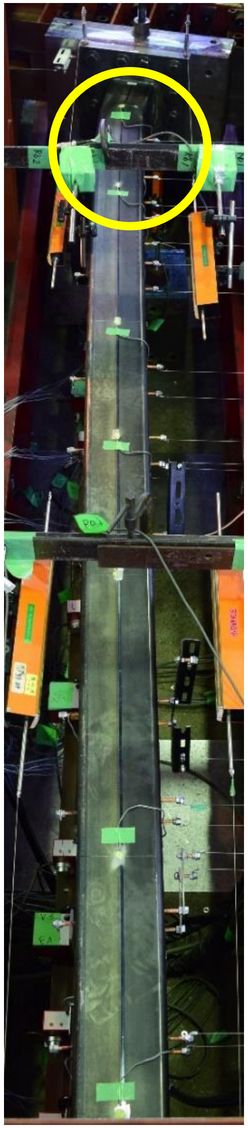


Under Antisymmetric Bending Moment



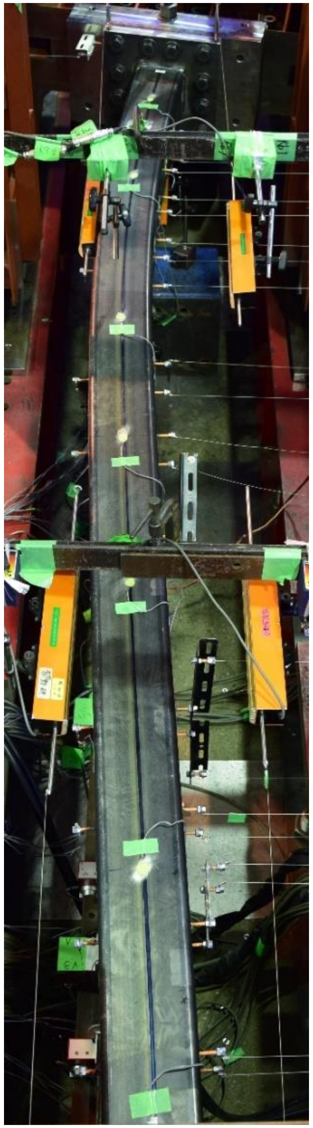
Local
Buckling

C.M.:L



Local
Buckling

C.M.: $P\delta$ +L



C.M.:$P\delta$

Test Results (One End Moment)

Strength

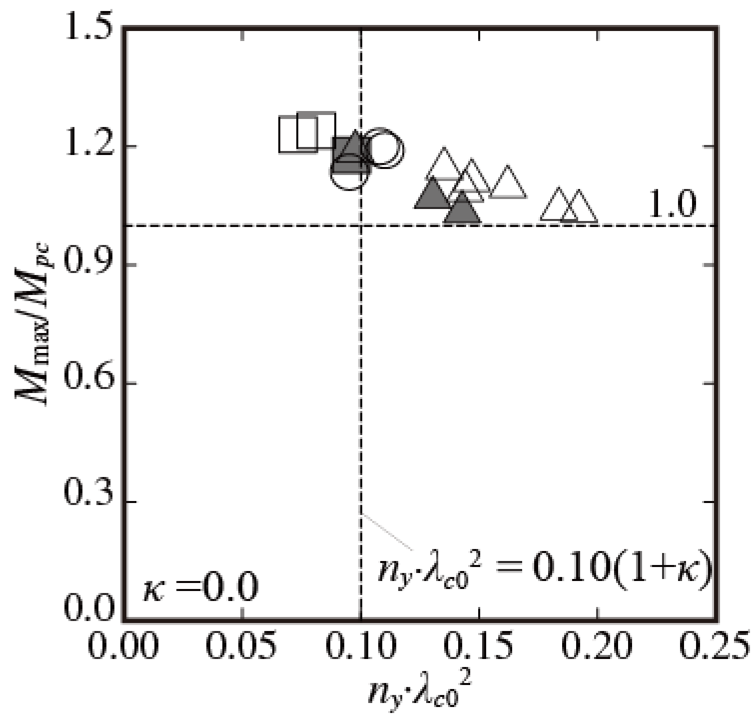


Fig. 2.15 Relationship between M_{max}/M_{pc} and $n_y \cdot \lambda_{c0}^2$

Deformation Capacity

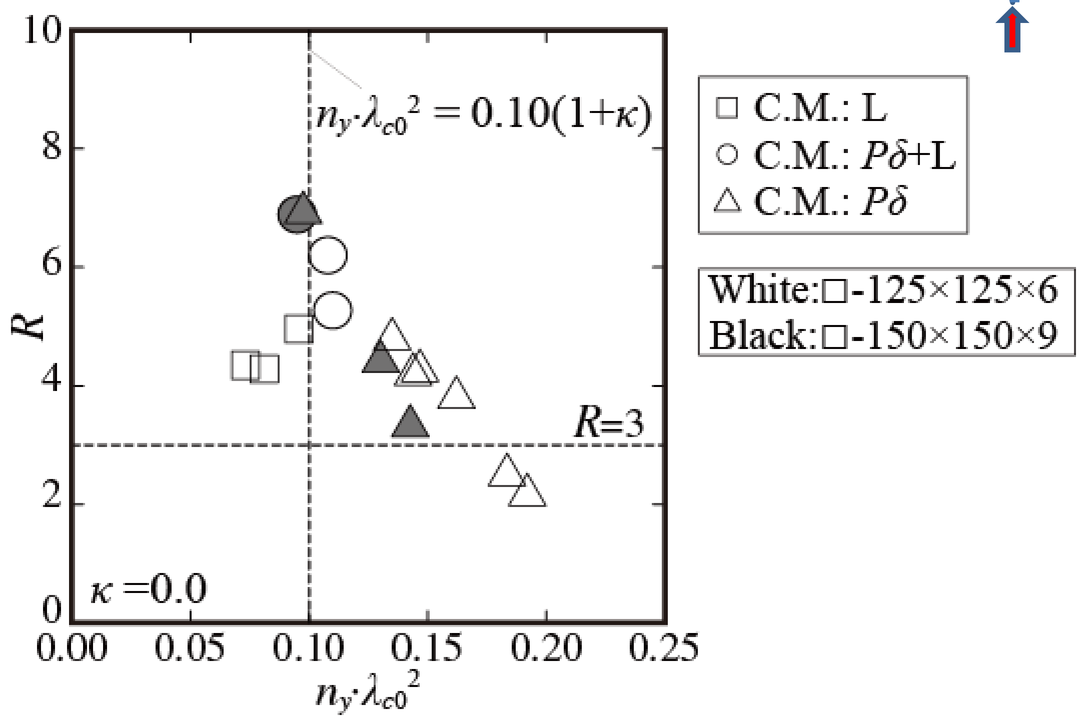
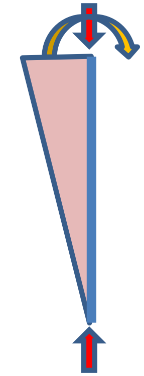
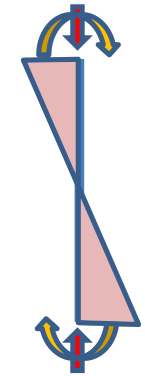


Fig. 2.16 Relationship between R and $n_y \cdot \lambda_{c0}^2$



Test Results (Antisymmetric Bending Moment)



Strength

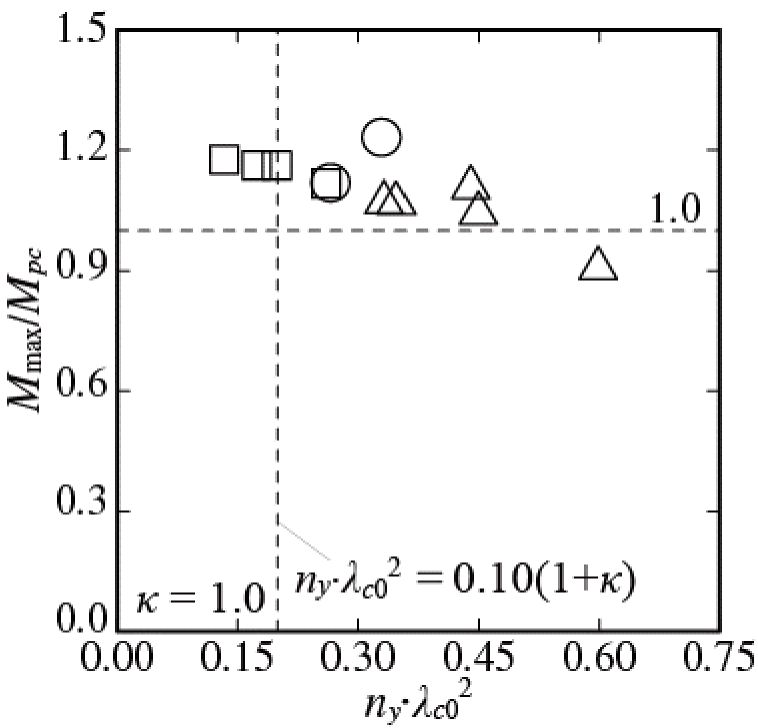


Fig. 4.15 Relationship between M_{max}/M_{pc} and $n_y \cdot \lambda_{c0}^2$

Deformation Capacity

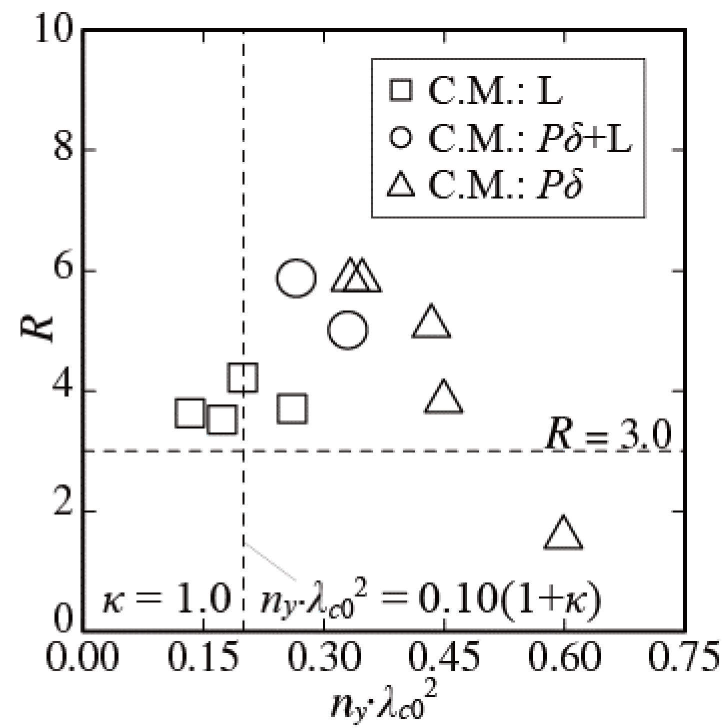
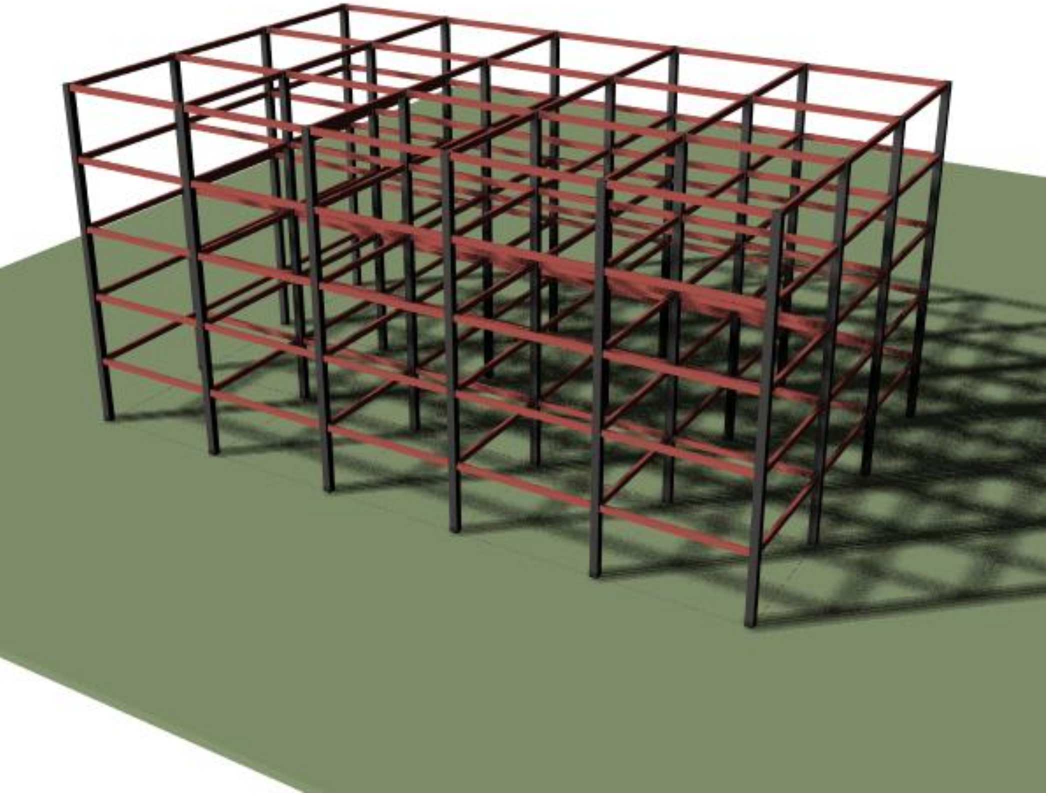


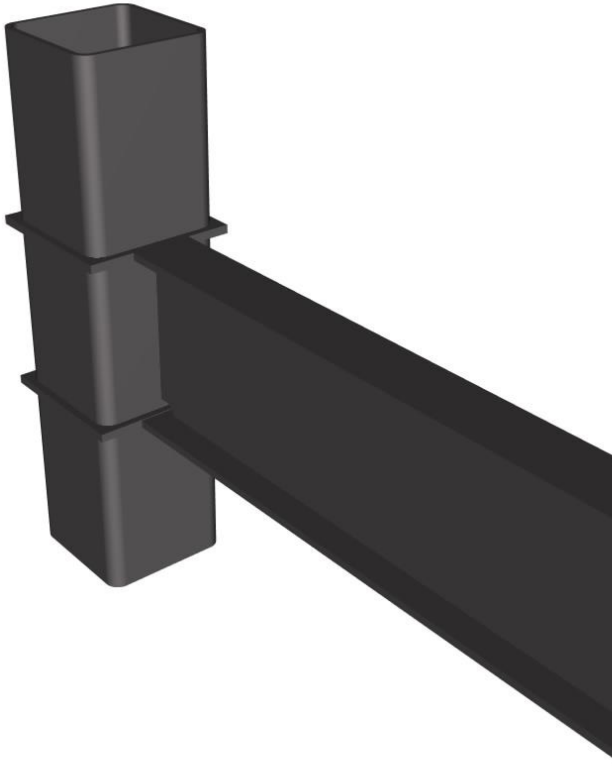
Fig. 4.16 Relationship between R and $n_y \cdot \lambda_{c0}^2$

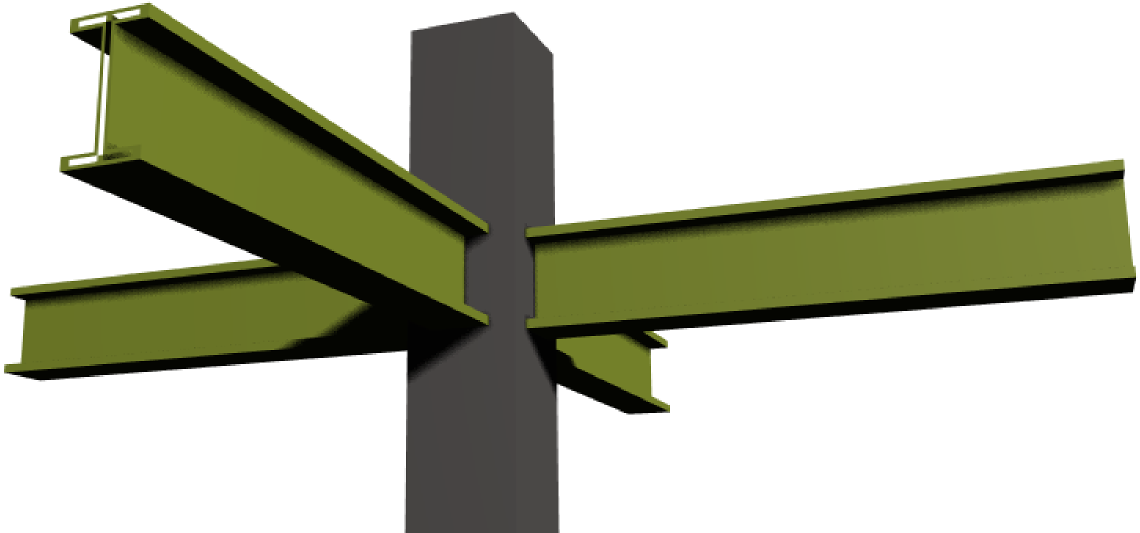
Steel Beam-to-Column Connection

Typical Steel Moment Frame

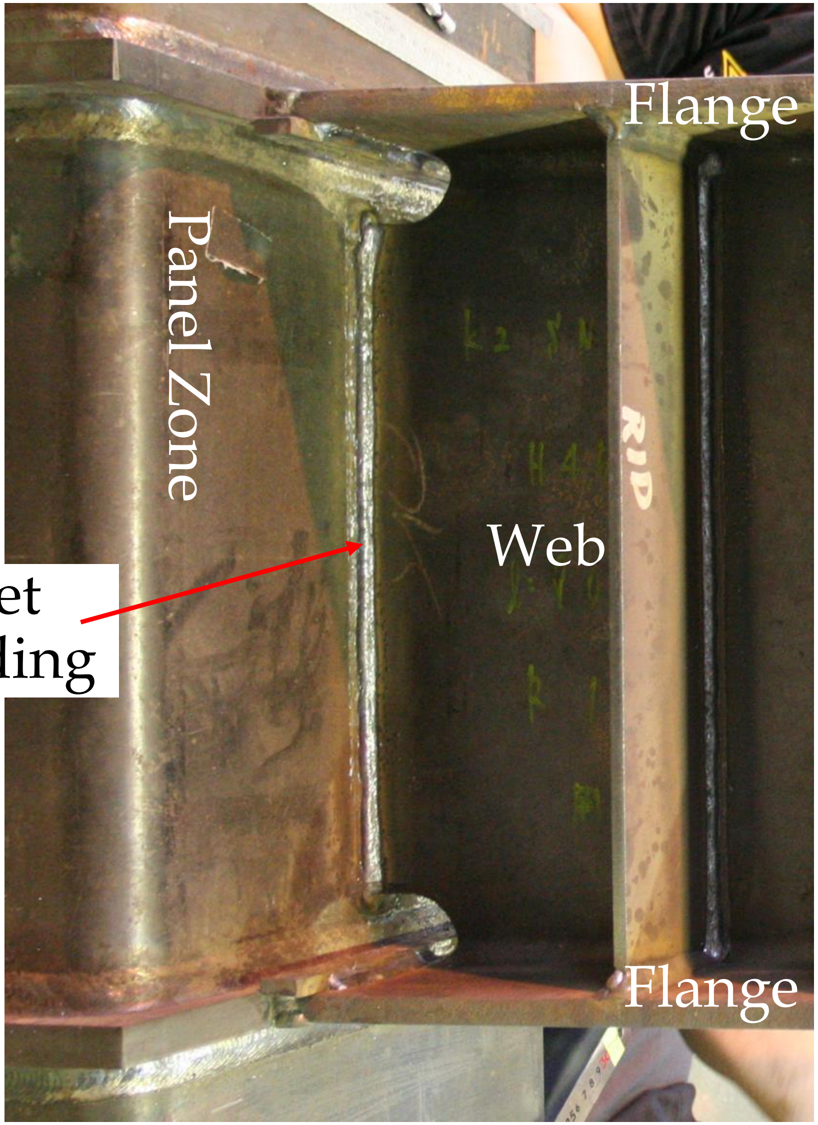
- Wide Flange Beam
- Box Section Column



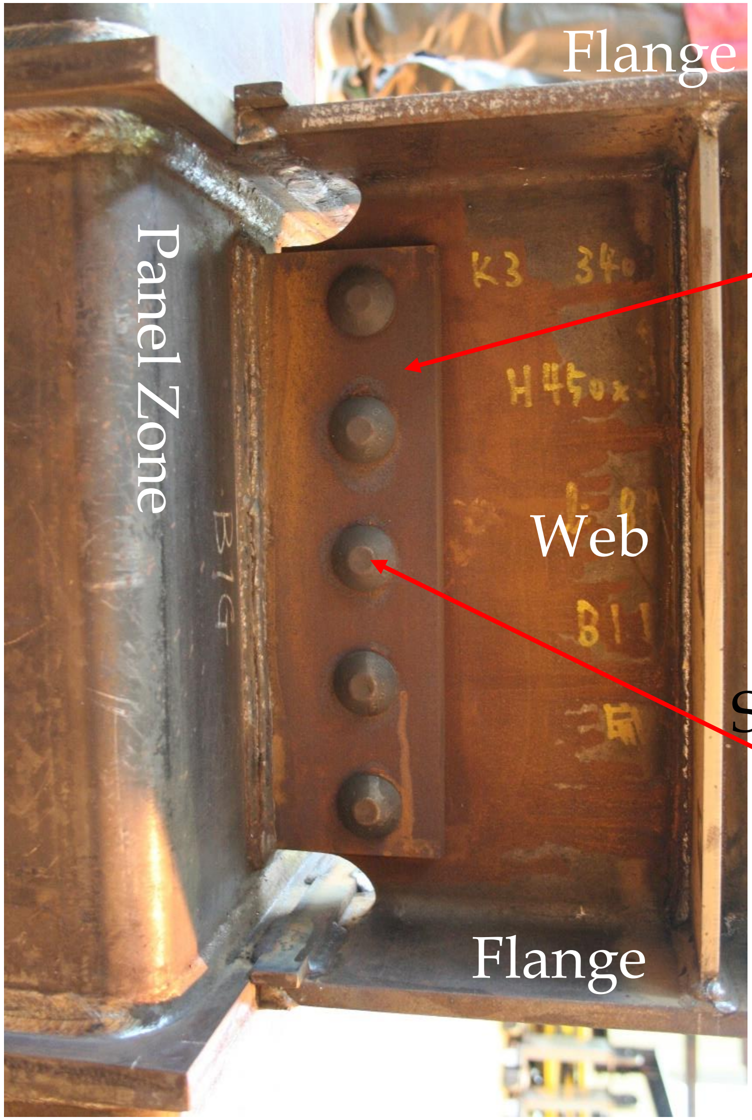




Difference Between Shop and Field



Shop Welding



Field Welding

Maximum Strength of the Beam Joint

- For Rigid Joints at ULS

$$\alpha \cdot {}_bM_p \leq {}_jM_u$$

<Beam>

$$\alpha \cdot {}_bM_p = \alpha \cdot Z_p \cdot F_{by}$$

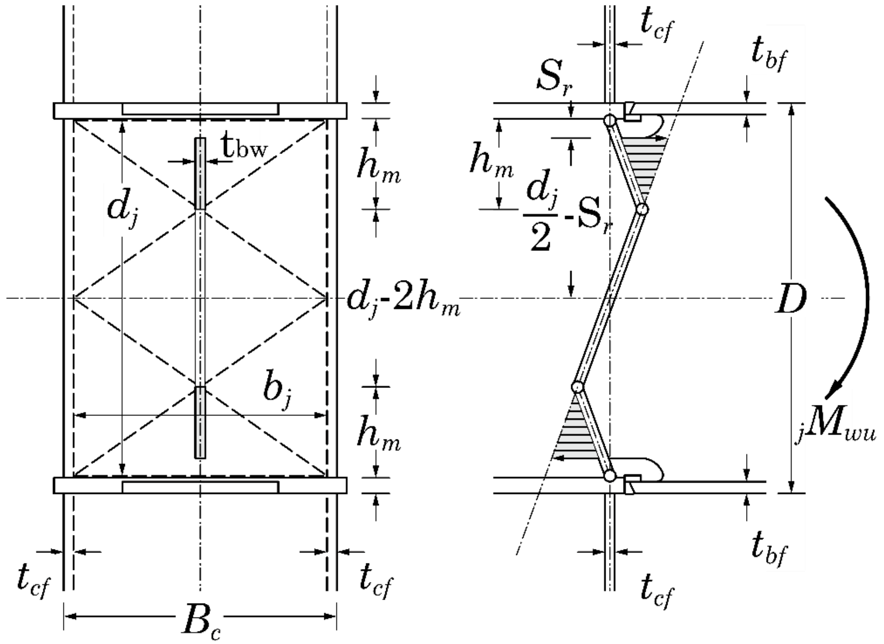
<Beam Joint Strength>

$${}_jM_u = {}_jM_{ju} + {}_jM_{wu}$$

<flange> $${}_jM_{fu} = A_f \cdot d_b \cdot F_{bu}$$

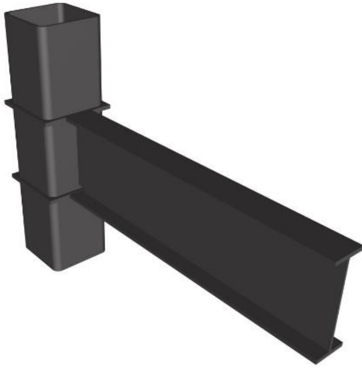
<web> $${}_jM_{wu} = m \cdot Z_{wpe} \cdot F_{by}$$

<web joint efficiency> $$m = \min\left\{1,\ 4\frac{t_{cf}}{d_j}\sqrt{\frac{b_j}{t_{bw}}\frac{F_{cy}}{F_{by}}}\right\}$$

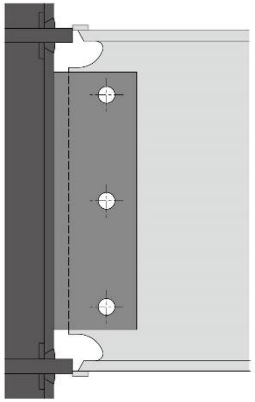
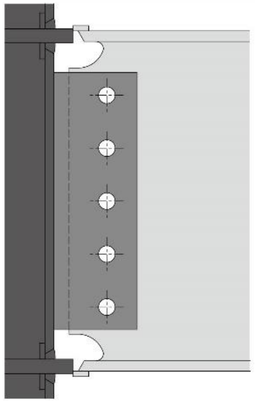
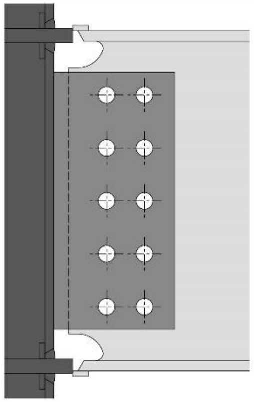
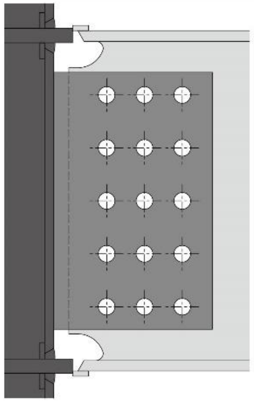
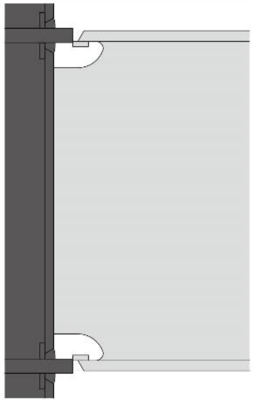


<effective area of beam web>

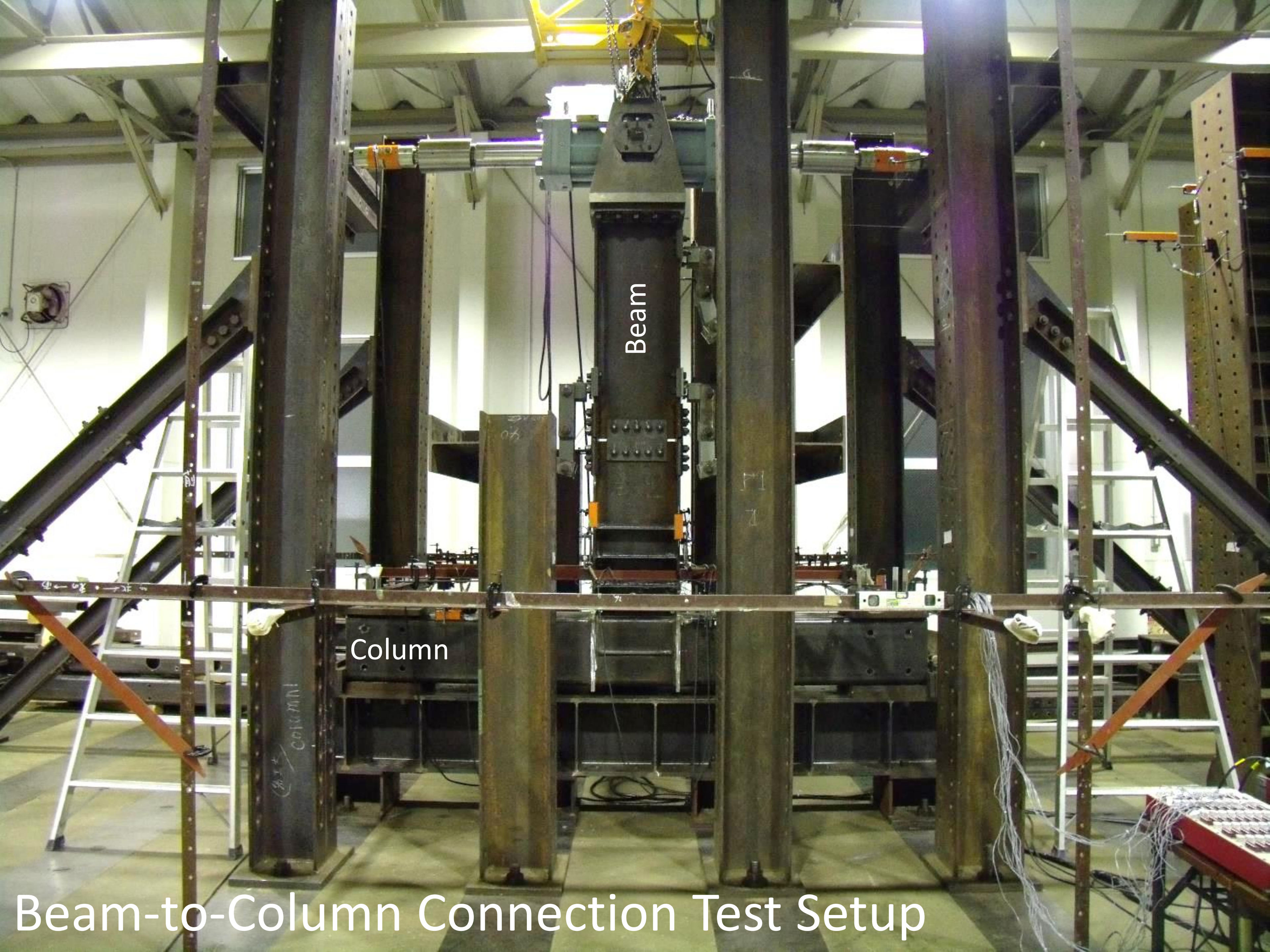
Test Specimens



Numerical model matrix-2

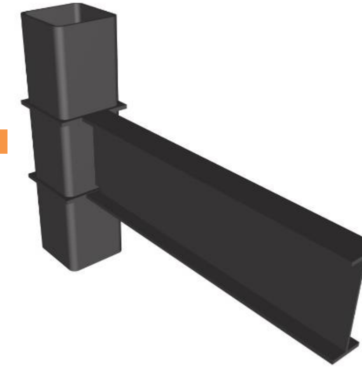
Name	B31	B51	B52	B53	W
Column	□350×350×**12** or **22** (BCR295)				
Beam	H-450×200×9×14 (SN490B)				
Joint Detail					

Note: Shear plate thickness is 16mm; steel grade is same as beam.

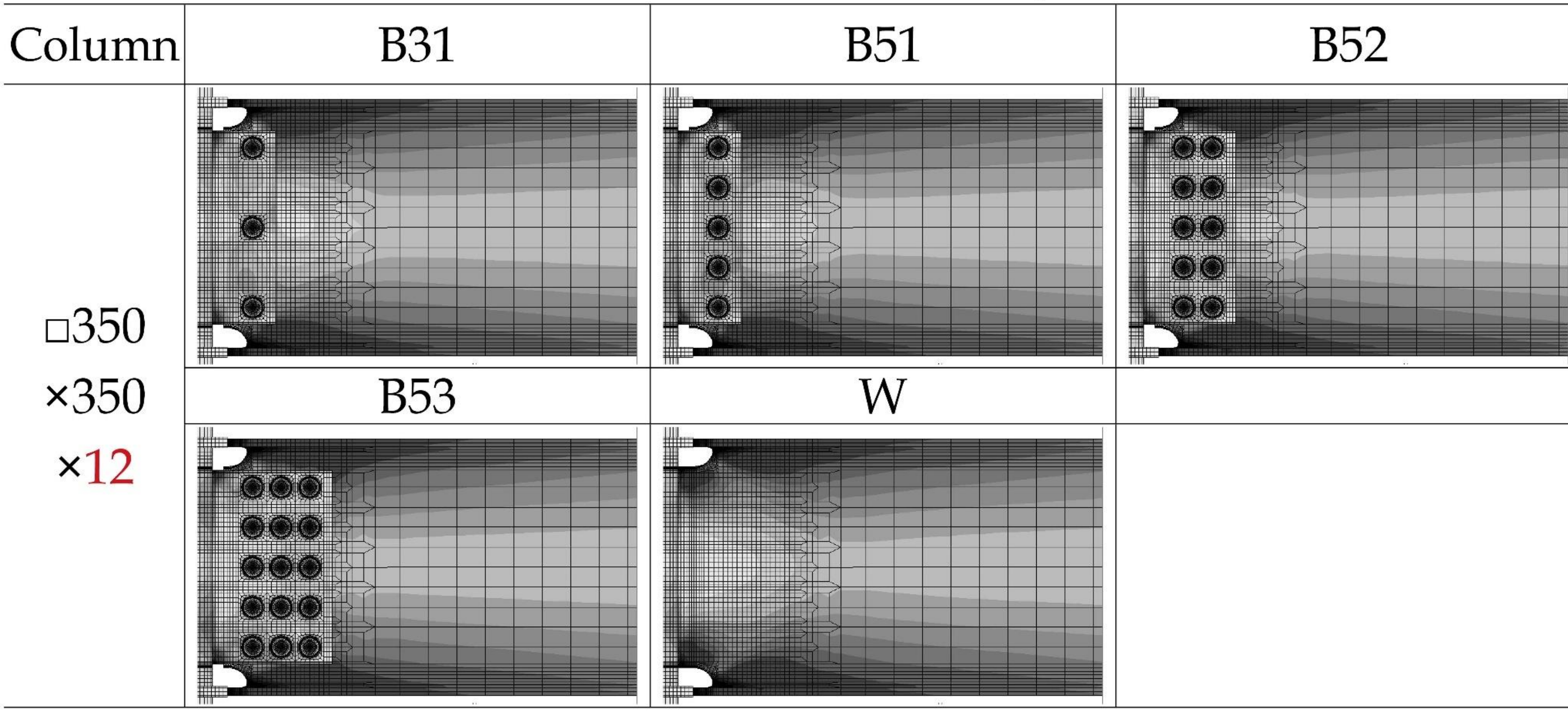

Beam
Column
Beam-to-Column Connection Test Setup

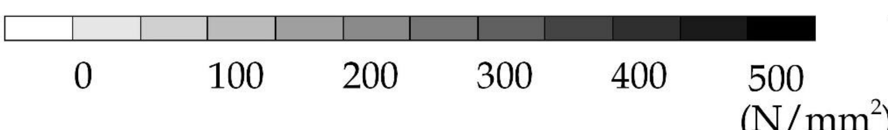
Beam-to-Column Connection

Numerical Simulation



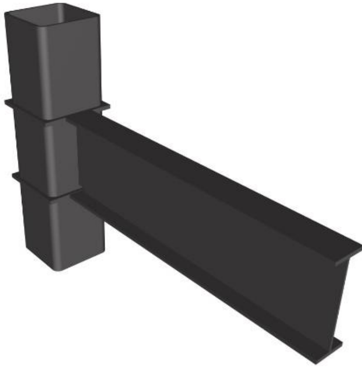
Equivalent Stress at ${}_b\theta/{}_b\theta_p = 3.0$



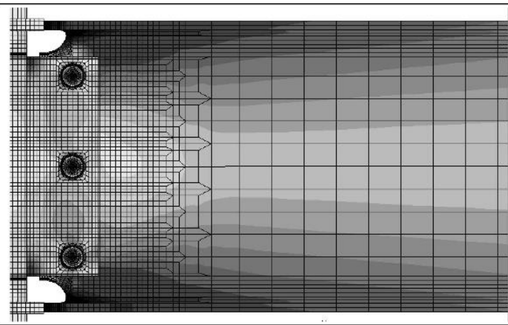
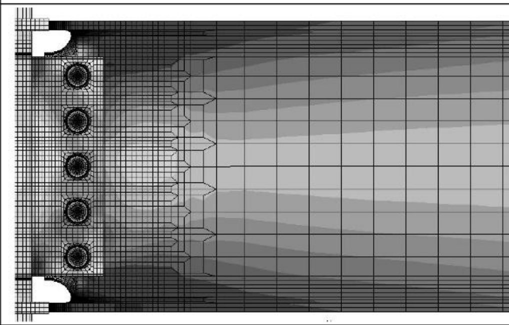
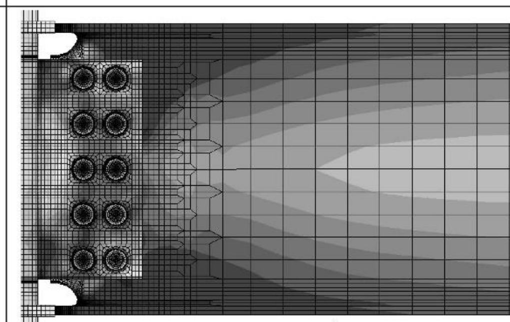
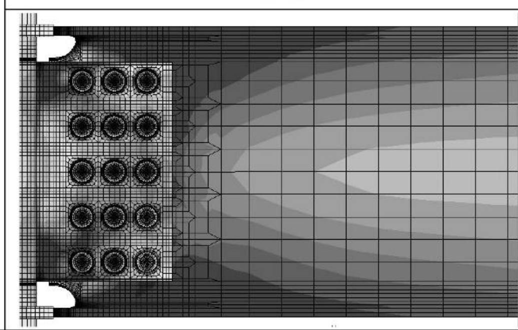
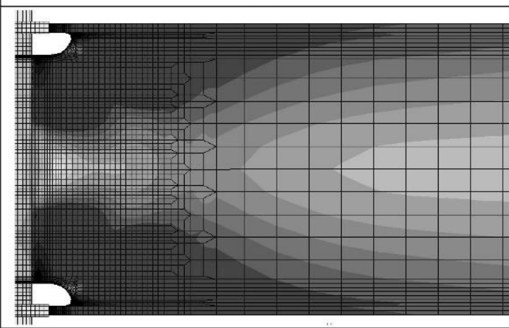


Beam: H-450×200×9×14 (SN490B)

Numerical Simulation

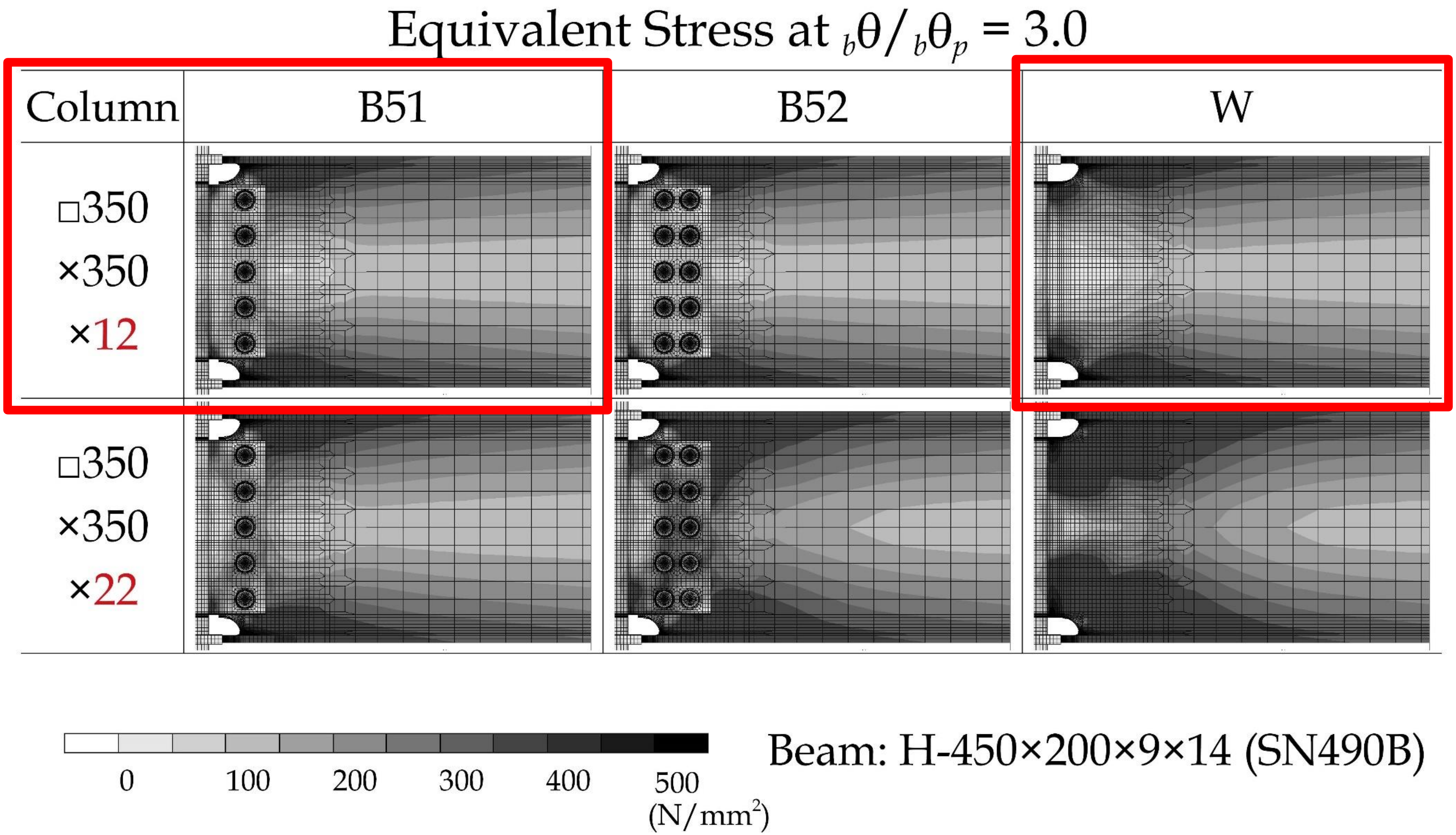


Equivalent Stress at ${}_b\theta / {}_b\theta_p = 3.0$

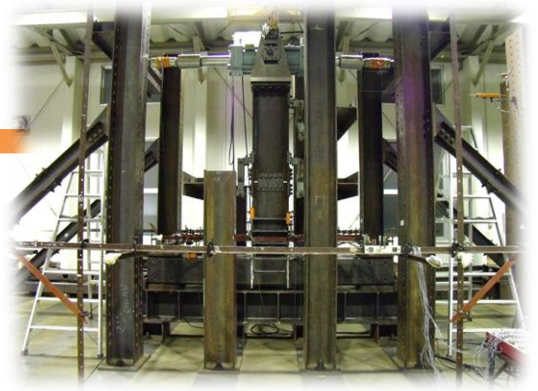
Column	B31	B51	B52
□350 ×350 ×22			
	B53	W	
			

Beam: H-450×200×9×14 (SN490B)

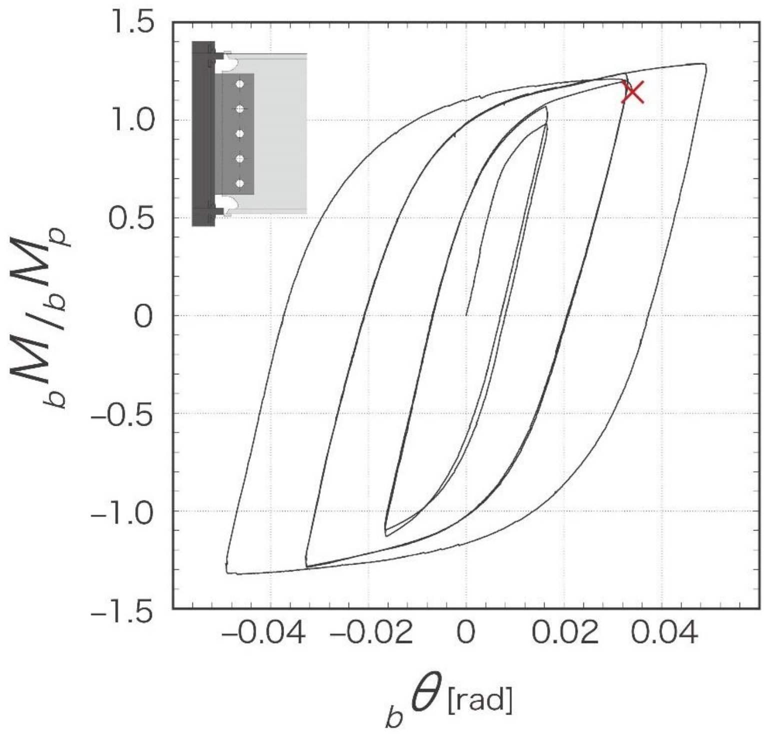
Numerical Simulation

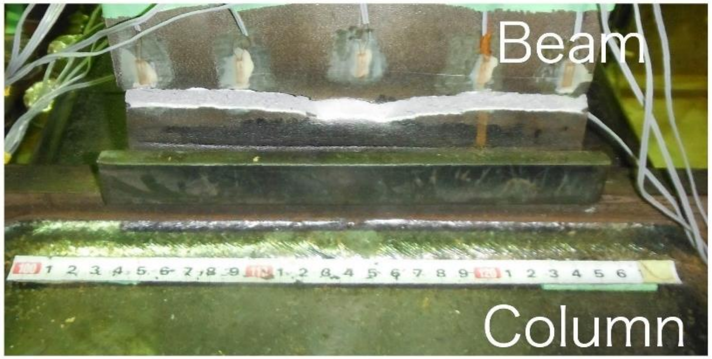


Full-Scale Testing



▶B51



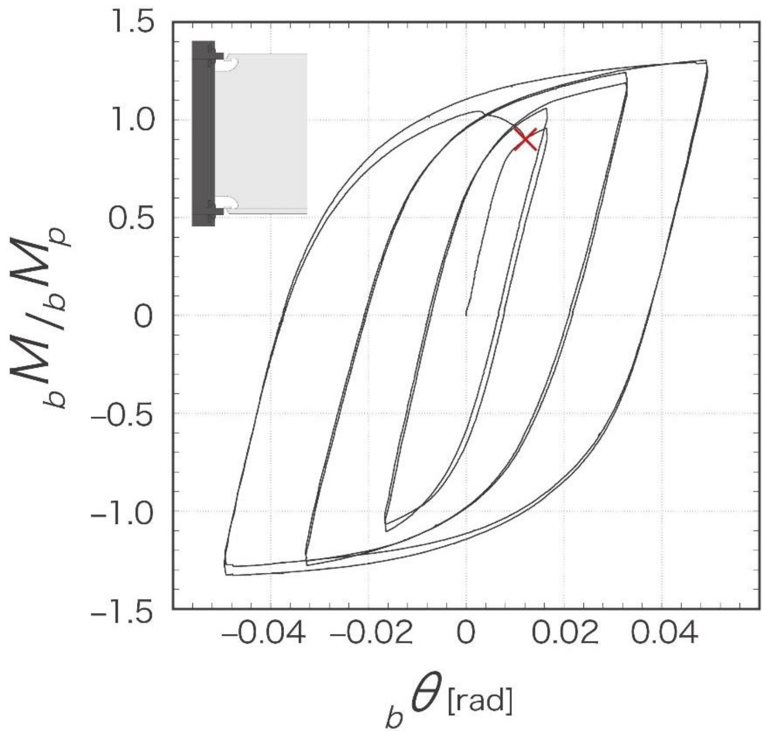


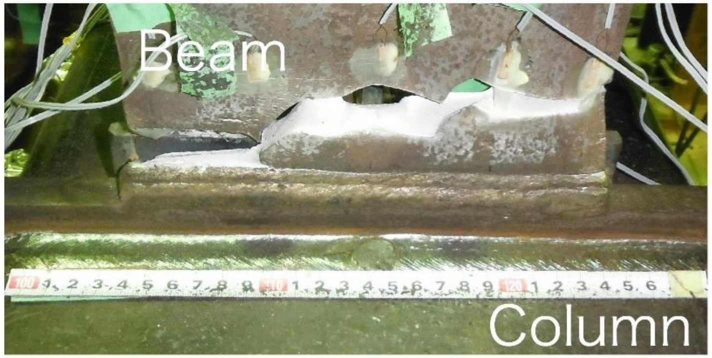
+6 $_b\theta_p$ (1) Crack Initiation at weld access hole

+6 $_b\theta_p$ (2) Fracture at Beam flange

Full-Scale Testing

▶ W





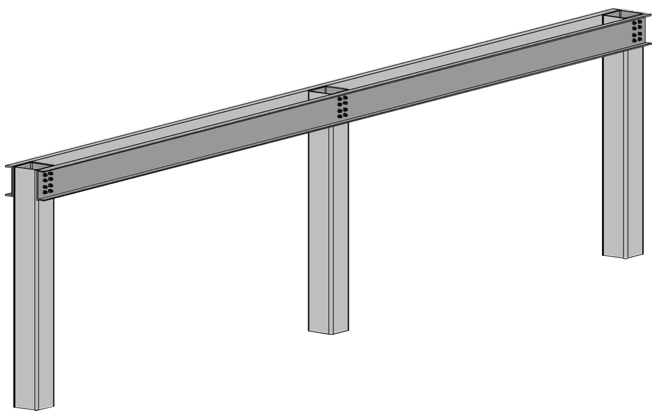
+4 $_b\theta_p$ (2) Crack Initiation at weld access hole

+8 $_b\theta_p$ (1) Fracture at Beam flange

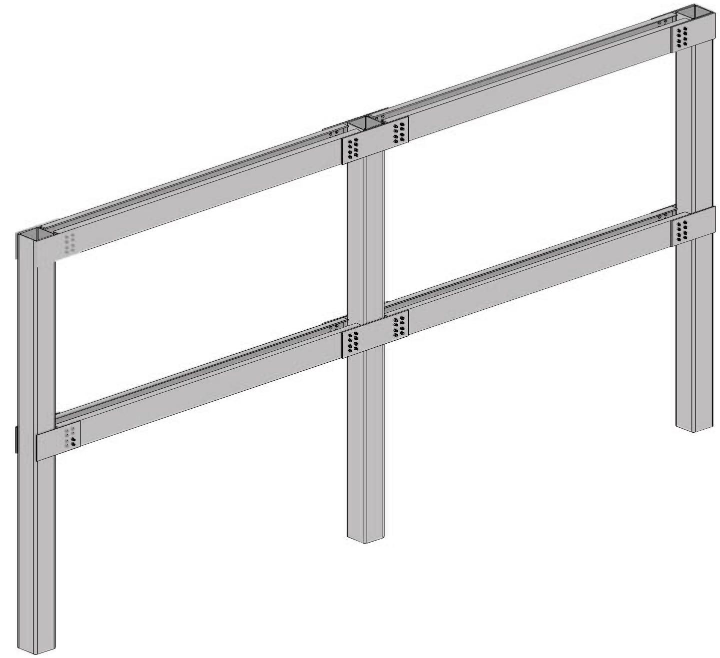

SPEEDO
TOYOTA
11
Cotton

Special Bolted Moment Frame (SBMF) System US Project

Special Bolted Moment Frame System



Ordinary Detail for One-Story Building

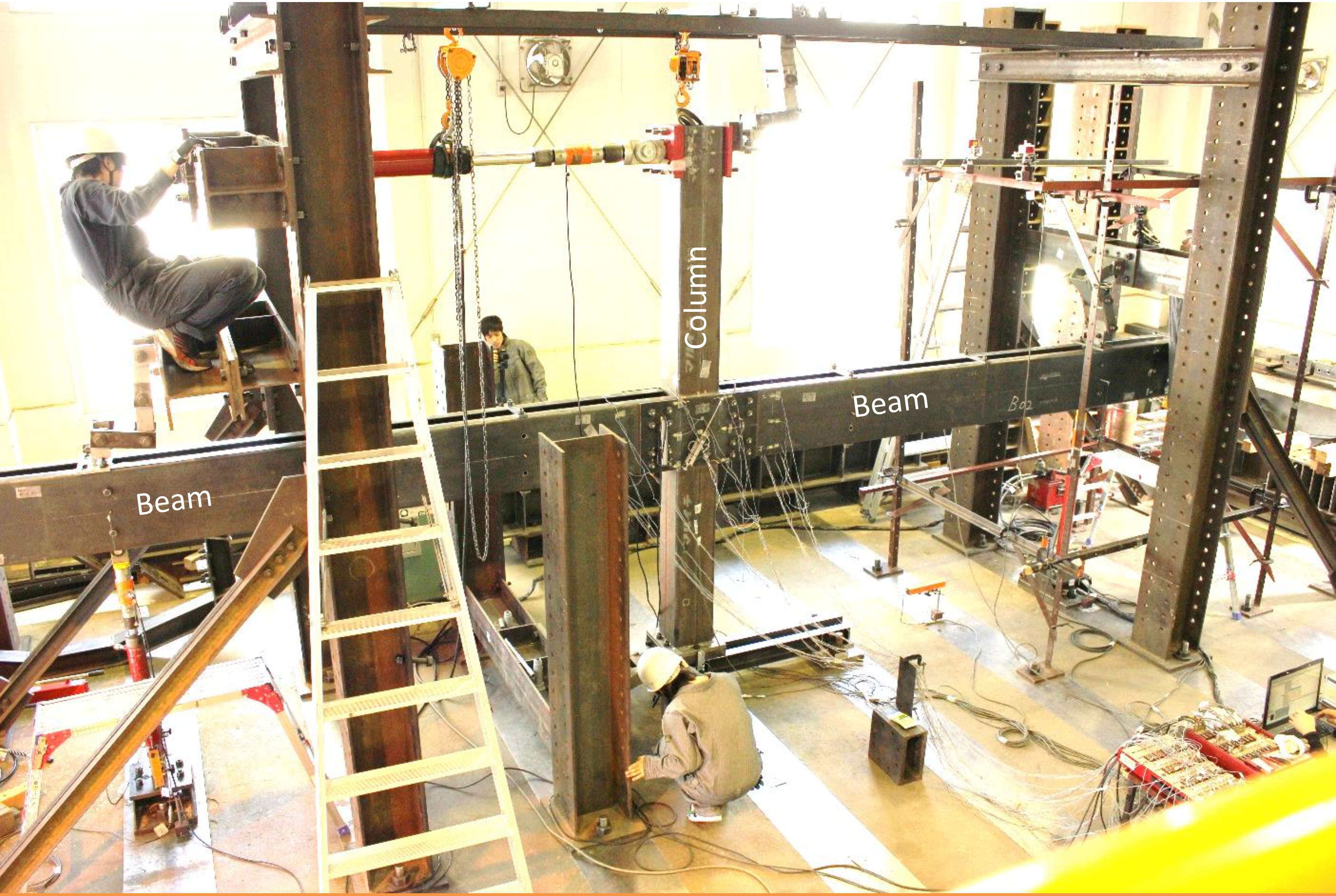


Proposed Detail for Multi-Story Building

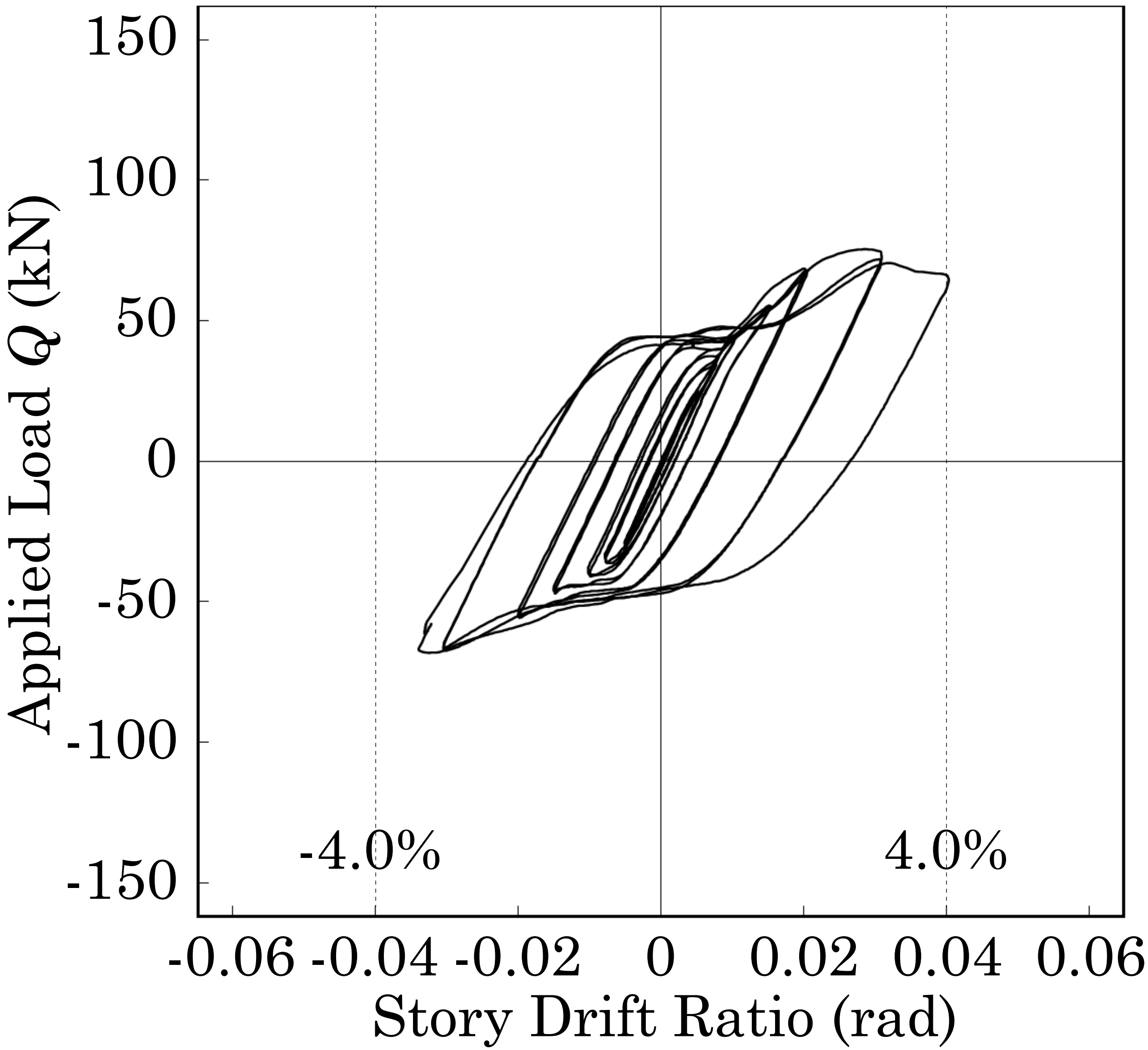
Establish Design Procedure of Multi-Story Moment-Frame using the proposed bolted connection design method


Column
Beam
Beam

Special Bolted Moment Frame (SBMF)



Sample Test Result

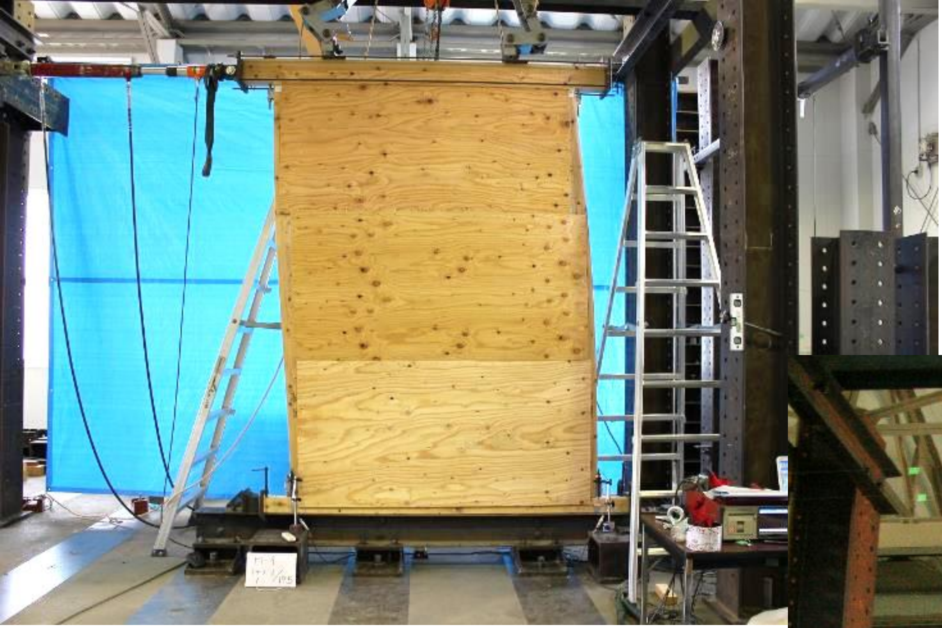


Failure mode observed in Special Bolted Moment Frame (SBMF)

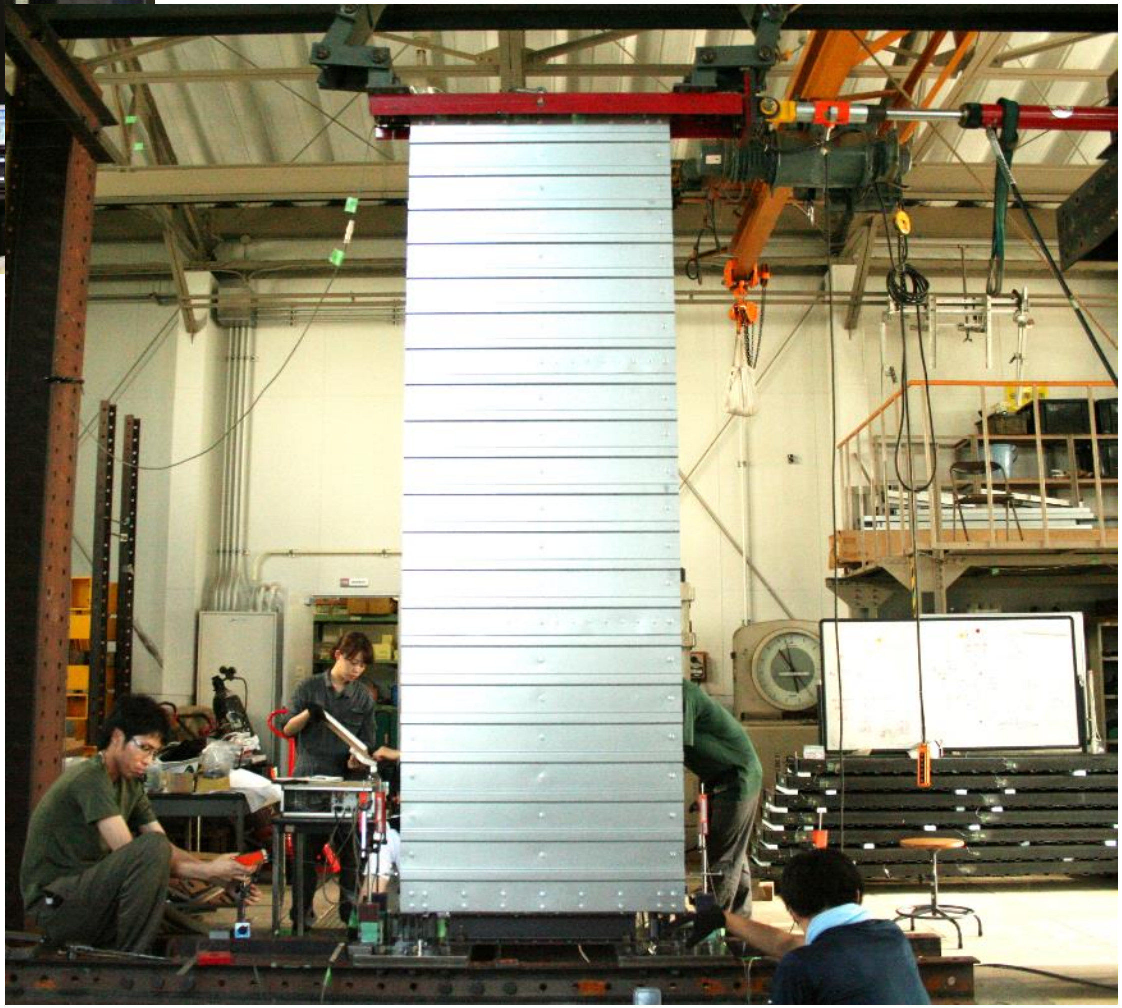
Steel Framed House (SFH)

What is Steel Framed House (SFH)

引用：米山鉄工所

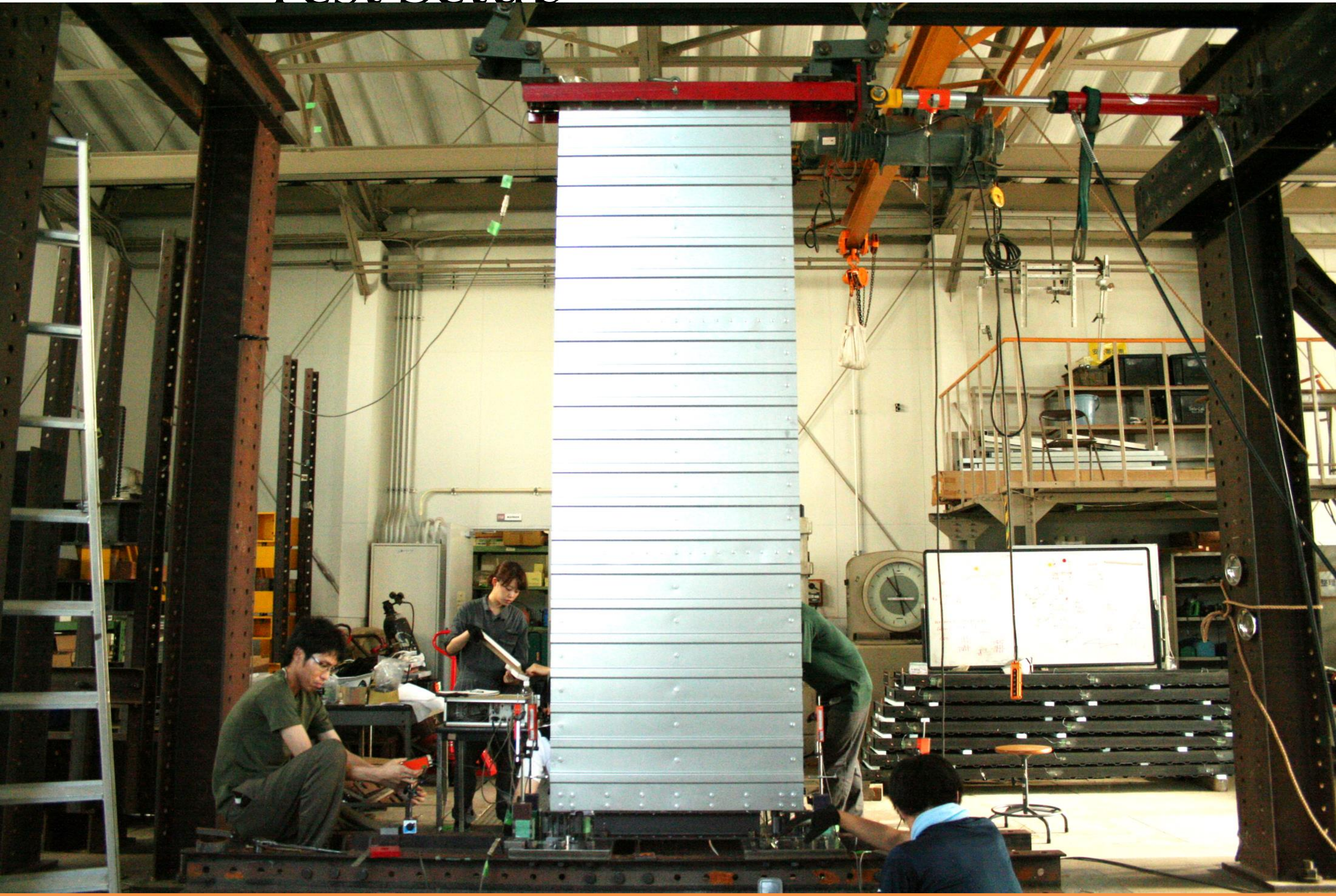


←Ordinary Shear Wall with Plywood



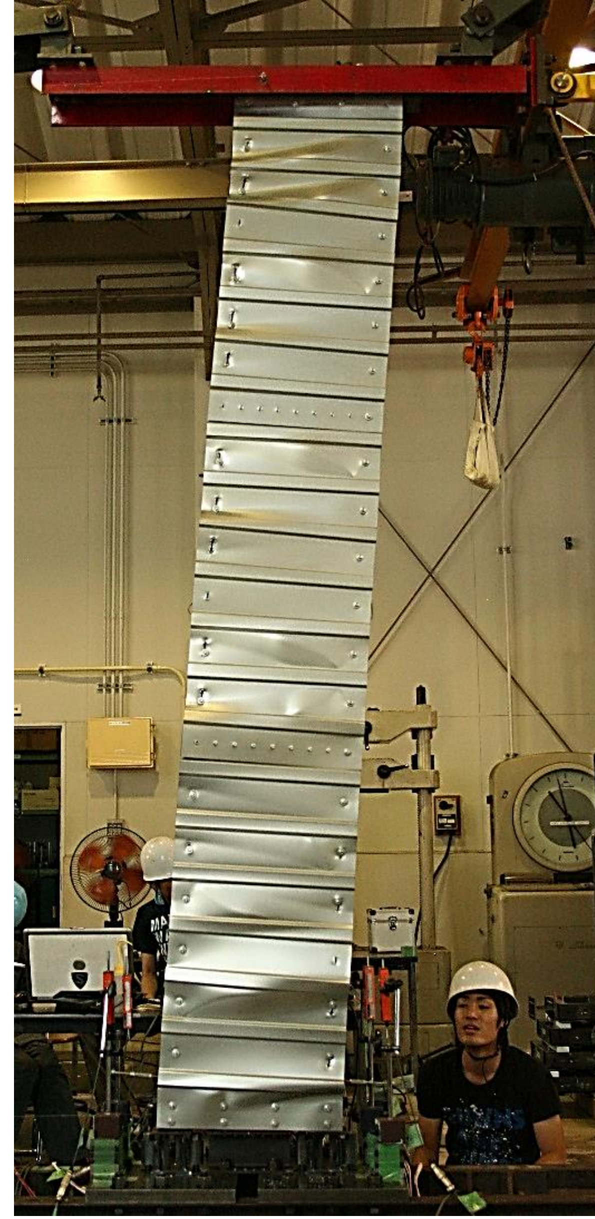
Newly Developed Shear Wall with Corrugated steel sheet

Test Setup



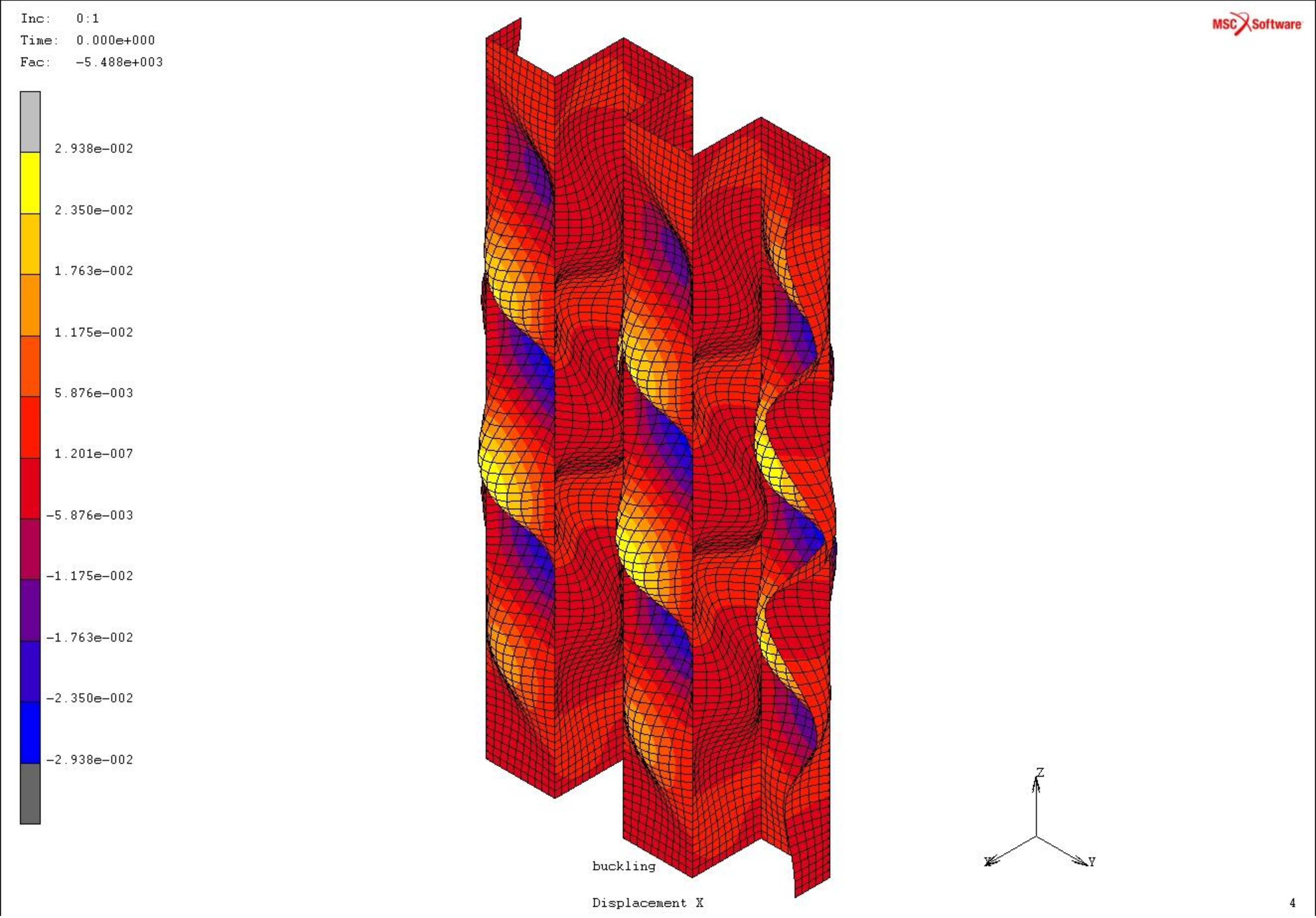
(a) 60W910

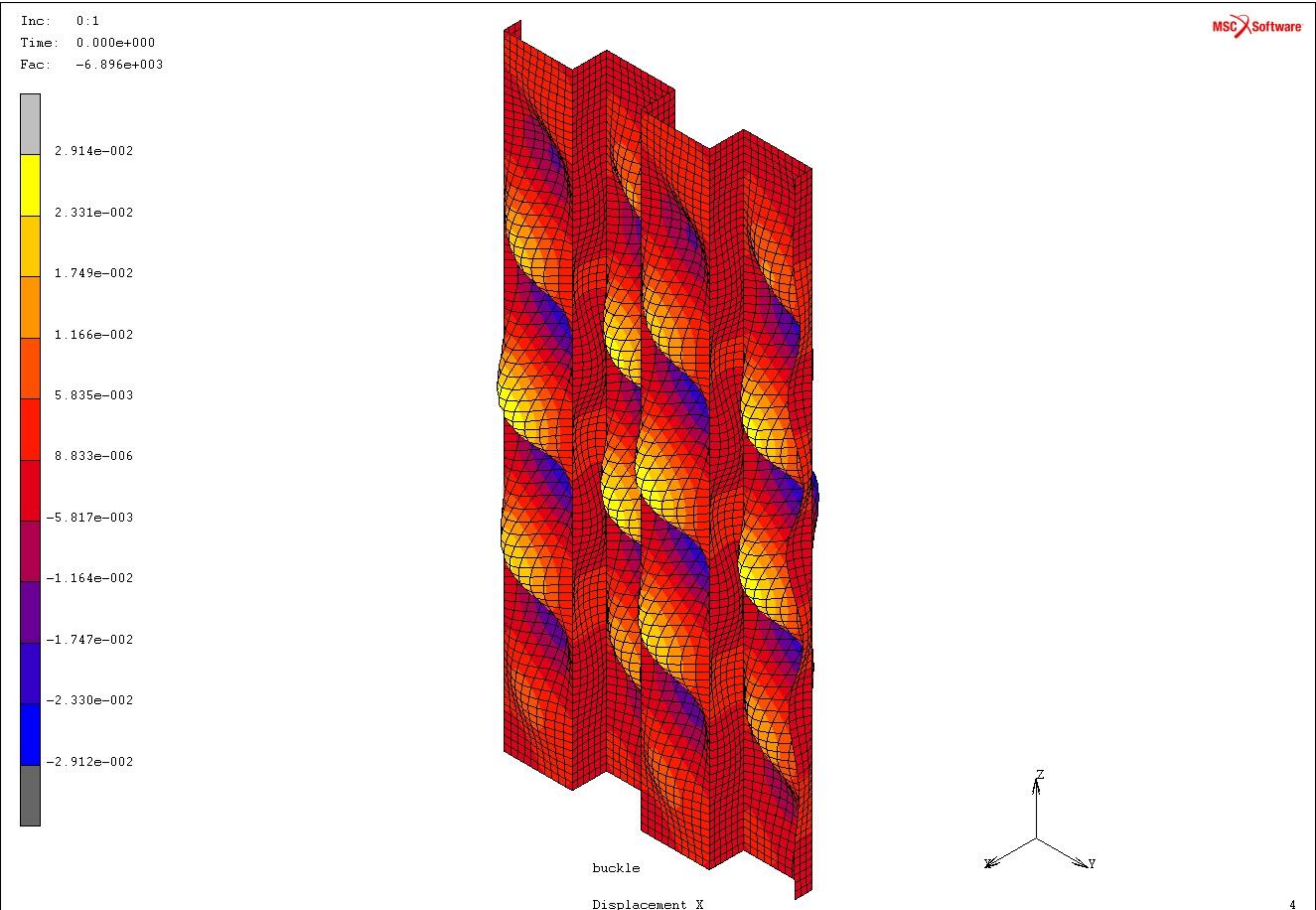
(b) 60W455

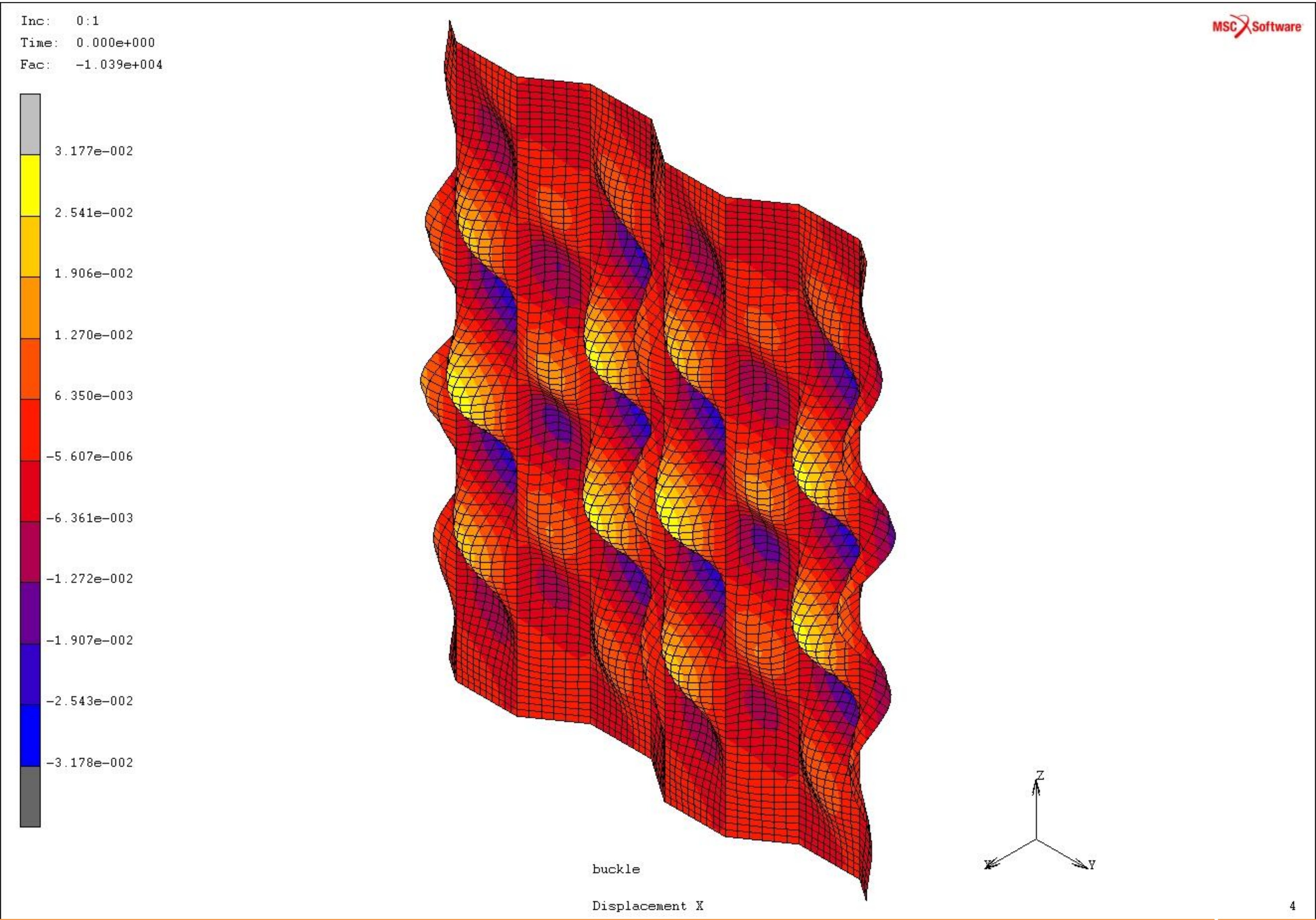


(c) 80W455

Failure Mode


Inc: 0:1
Time: 0.000e+000
Fac: -5.488e+003
2.938e-002
2.350e-002
1.763e-002
1.175e-002
5.876e-003
1.201e-007
-5.876e-003
-1.175e-002
-1.763e-002
-2.350e-002
-2.938e-002
MSC Software
buckling
Displacement X
Z
X
Y
4


Inc: 0:1
Time: 0.000e+000
Fac: -6.896e+003
2.914e-002
2.331e-002
1.749e-002
1.166e-002
5.835e-003
8.833e-006
-5.817e-003
-1.164e-002
-1.747e-002
-2.330e-002
-2.912e-002
MSC Software
buckle
Displacement X
Z
X
Y
4


Inc: 0:1
Time: 0.000e+000
Fac: -1.039e+004
MSC Software
3.177e-002
2.541e-002
1.906e-002
1.270e-002
6.350e-003
-5.607e-006
-6.361e-003
-1.272e-002
-1.907e-002
-2.543e-002
-3.178e-002
Z
X
Y
buckle
Displacement X
4

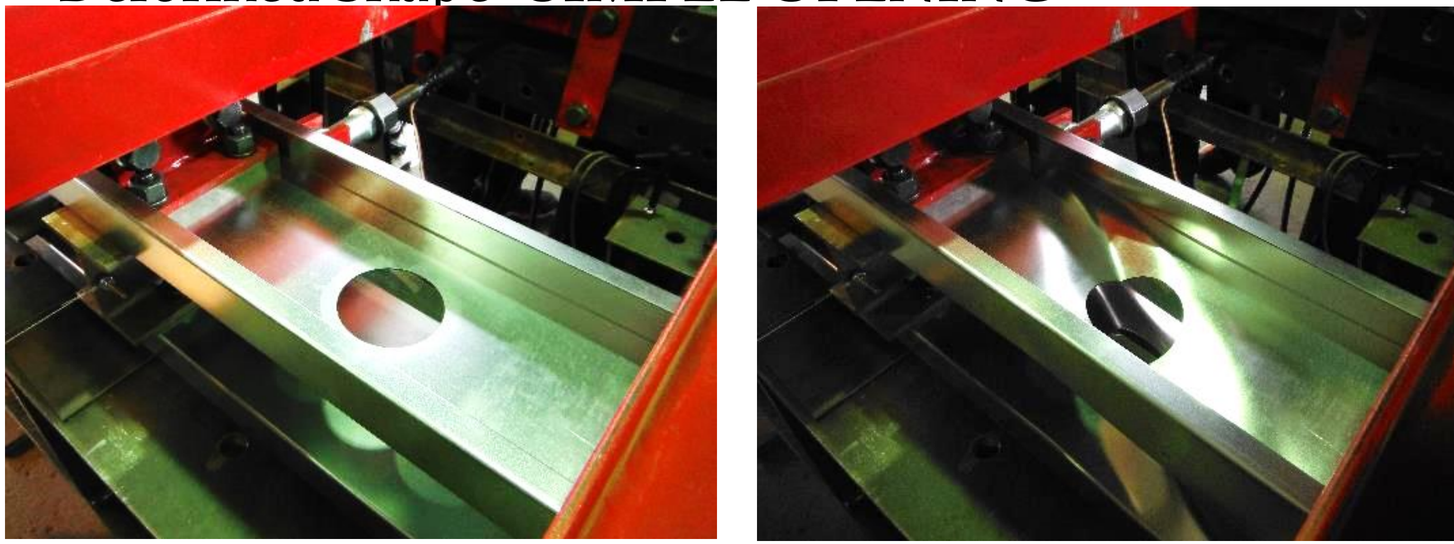
Buckling Strength of Light-Gauge Members with Large Openings

INTRODUCTION

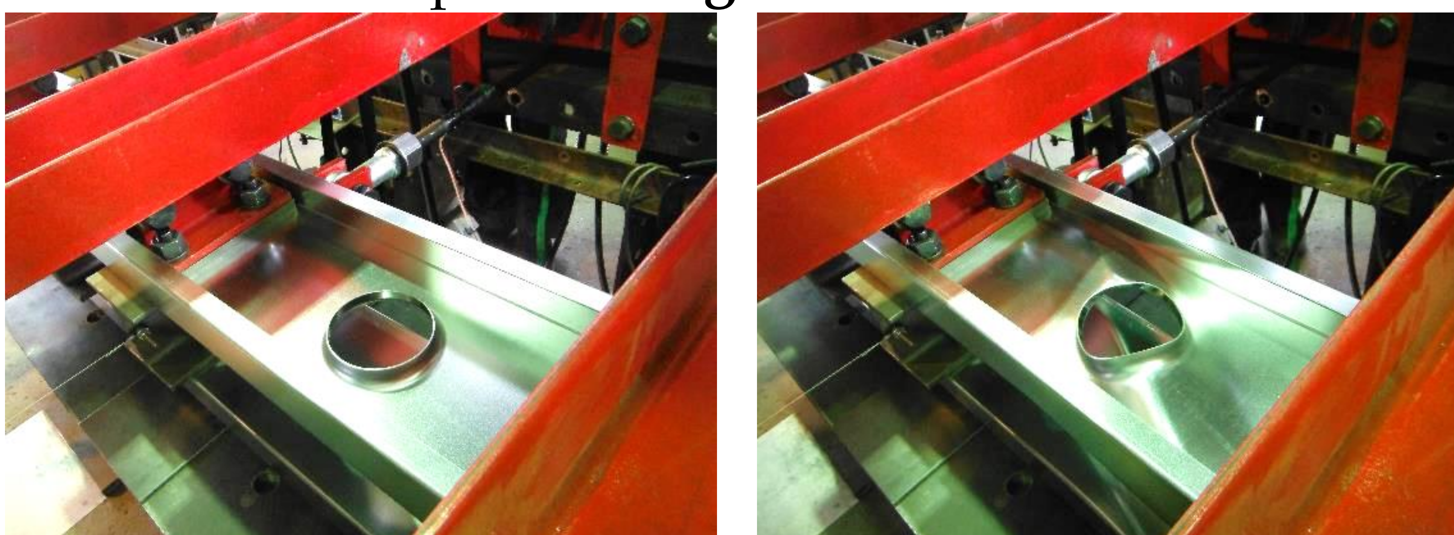
http://www.rewardwalls.com/

TEST RESULTS

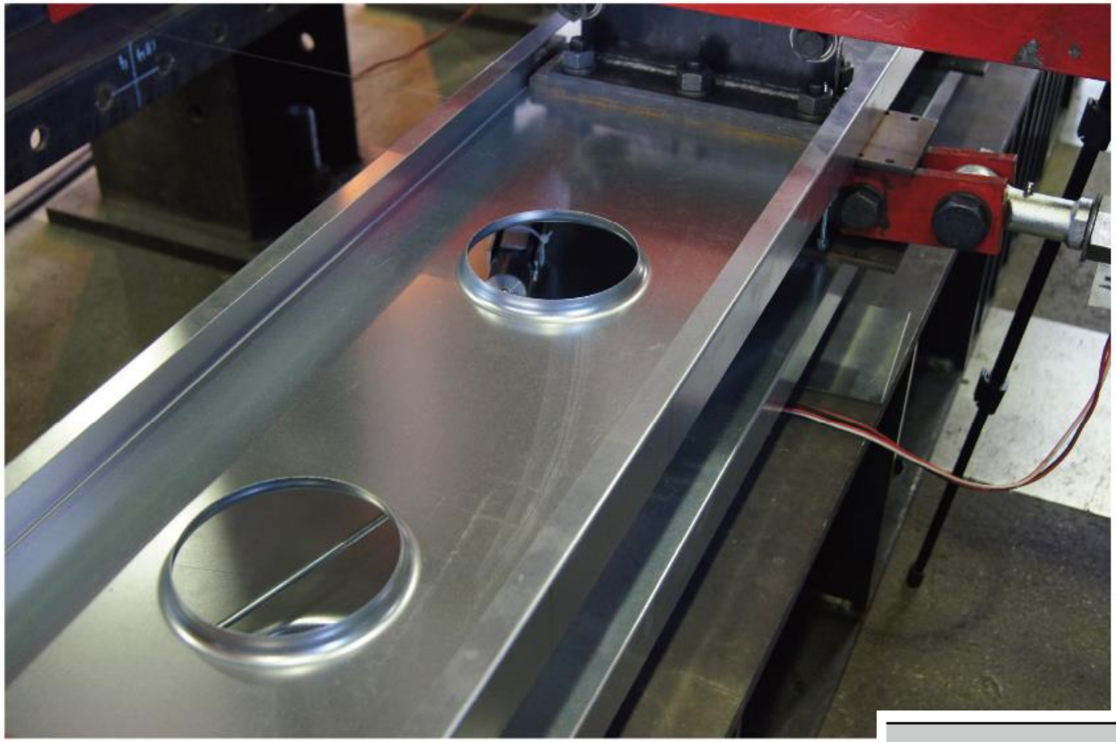
- Deformed Shape **-SIMPLE OPENING-**

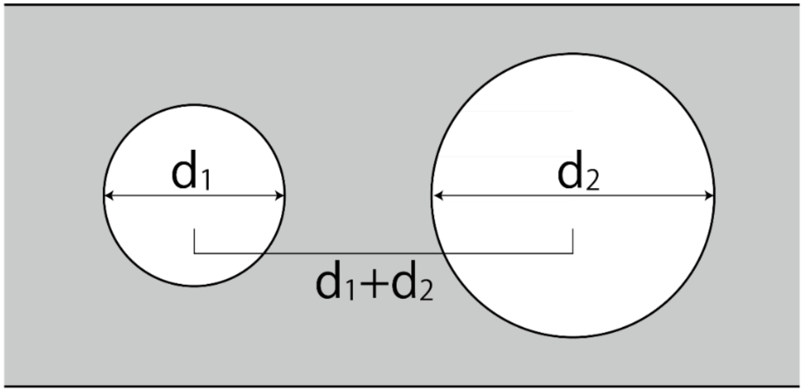


- Deformed Shape **-Burring OPENING-**

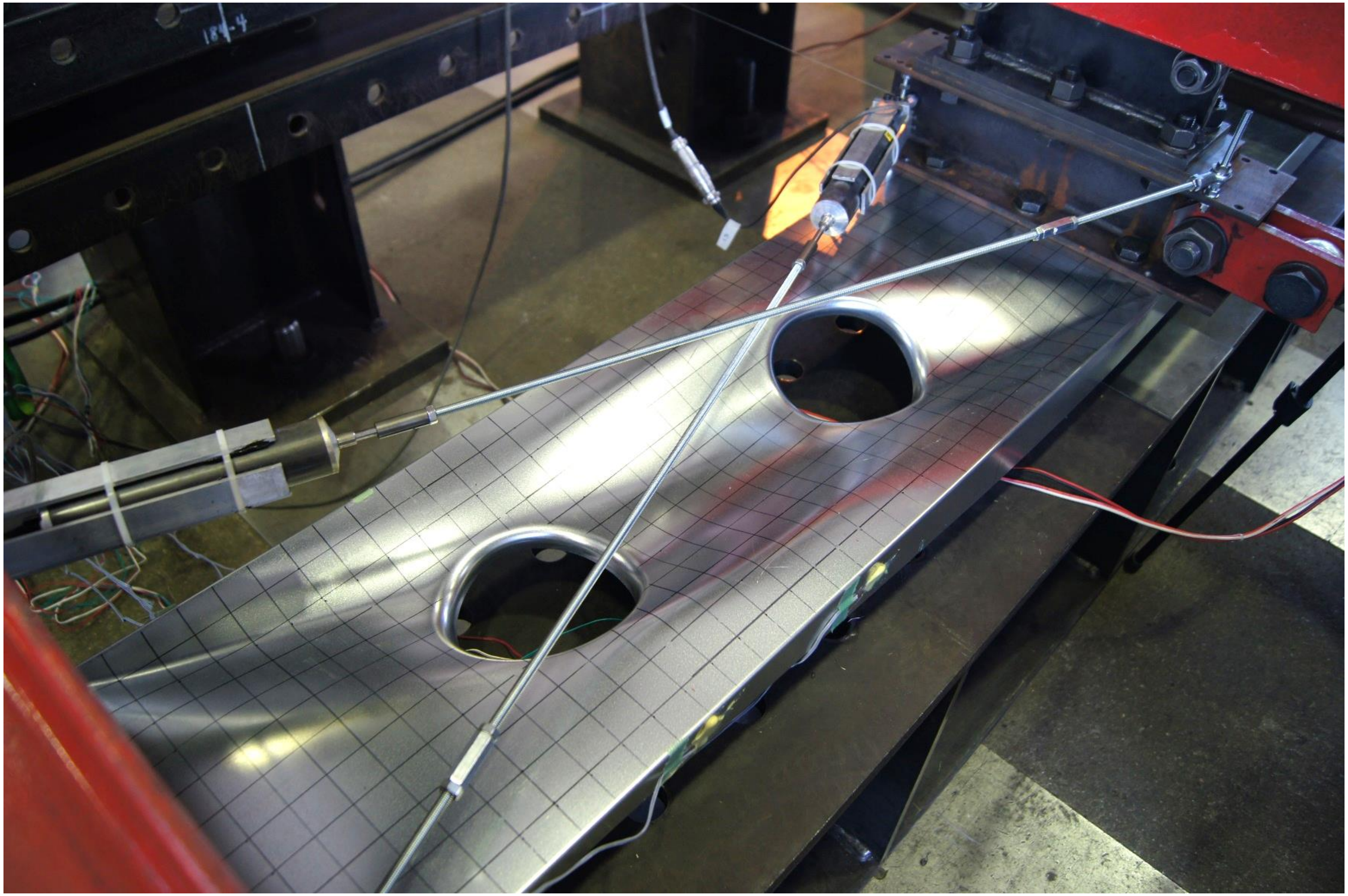


Aligned Burring Openings (Burring)

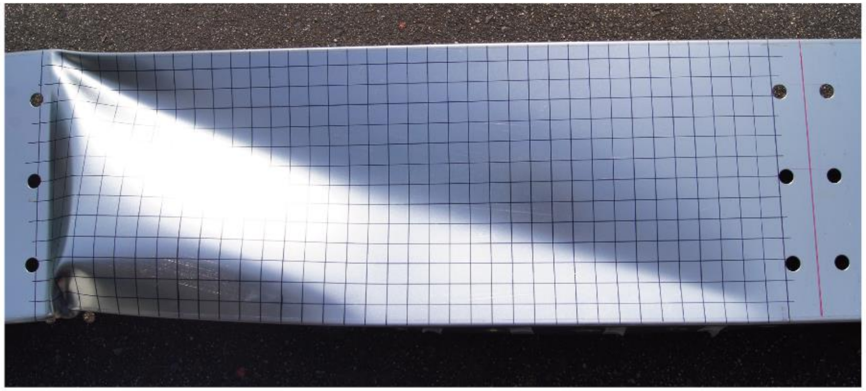




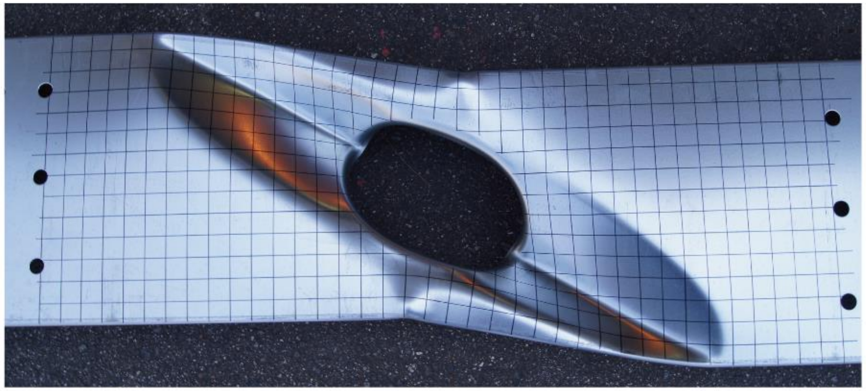
Aligned Burring Openings (Burring)



Aligned Burring Openings (Burring)

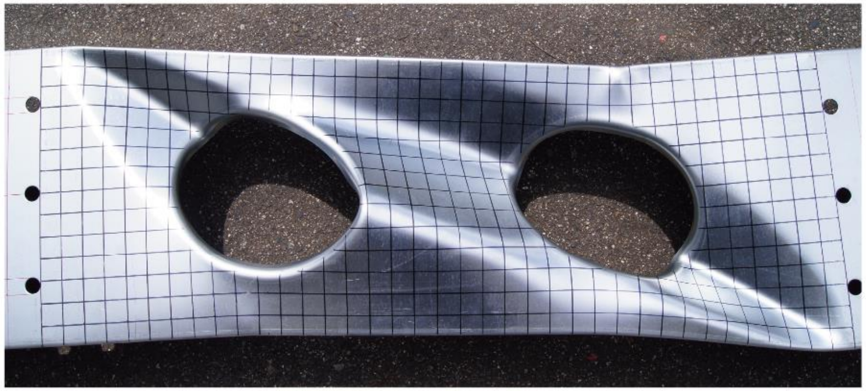


Full Web



Single Opening

Aligned Opening with Different size



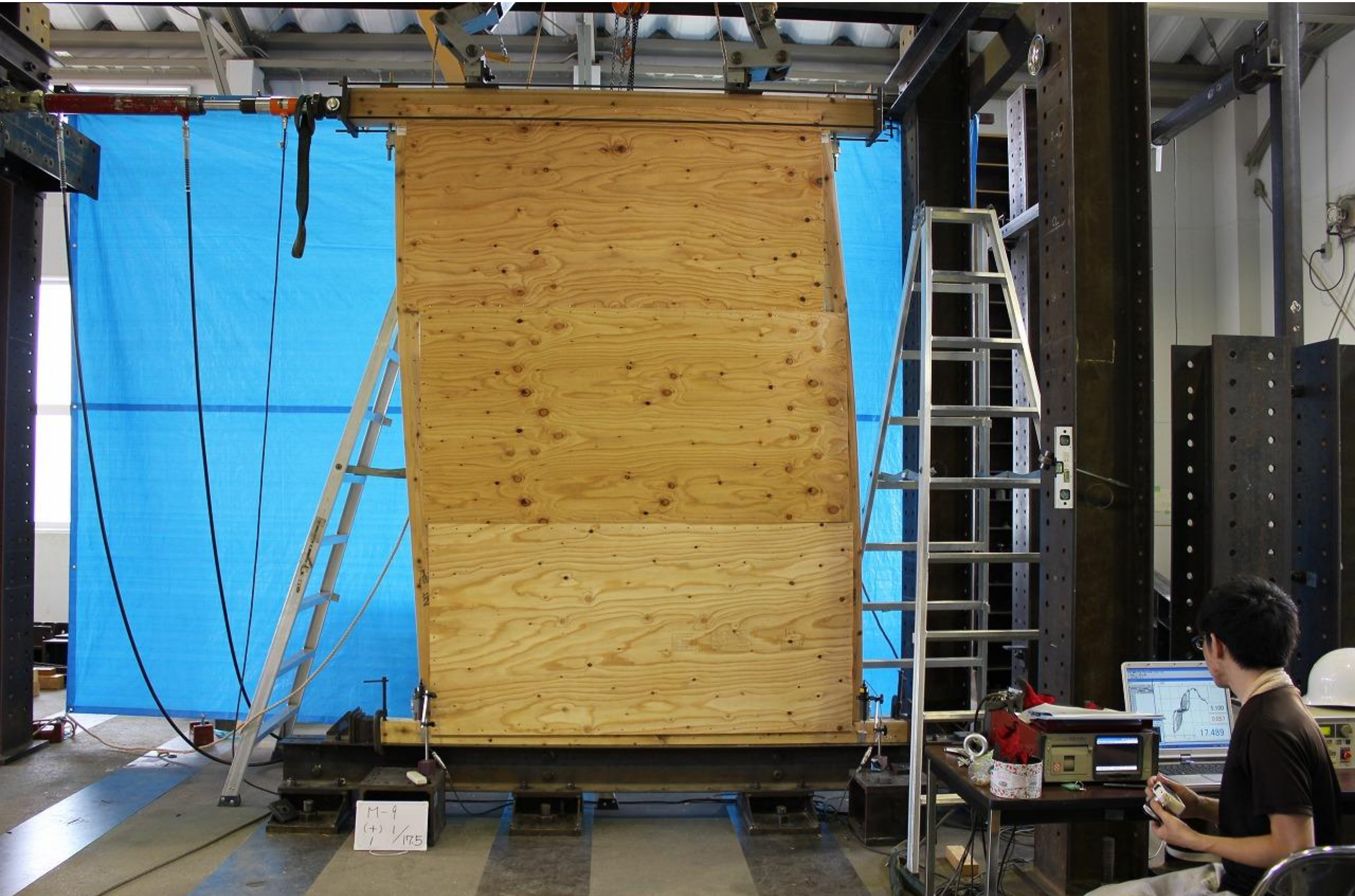
Aligned Opening with Same size

Burring Shear Wall System in Real Practice

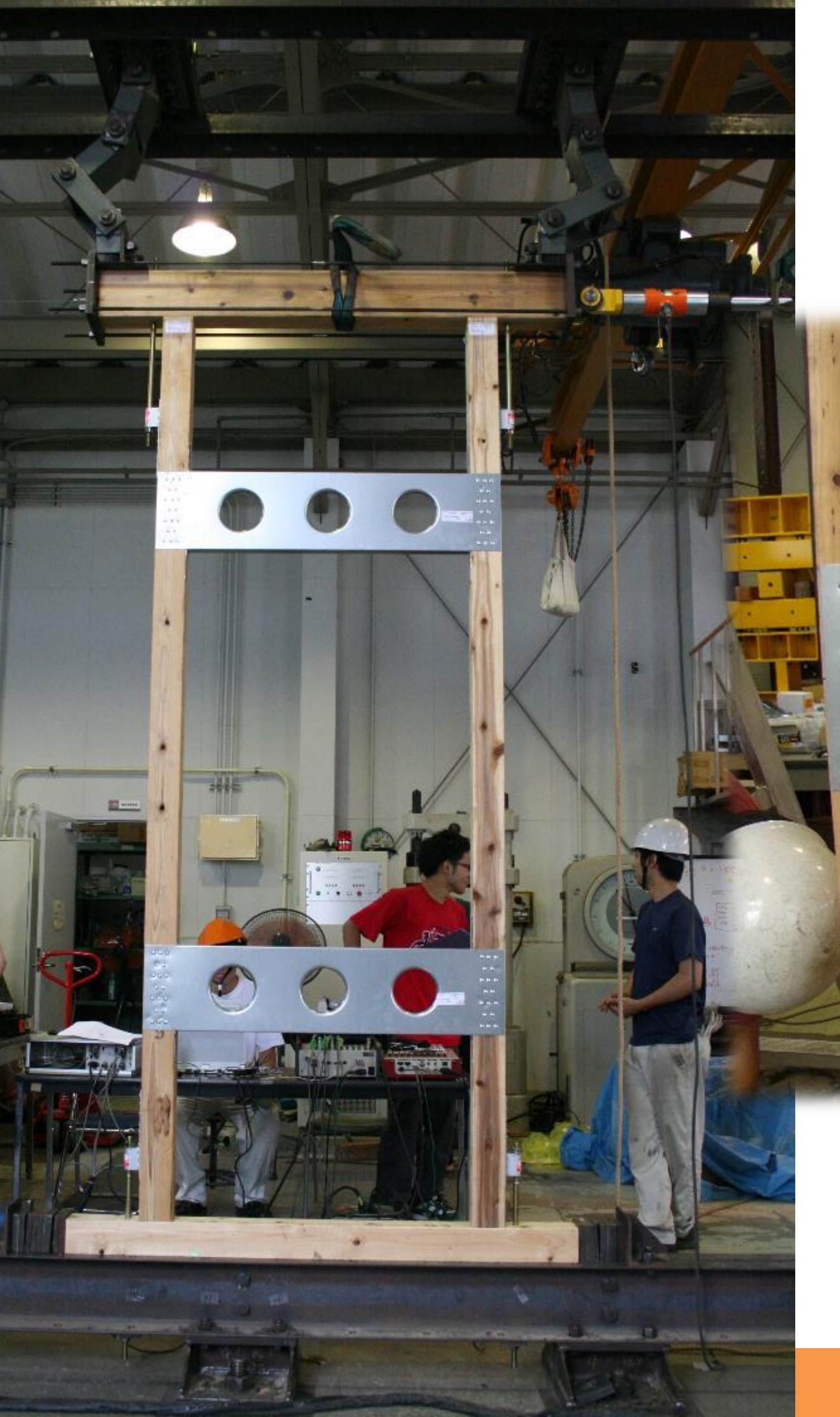
Burring Shear Wall System in Real Practice

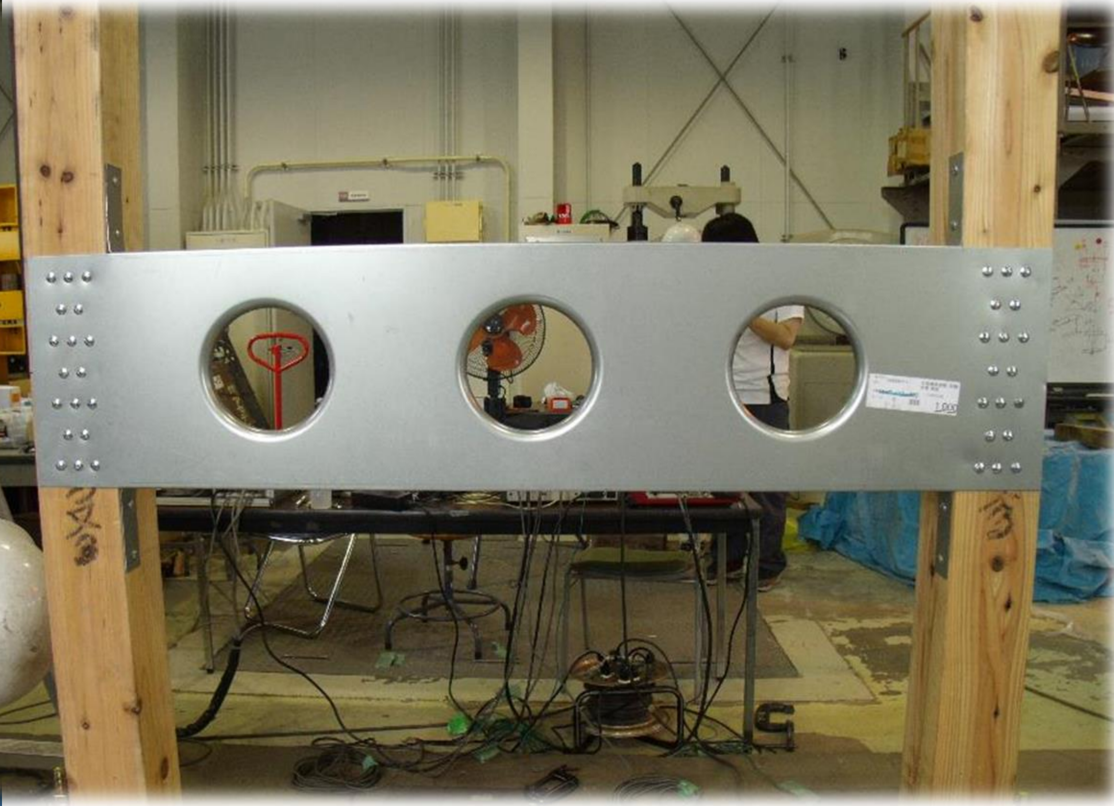
Burring Shear Wall System in Real Practice

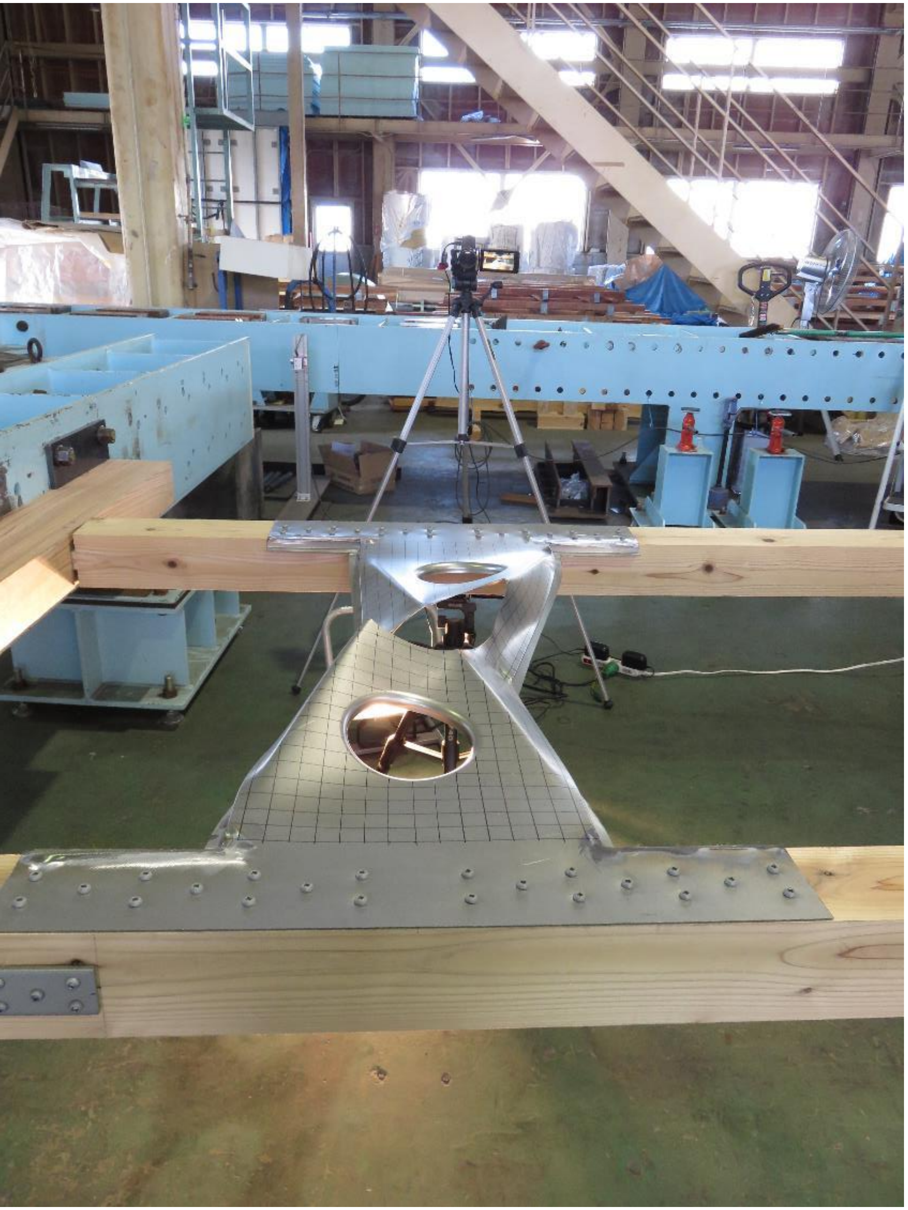
Timber Structure Retrofit Project

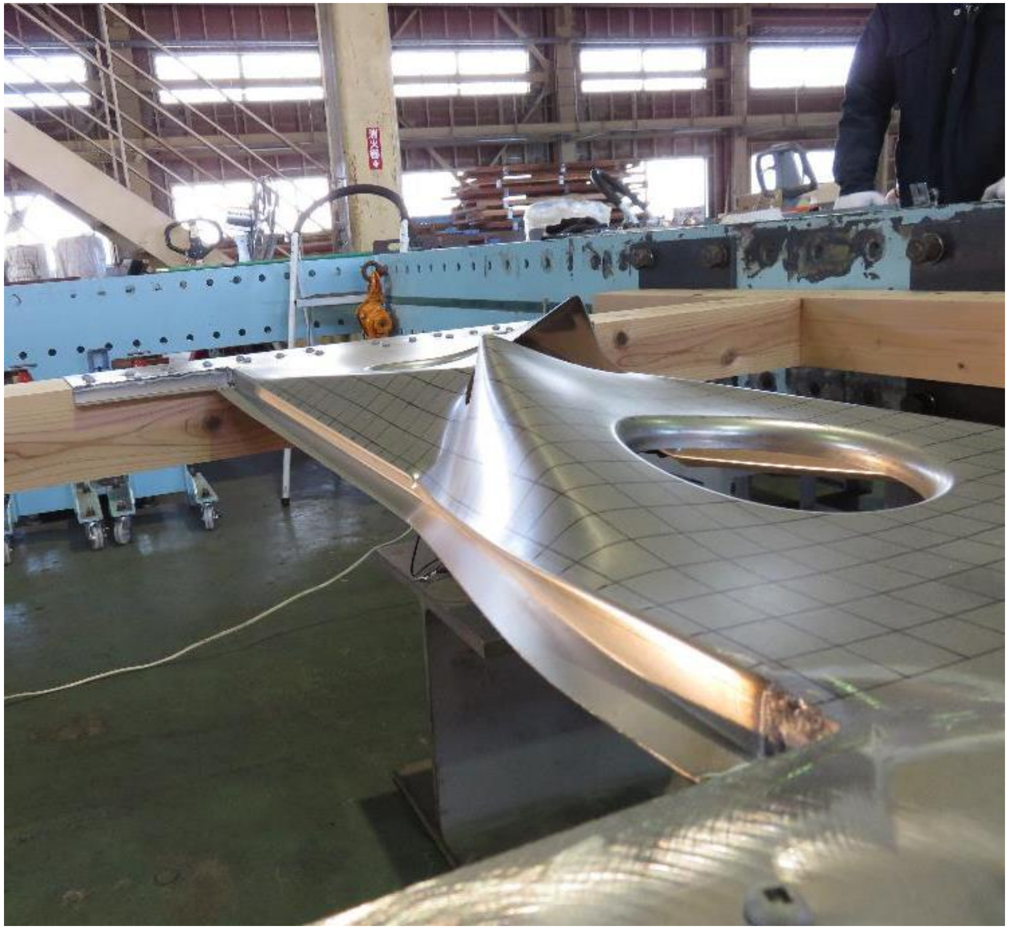

M-9
(+) 1/17.5
17.489


M-11-1
(+) 1/12.2

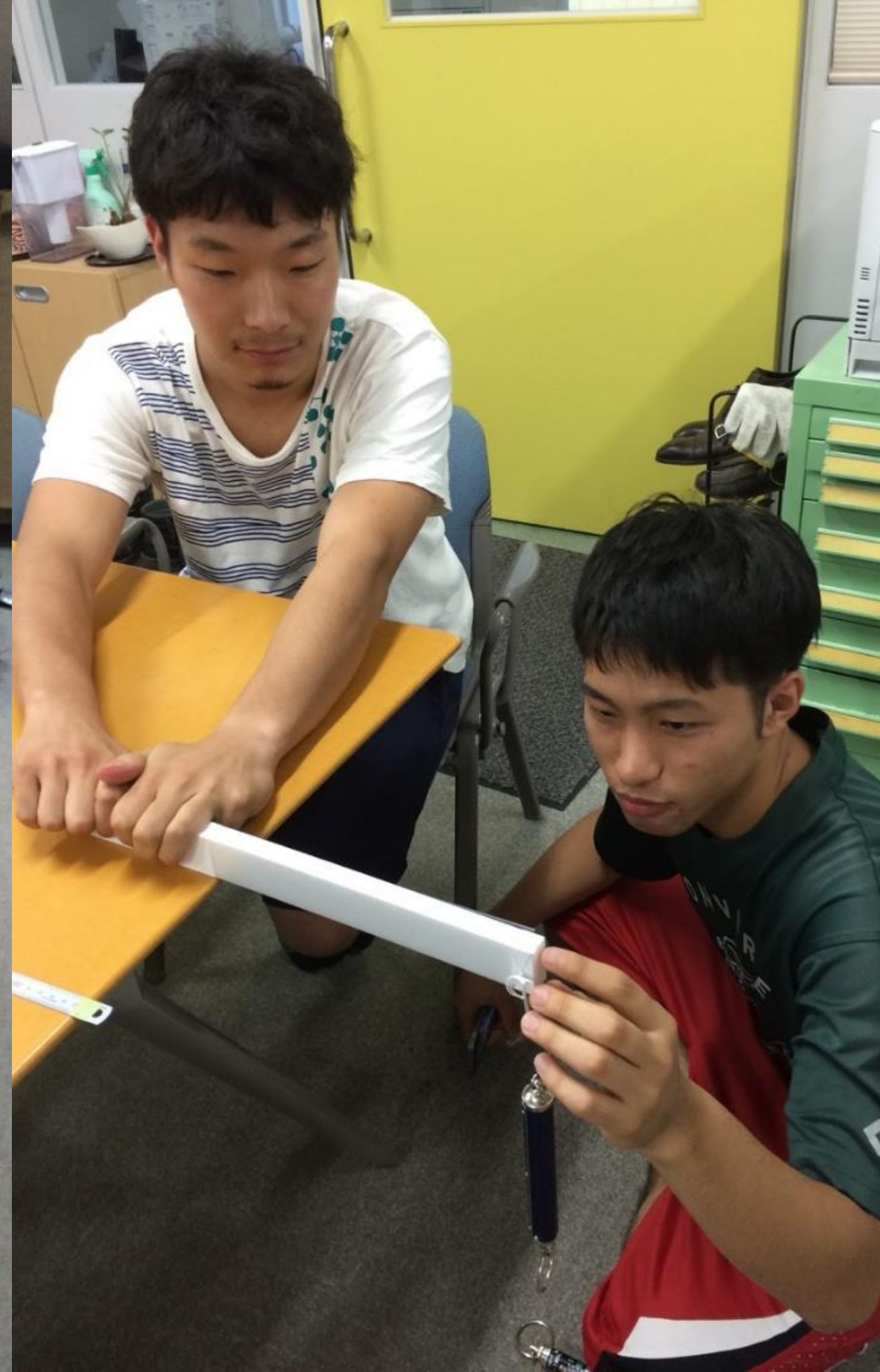








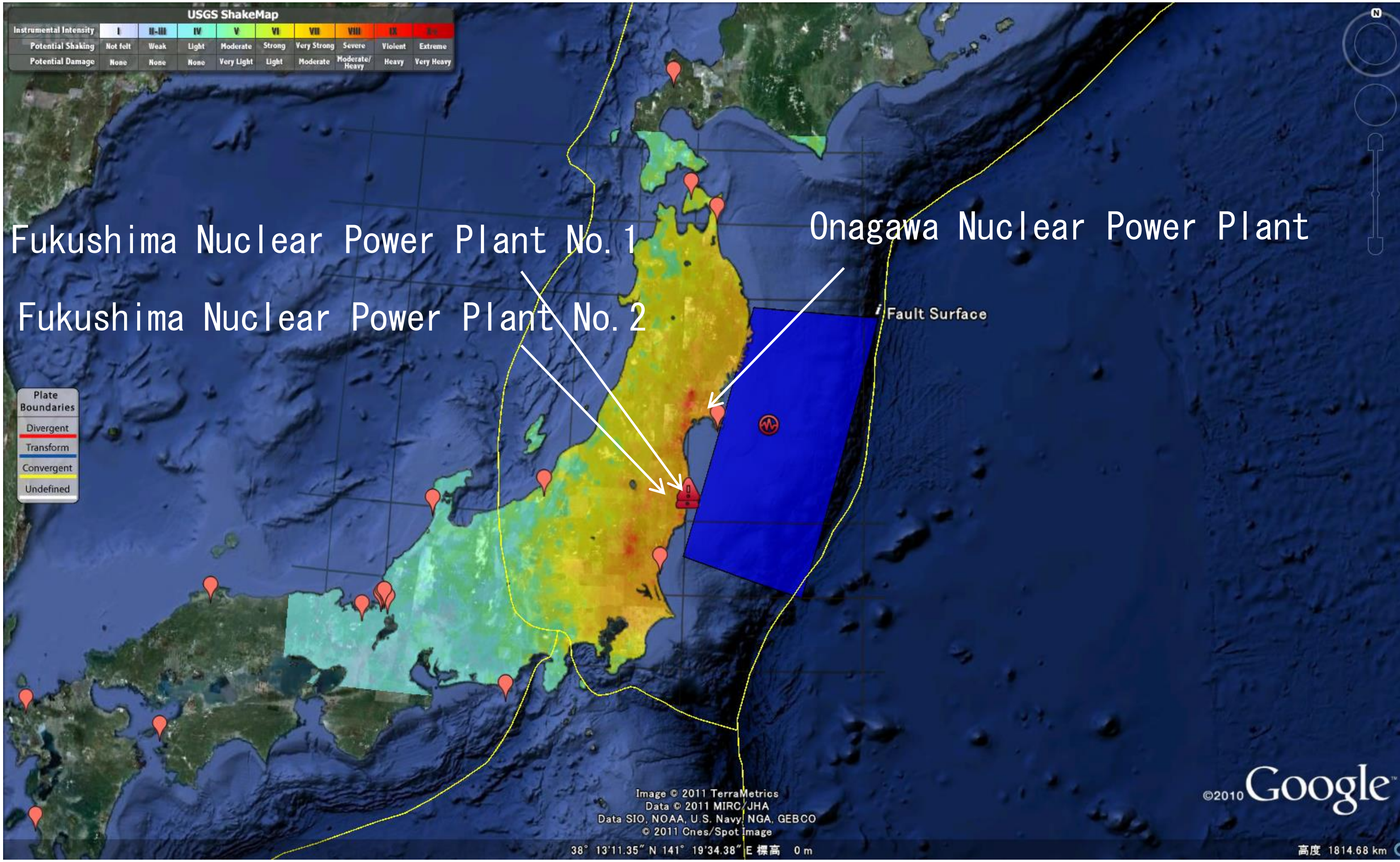
Typical Life Style in the Lab.
(example of Sato Lab.)



Thank you

Atsushi SATO

The 2011 off the Pacific coast of Tohoku Earthquake (March 11, 2011)



Structural Damage at Car Pool

Damage at Column Bases

Buckling at Braces

Damage at Braces

Reference：AIJ：2011 Tohoku Earthquake progress Report,2011.7

Minatomachi

Minatomachi

Minatomachi