EPFL

Seismic stability of steel moment resisting frames: recent findings and implementation in design standards

SSEF Structural Steel Education Forum

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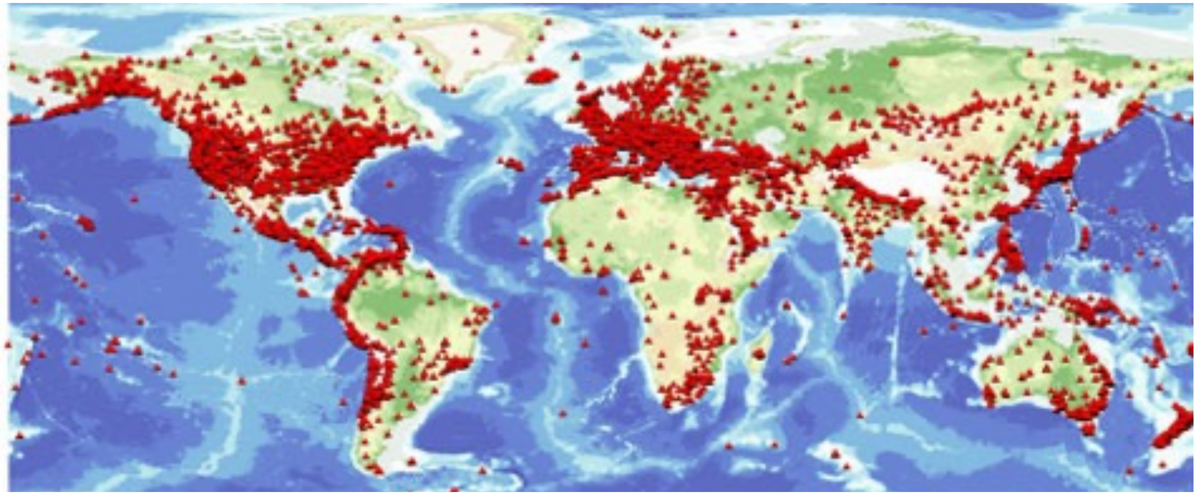
Structural Engineer, INGPHI, Switzerland

✓ Andronikos Skiadopoulos, PhD, EPFL
Post-doctoral scientist, Stanford University, USA

Earthquakes worldwide



@BGS earthquake seismology

@the Guardian (Central Italy)

Key statistics worldwide

Human Casualties: 28’000 / year

Displaced Population: 317’500 / year

Source: UN Office for Disaster Risk Reduction

Underlying physics

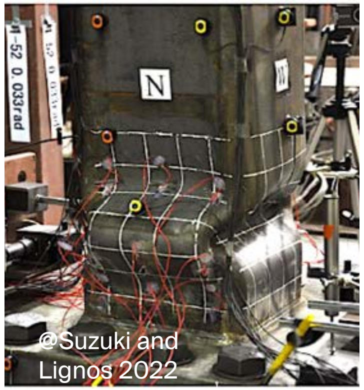
External force		Inertia forces		Damping forces		Restoring forces

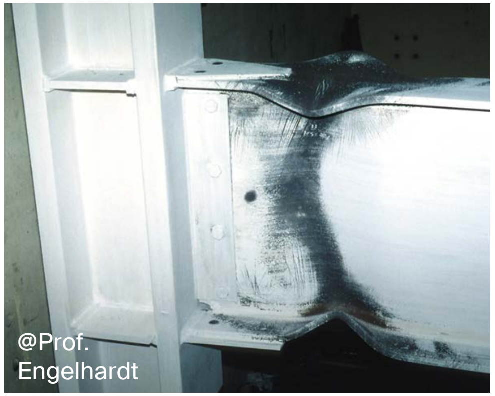
$$-\mathbf{m}\ddot{u}_g(t) = \mathbf{m}\ddot{\mathbf{u}}(t) + \mathbf{f}_D(\mathbf{u},\dot{\mathbf{u}}) + \mathbf{f}_S(\mathbf{u},\dot{\mathbf{u}})$$

Restoring forces → Highly nonlinear

Primary challenges in simulating collapse:

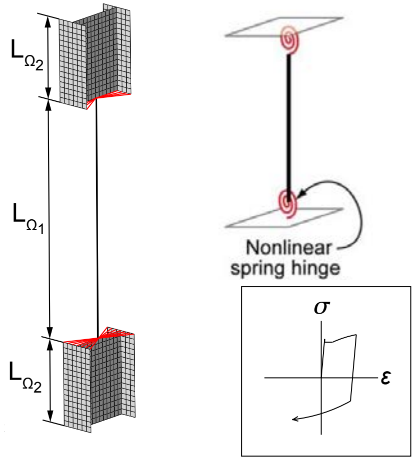
- Material nonlinearity
- Nonlinear geometric instabilities (→ softening)
- Scarcity of available data at large deformations





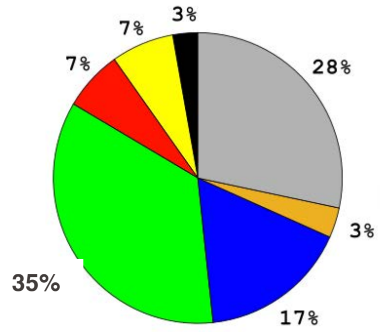
Resilient Steel Structures Lab @ EPFL in a nutshell

Nonlinear simulation models

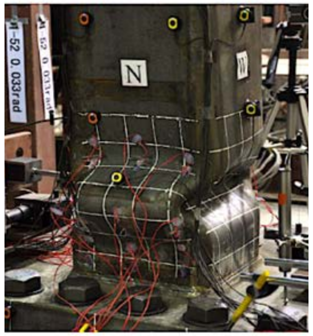


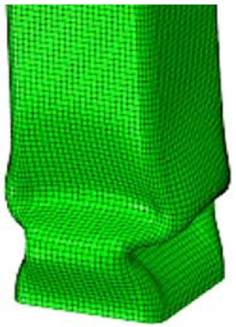
Methods for simulation-assisted infrastructure assessment

Total financial losses

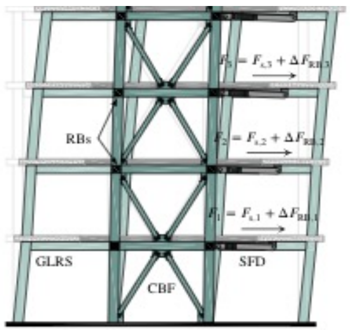


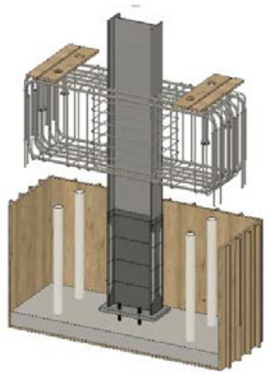
Multi-scale experimentation





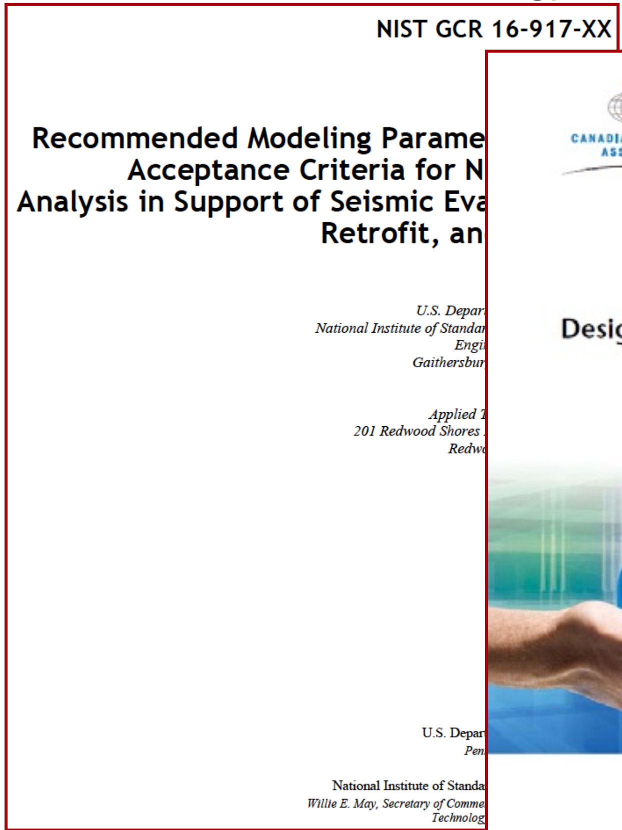
New concepts for minimal structural damage



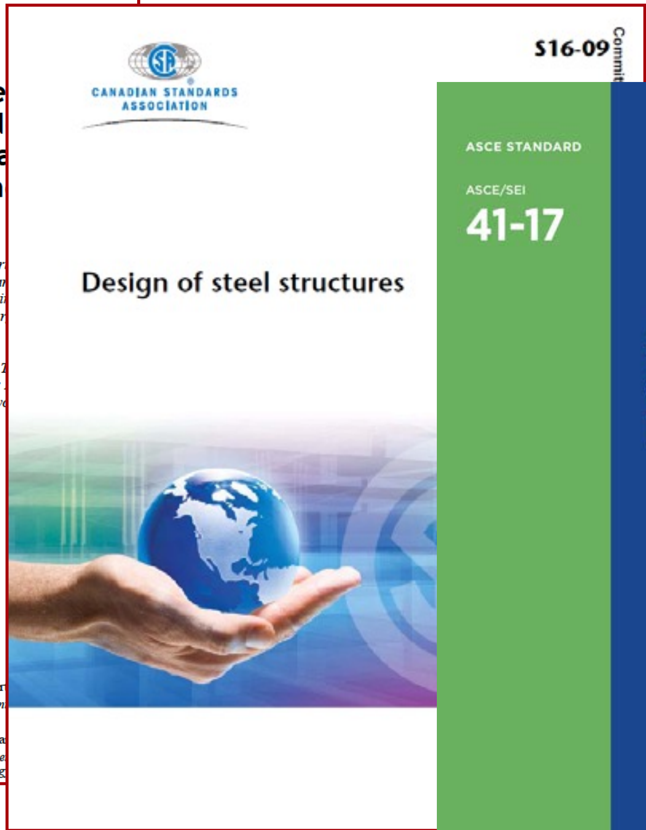


Contributions to international standards

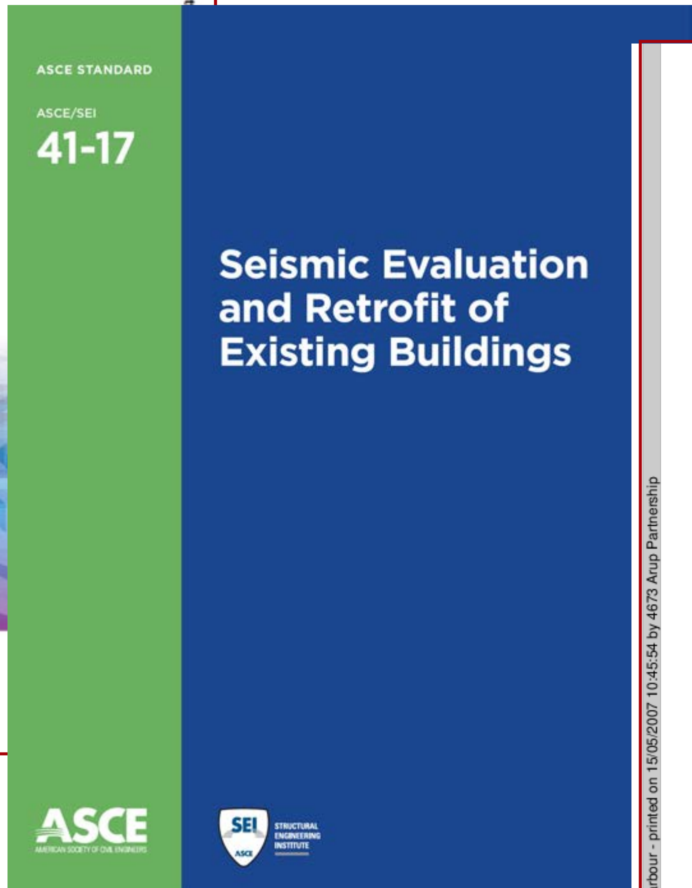
National Institute of Standards and Technology



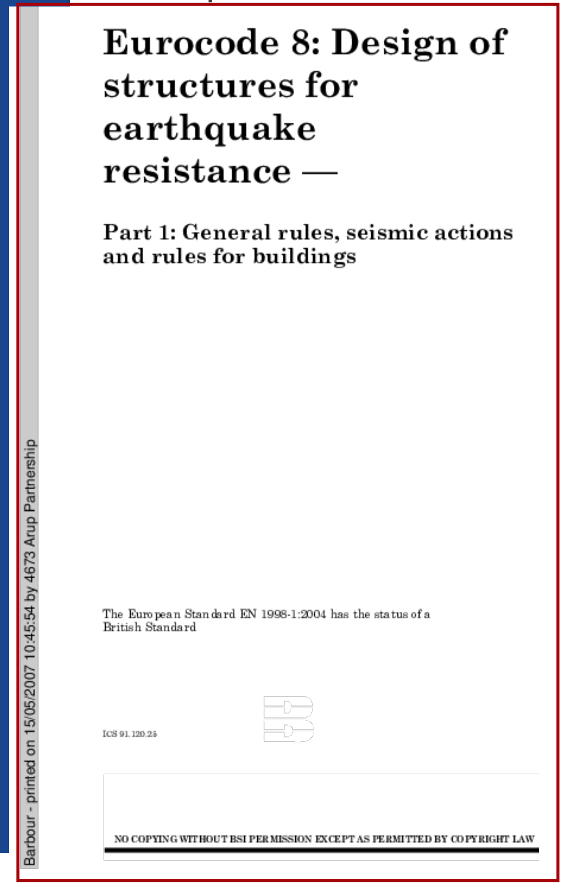
Canadian standard



US standard



European standard



Our focus for today: Steel moment frames

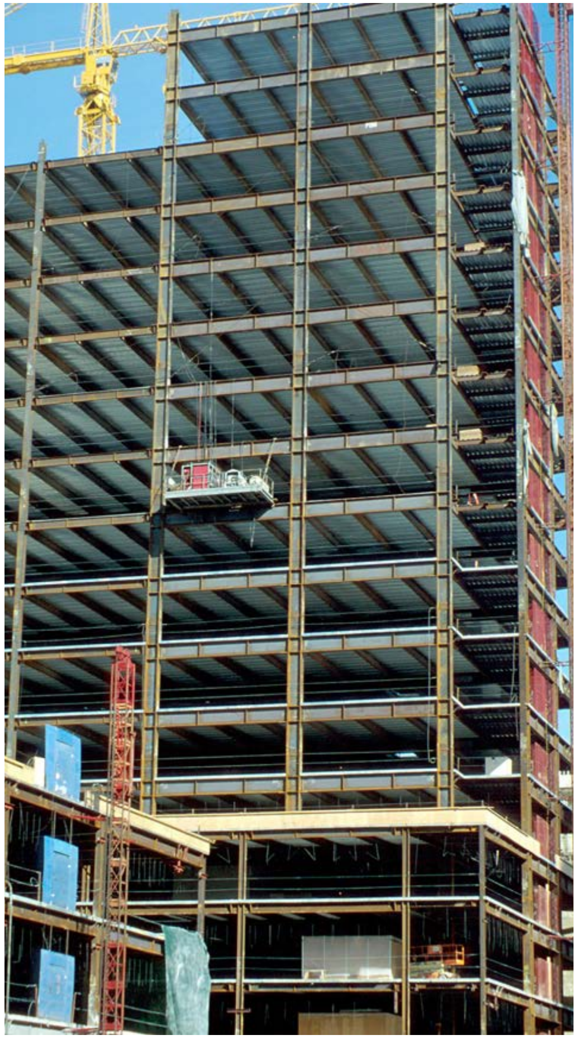
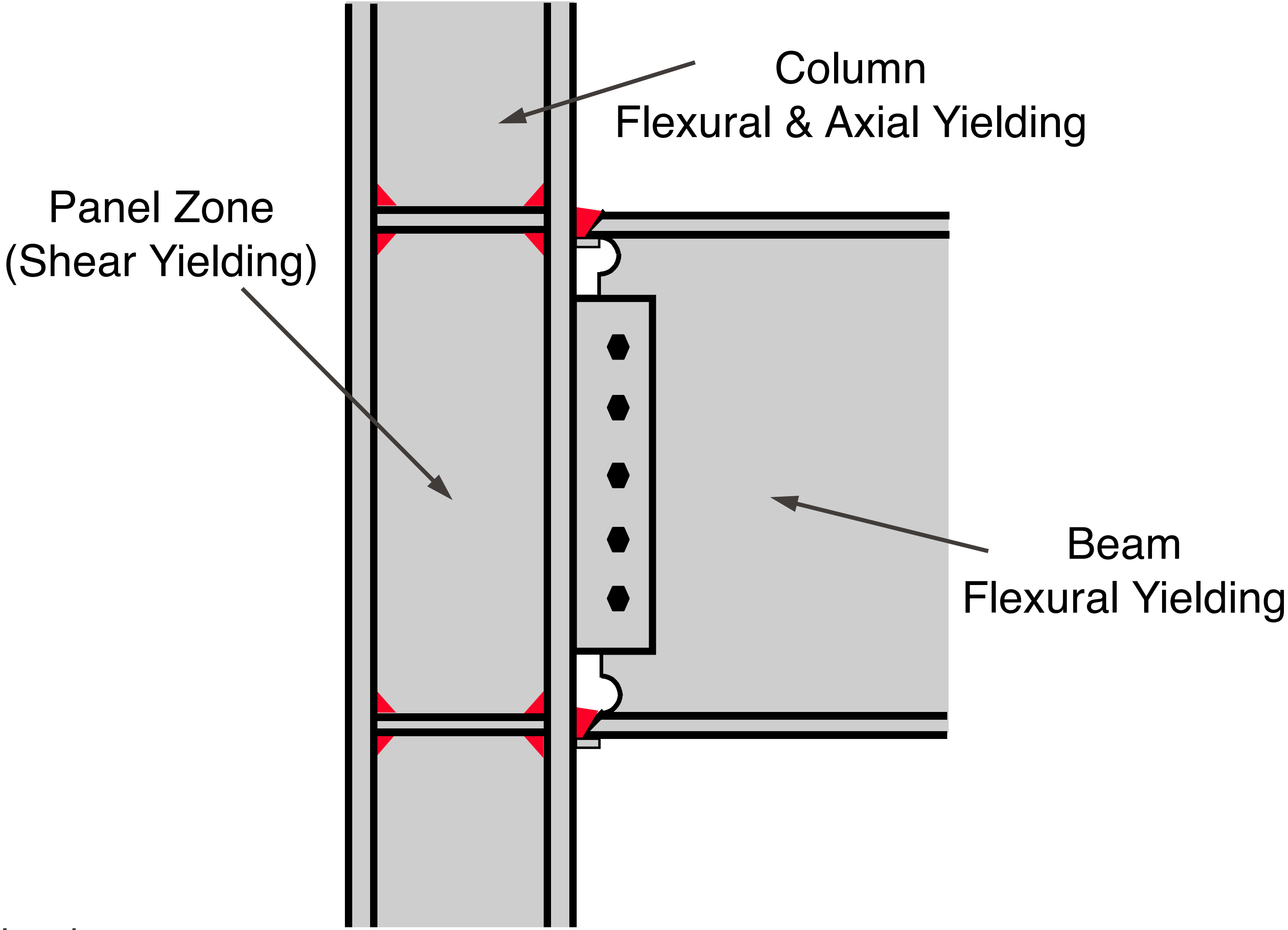


Image courtesy of Prof. M. Engelhardt



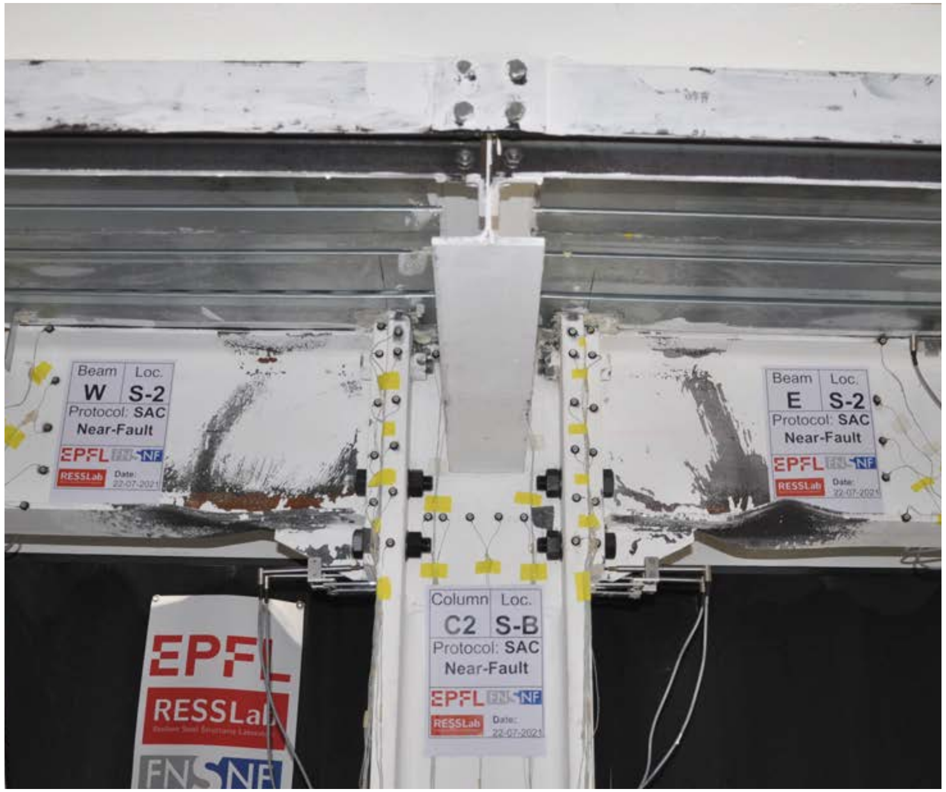
Steel moment frames

-Primary structural elements (our focus for today)

Steel columns

Panel zones

Composite effects



Images @Prof. Lignos

Motivation: Steel moment frames

- Design considerations: primarily based on cyclic tests from pre-qualified moment connections (after Northridge 1994).

- Emphasis was mostly on connection performance up to a lateral drift ratio of 4% rad.

- Stiffness typically controls column design to achieve the target design drifts.

- Often deep ($d > 400mm$) and slender members are commonly used (weight consideration).

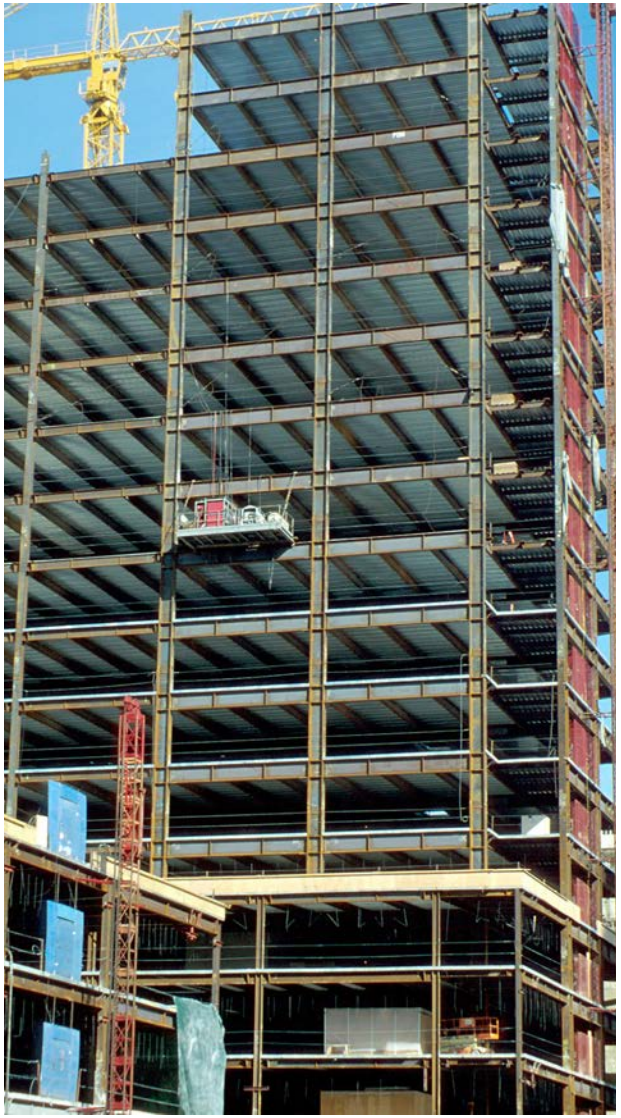
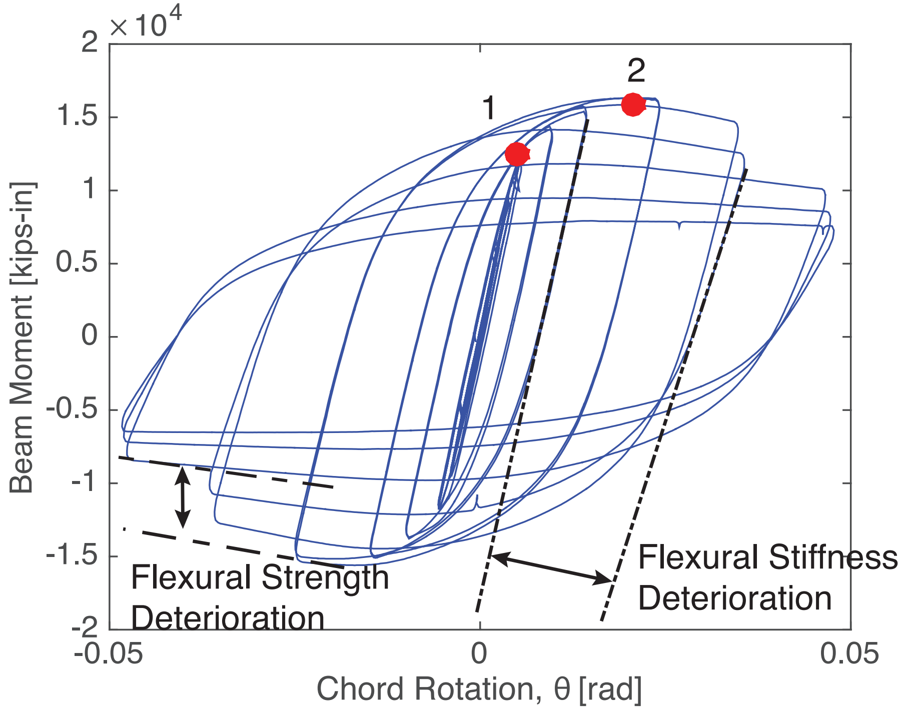


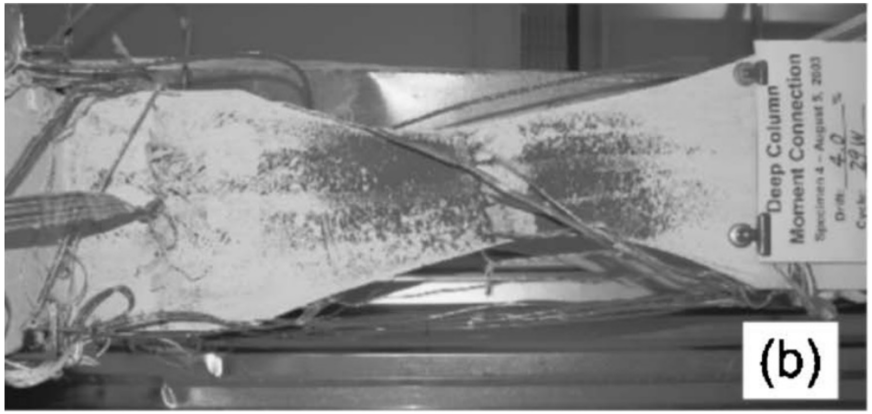
Image courtesy of Prof. M. Engelhardt

Beam-to-column connections
-Key performance aspects

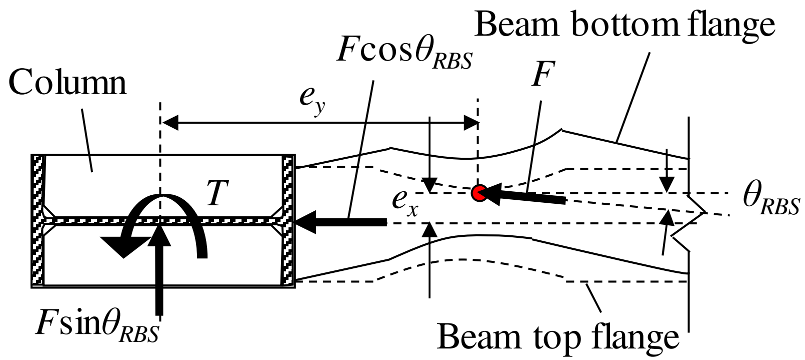
Image courtesy of Prof. M. Engelhardt



Key aspects on steel column demands

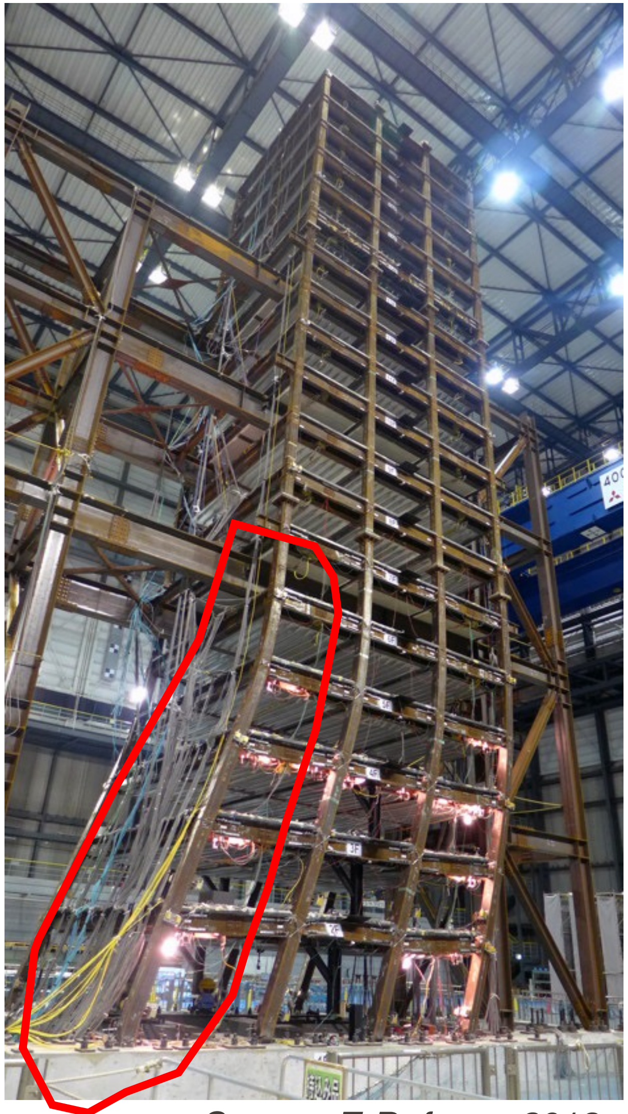


Zhang and Ricles (2006)

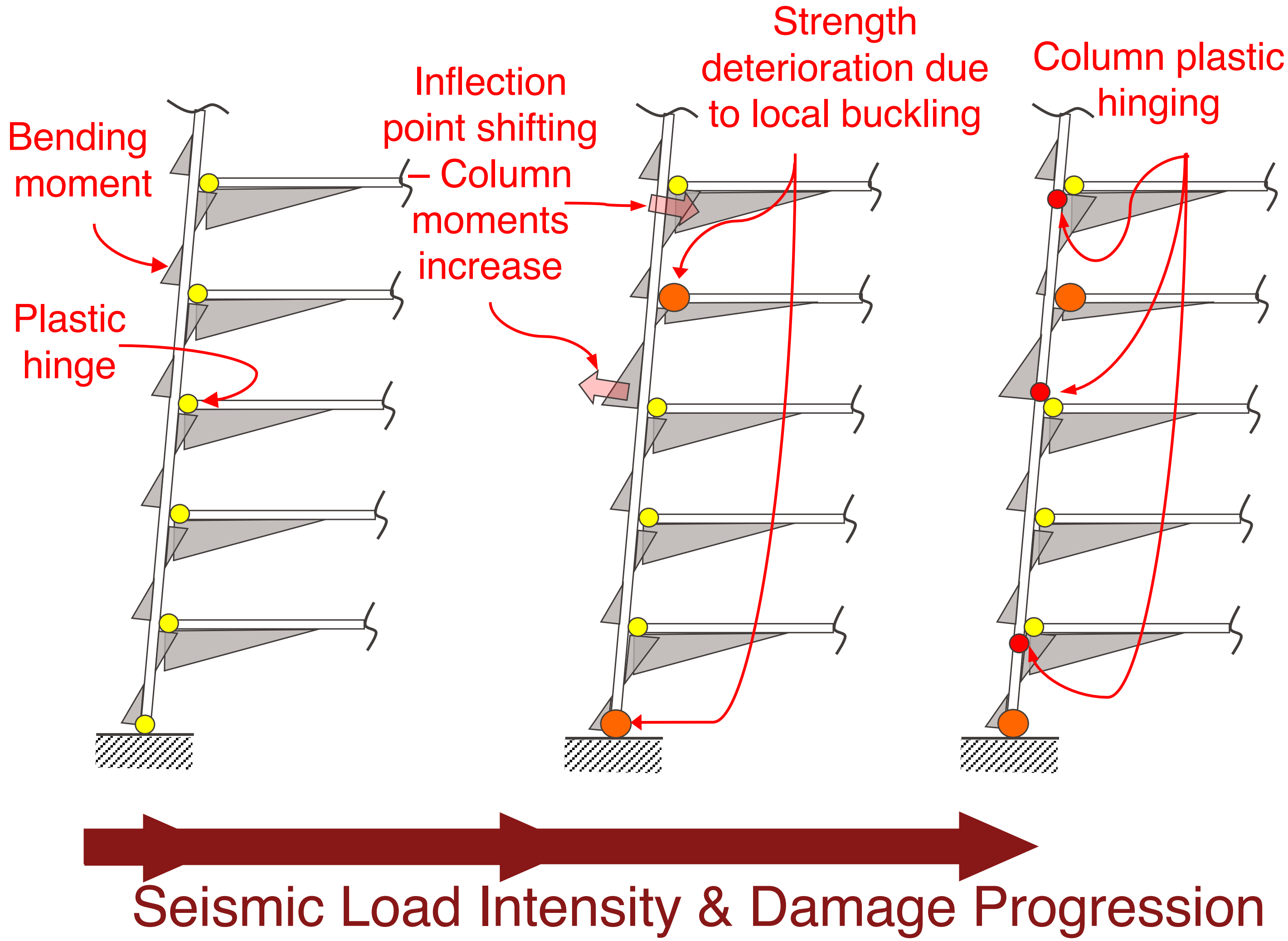


Inamasu, Kanvinde and Lignos (2019)

System behavior to earthquake-induced collapse



Source: E-Defense 2013

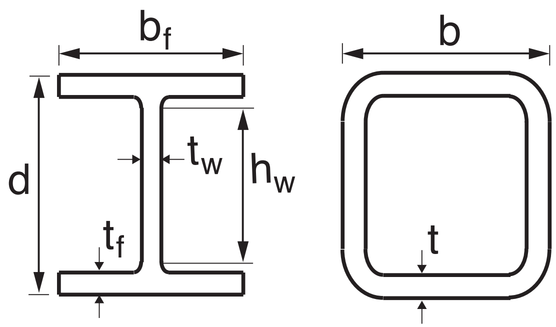


Available experiments on steel columns (till 2010)

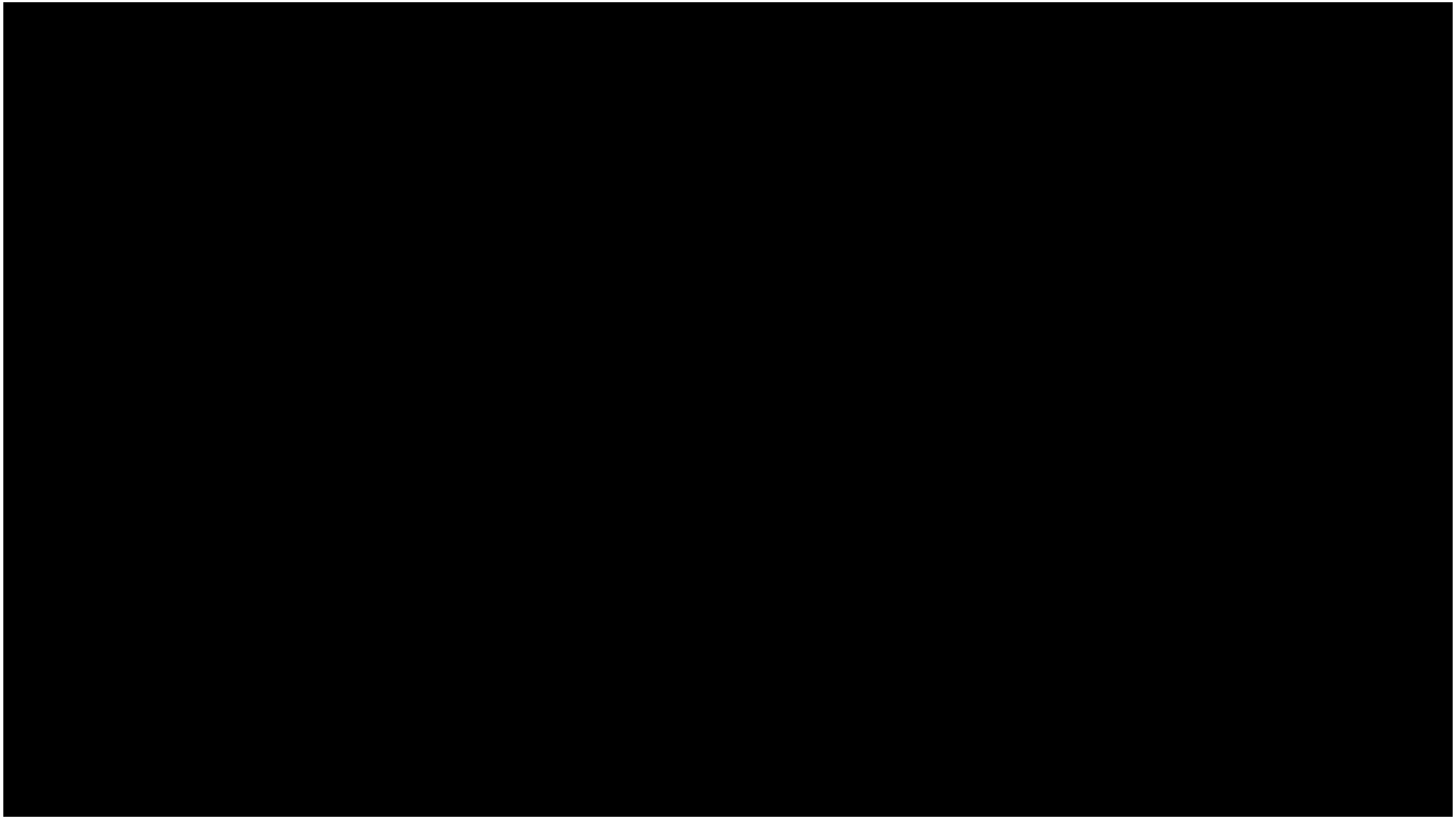
RESEARCHER	SECTION SIZE	P/P_y	LATERAL LOAD
Popov et al. (1975)	W8x24 W8x48	0.3 ~ 0.80	Cyclic
MacRae et al. (1990)	250 UC 73 (W10x49)	0.0 ~ 0.80	Cyclic
Nakashima et al. (1991)	W4x13 W5x19 W6x9	0.0 ~ 0.60	Monotonic
Newell and Uang (2006)	W14x132 W14x176 W14x233	0.0 ~ 0.75	Cyclic

Testing matrix for full-scale column testing

Section Size	Steel Type	$b_f/2t_f$	h_w/t_w	Loading Scheme	Number of Specimens
W14x53	A992 Gr. 50 (Equiv. S355)	6.1	34.1	Cyclic/Unidirectional	6
W14x61		7.7	33.7	Cyclic/Unidirectional	6
W14x82		5.9	24.6	Cyclic/Unidirectional	6
W16x89		5.92	27	Cyclic/Unidirectional	6
W24x84		5.9	45.9	Cyclic/Unidirectional/Bidirectional	4
W24x146		5.9	38.0	Cyclic/Unidirectional/Bidirectional	6
HSS-250x9.5	ASTM A500 F_y=400MPa	26.7		Cyclic/Uniaxial	6
HSS-300x16		19.0		Cyclic/Uniaxial	6

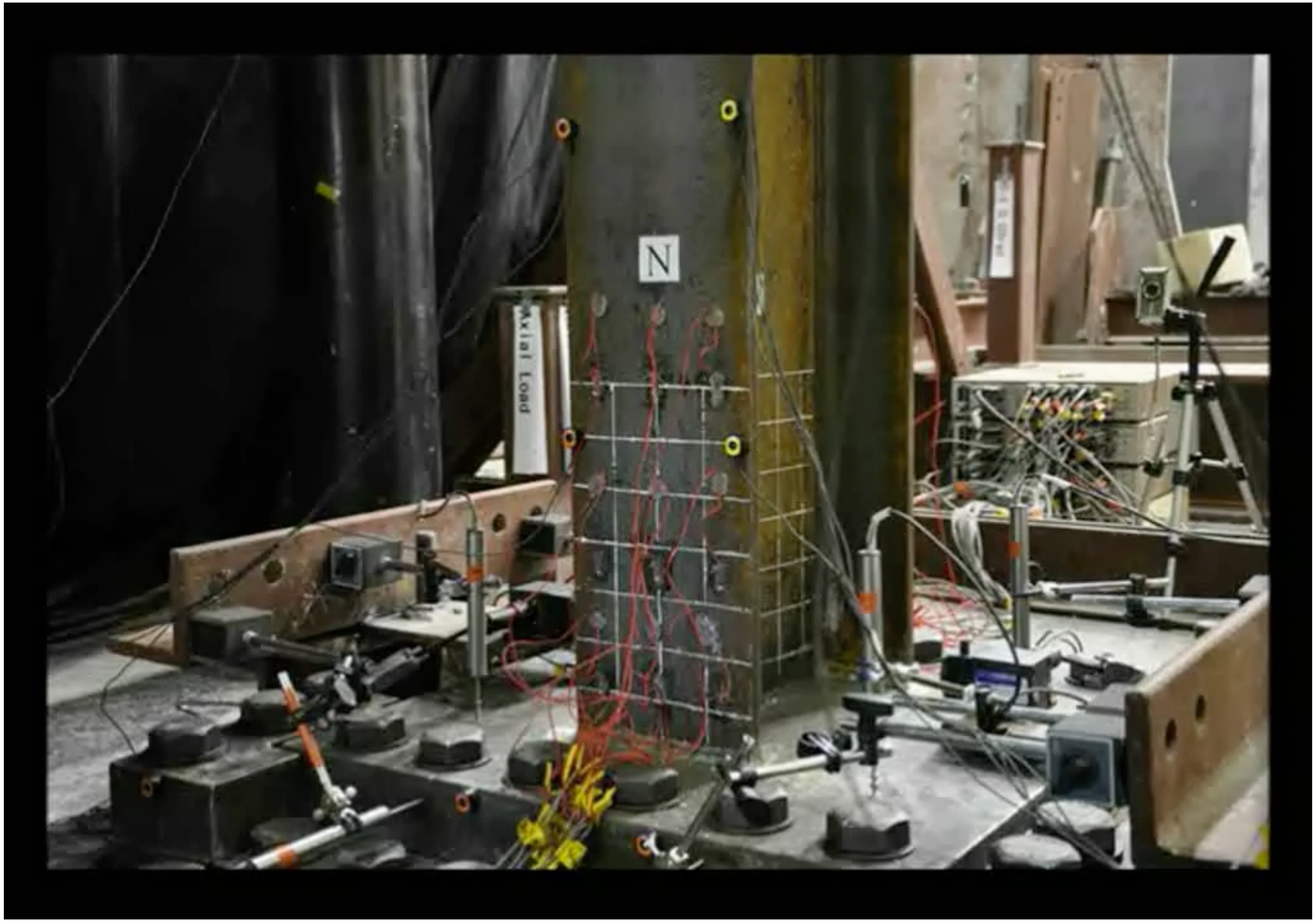


Full-scale steel column experiment



Source: Elkady and Lignos (2018)

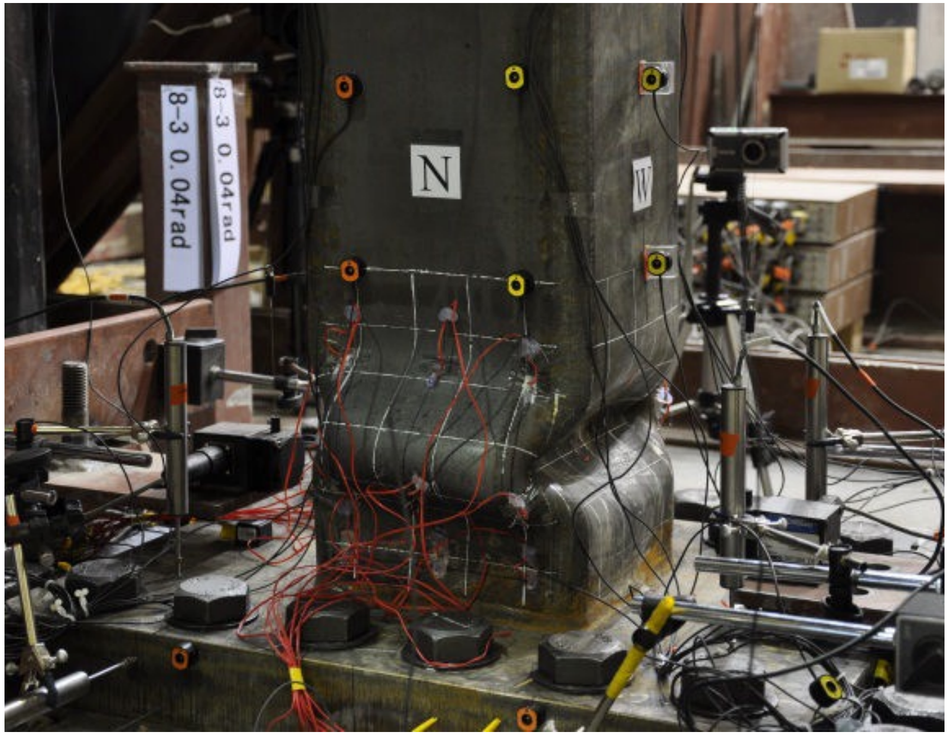
A closer view of column axial shortening

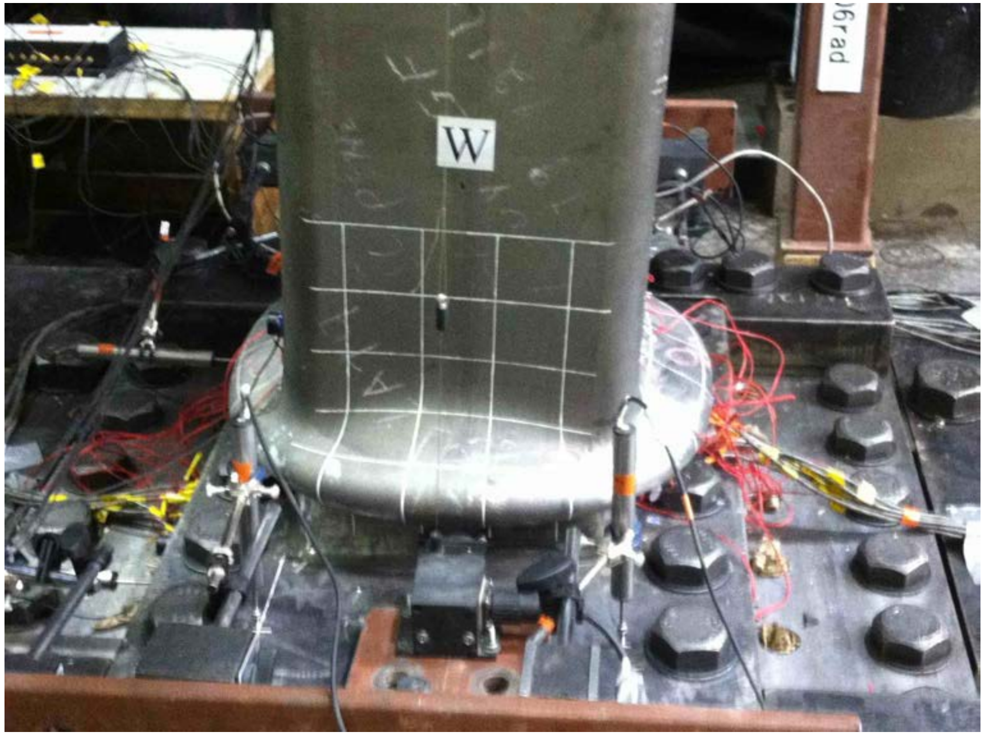


Suzuki and Lignos (2015)

Effect of cross-sectional shape

-The issue of column axial shortening



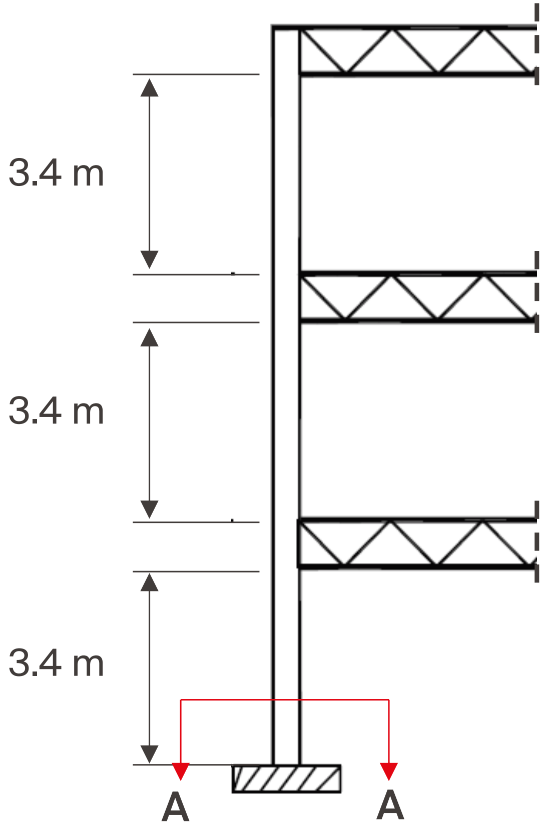


Suzuki and Lignos (2015)

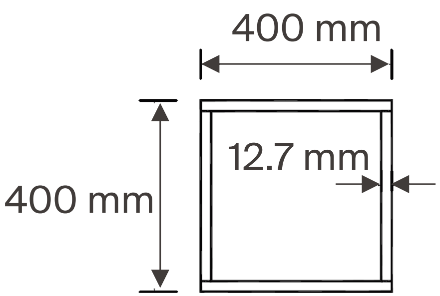
Observations from field reconnaissance

-2017 Puebla-Morelos earthquake, Mexico City

Building elevation



Section A-A



Column cross section
(Seismically compact)

Fixed-end column base

4 days after the earthquake

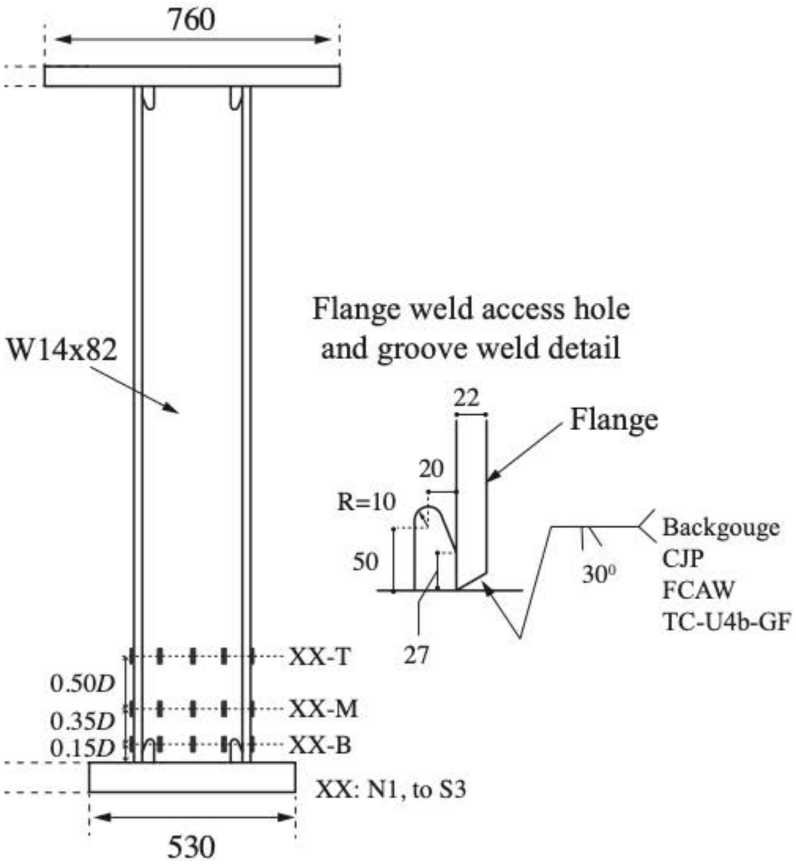
51 days after the earthquake

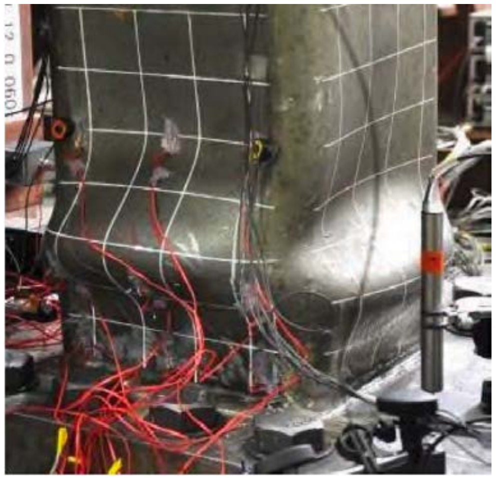
Complete Squashing

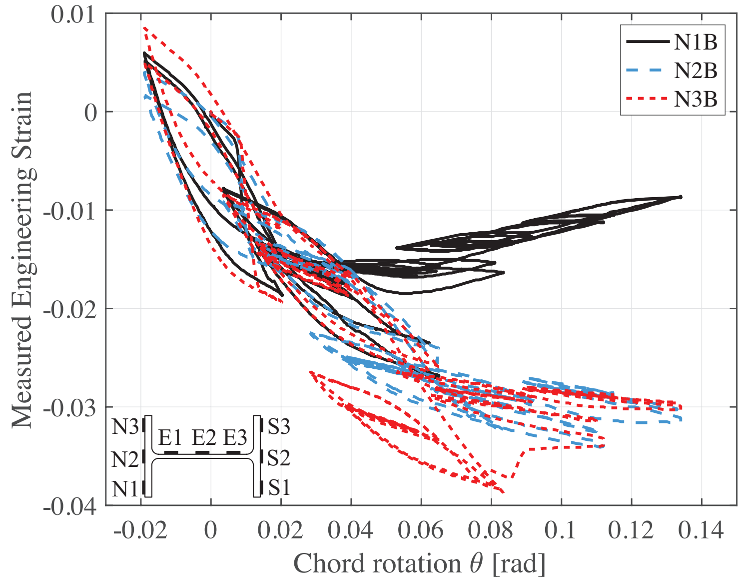
Residual story drift was nearly zero!

Tapia-Hernández and García-Carrera (2020)

Implications on weld demands & weld details



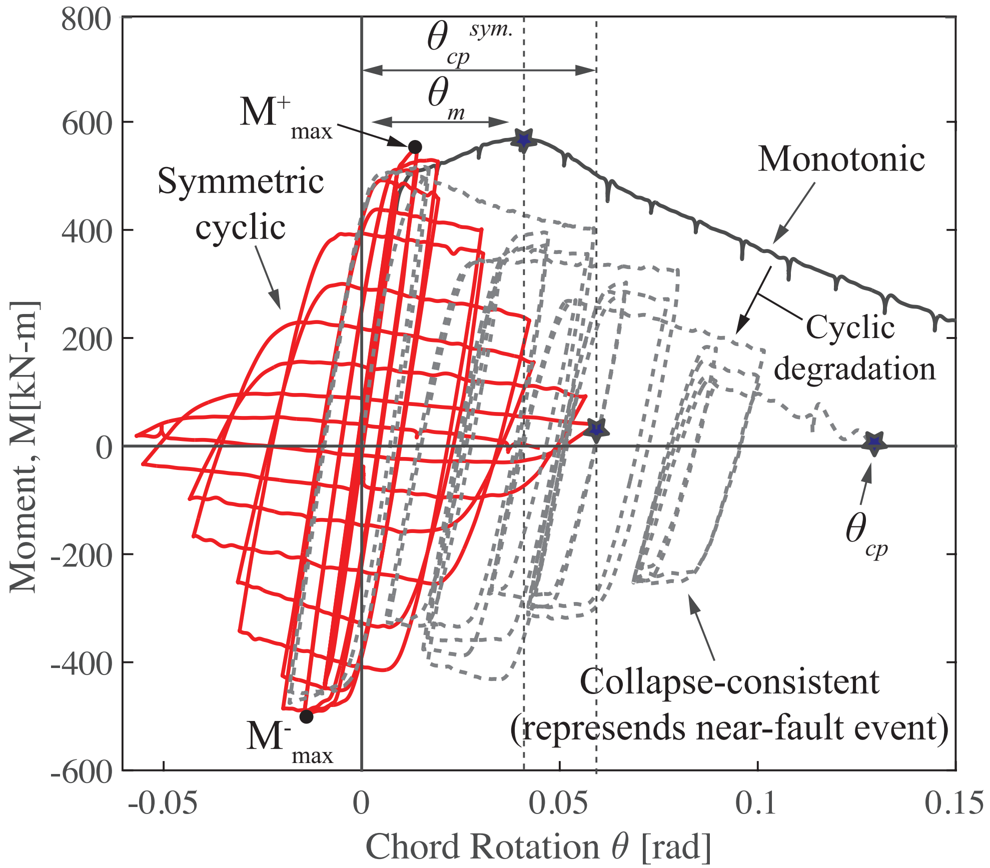


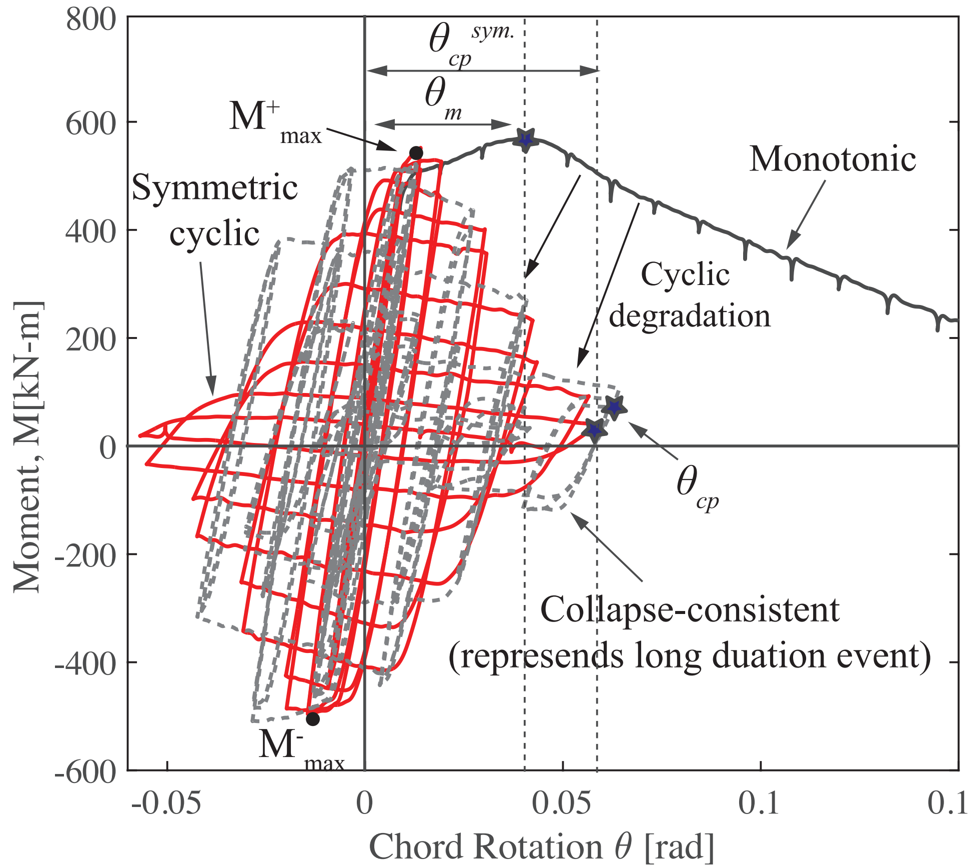


(Suzuki and Lignos, 2021)

*Suzuki, Y., Lignos, D.G. (2021). "*Experimental Evaluation of Steel Columns under Seismic Hazard-Consistent Collapse Loading Protocols*, ASCE Journal of Structural Engineering, Vol. 147(4), pp. 04021020, doi: 10.1061/(ASCE)ST.1943-541X.00022963

Selected experimental findings
-Influence of loading history

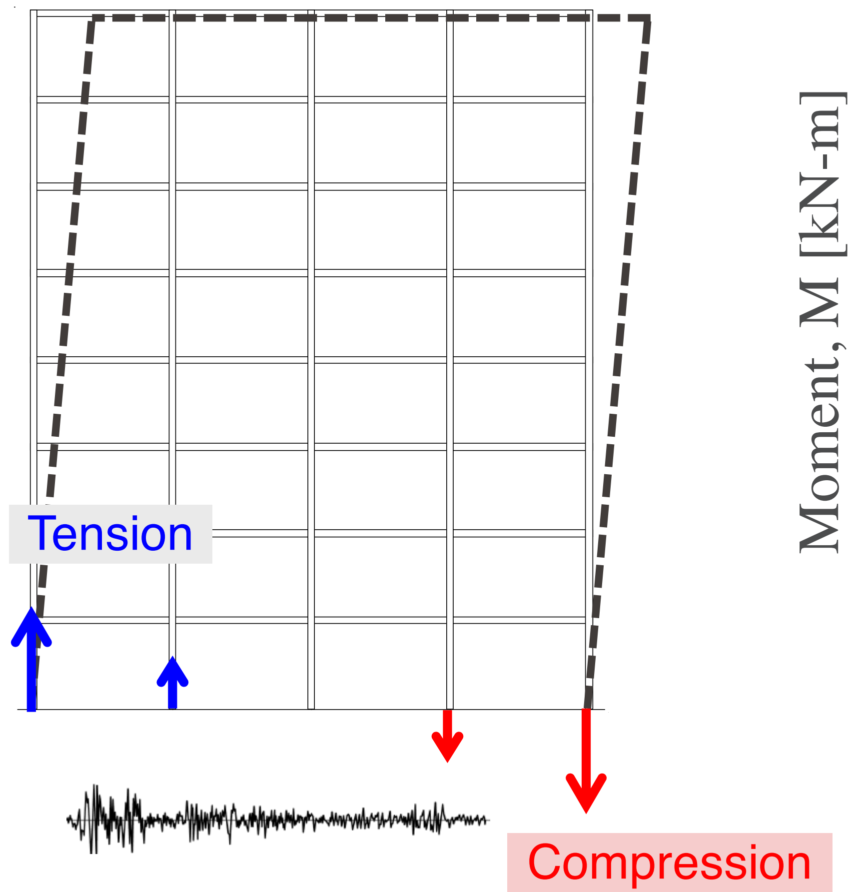


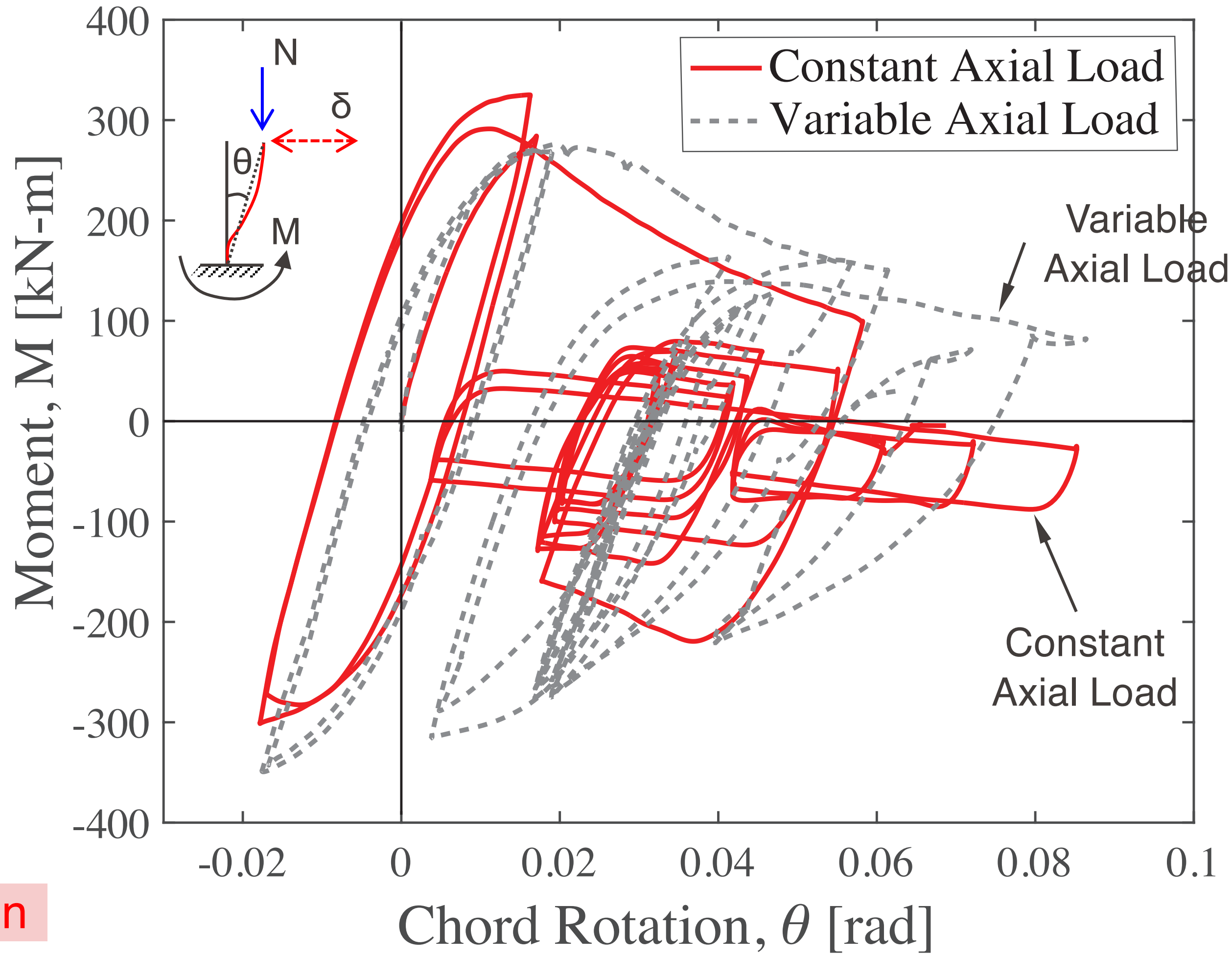


Source: Suzuki and Lignos (2021)

*Suzuki, Y., Lignos, D.G. (2021). "*Experimental Evaluation of Steel Columns under Seismic Hazard-Consistent Collapse Loading Protocols*, ASCE Journal of Structural Engineering, Vol. 147(4), pp. 04021020, doi: 10.1061/(ASCE)ST.1943-541X.00022963

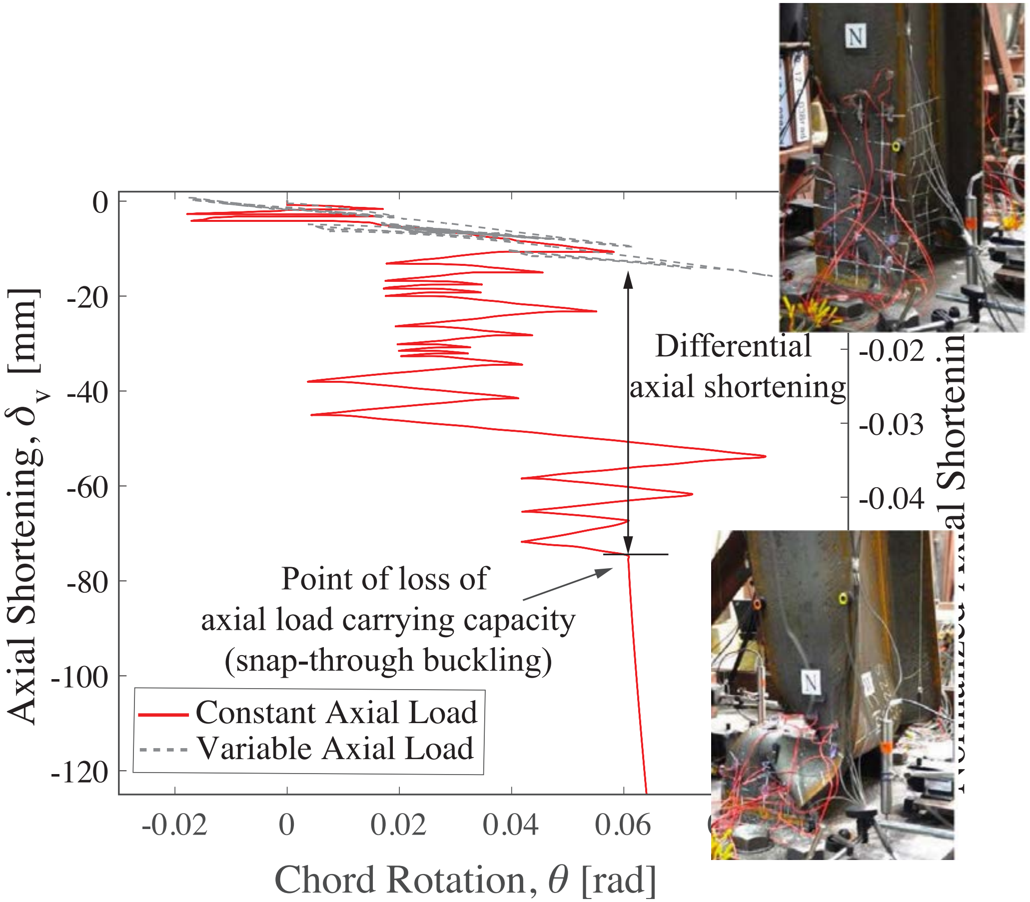
Effect of axial load (Variable versus constant axial load)

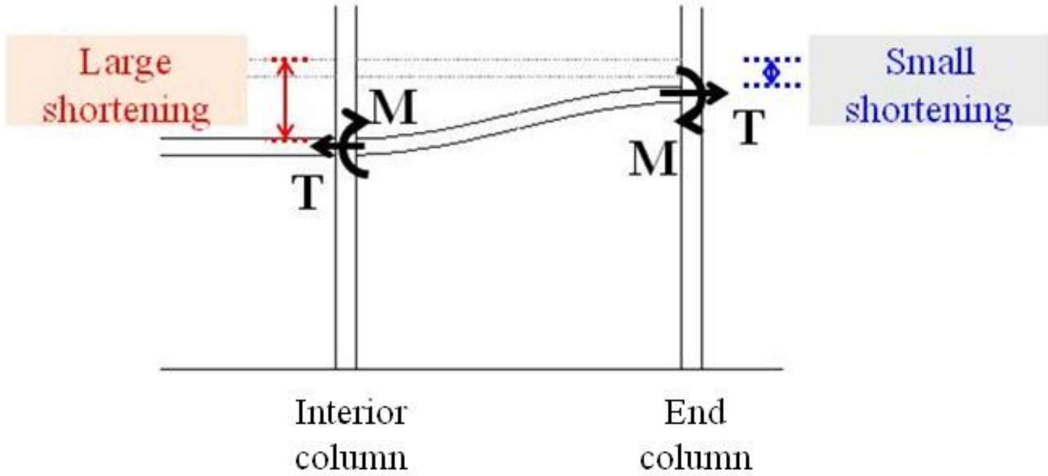




Source: Suzuki and Lignos (2021)

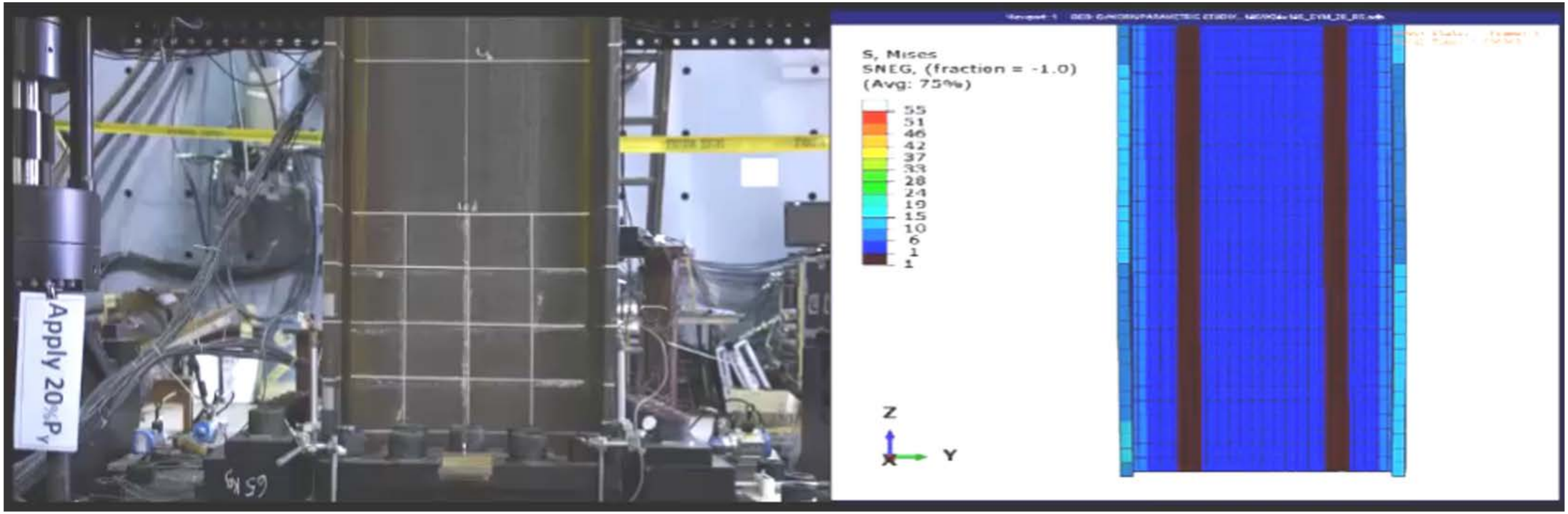
Effect of axial load (Variable versus constant axial load)





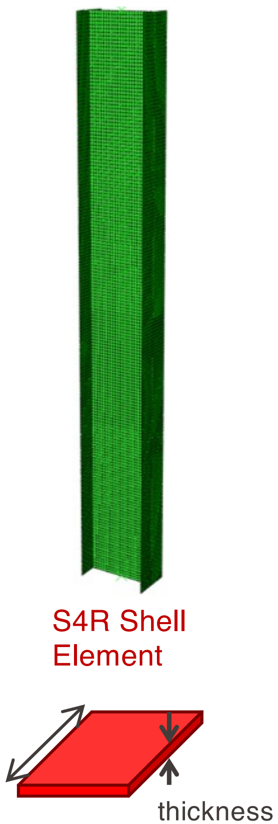
Source: Suzuki and Lignos (2021)

Validated continuum finite element models

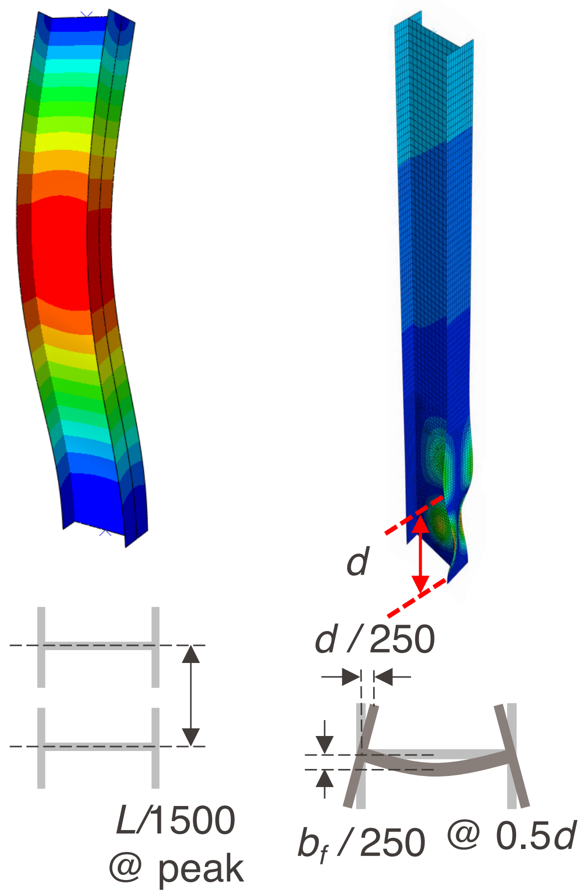


Source: Elkady and Lignos (2018)

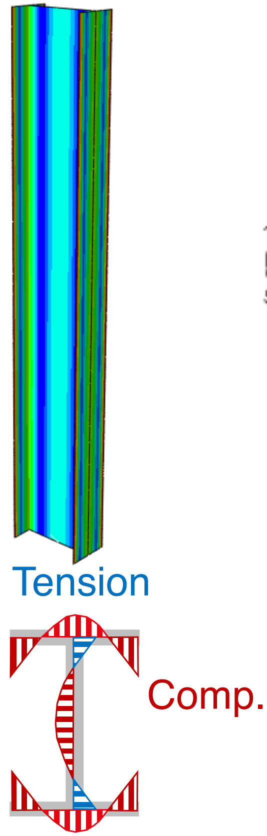
Continuum finite element model specifics



Imperfections

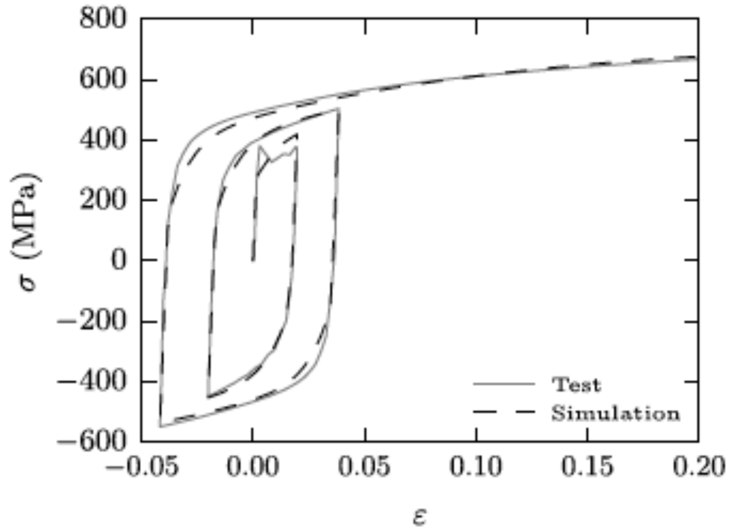


Residual stresses



*Source: Elkady and Lignos (2018)**

Loading history-independent
Material parameters for cyclic plasticity



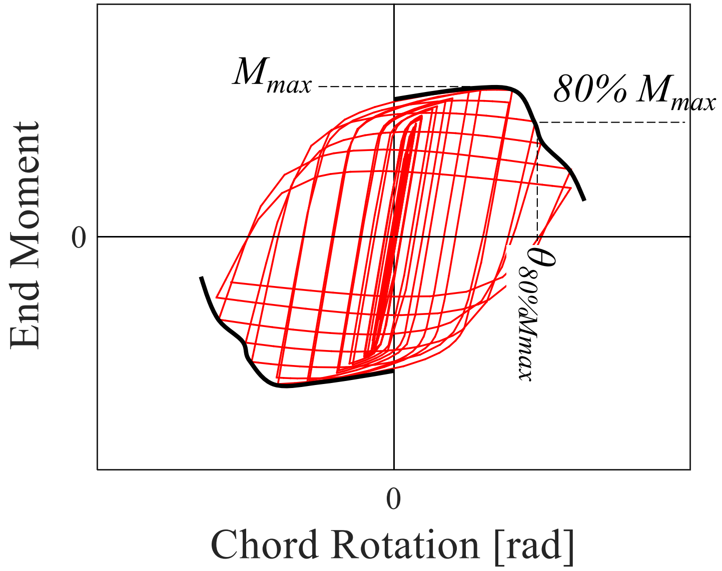
Source: Hartloper et al. (2021)

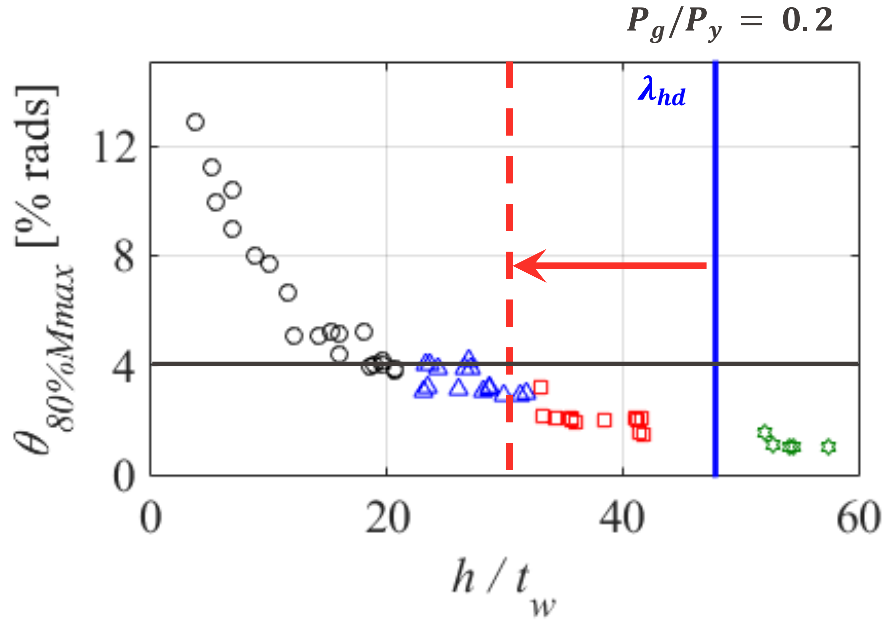
*Elkady, A., Lignos, D.G. (2018). "*Improved Seismic Design and Nonlinear Modeling Recommendations for Wide-Flange Steel Columns*", ASCE Journal of Structural Engineering, doi: 10.1061/(ASCE)ST.1943-541X.0002166

Hartloper, A., de Castro e Sousa, A., and Lignos, D. G. (2021). "Constitutive Modeling of Structural Steels: Nonlinear Isotropic/Kinematic Hardening Material Model and Its Calibration." ASCE Journal of Structural Engineering. Vol. 147(4), pp. 04021031.

Continuum finite element parametric simulations

-Proposed design recommendations





Source: Elkady and Lignos (2018)

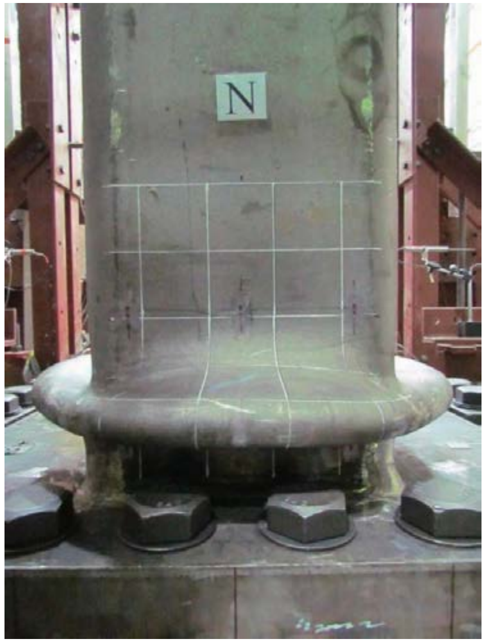
- Proposed reduction to 2/3 of the current web compactness limit as per AISC-341-16 for first story steel columns
- Impose an upper limit of 30% on gravity induced axial load ratio for first story steel columns in type D MRFs (adopted in CSA S16)

Proposed design recommendations for steel columns

-Adopted in new Eurocode 8 EN1998:2023-1-2

Wide flange or HSS

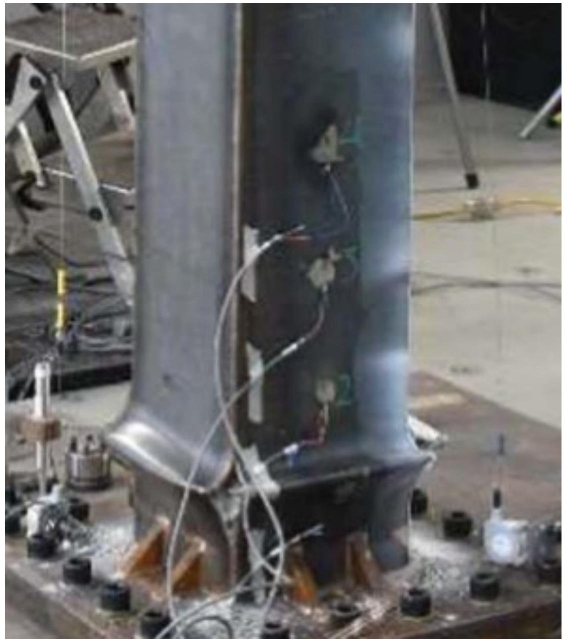
$$\frac{P_G}{P_y} \leq 0.30$$



Suzuki and Lignos (2015)

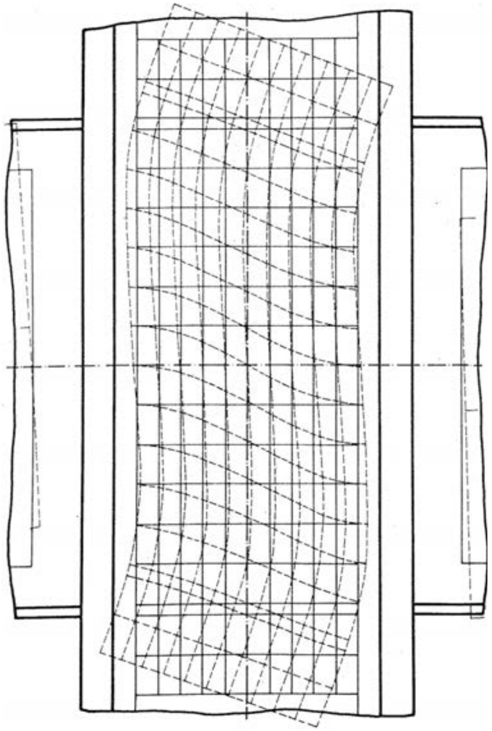
Encased, partially encased or filled composite

$$\frac{P_G}{P_y} \leq 0.75$$

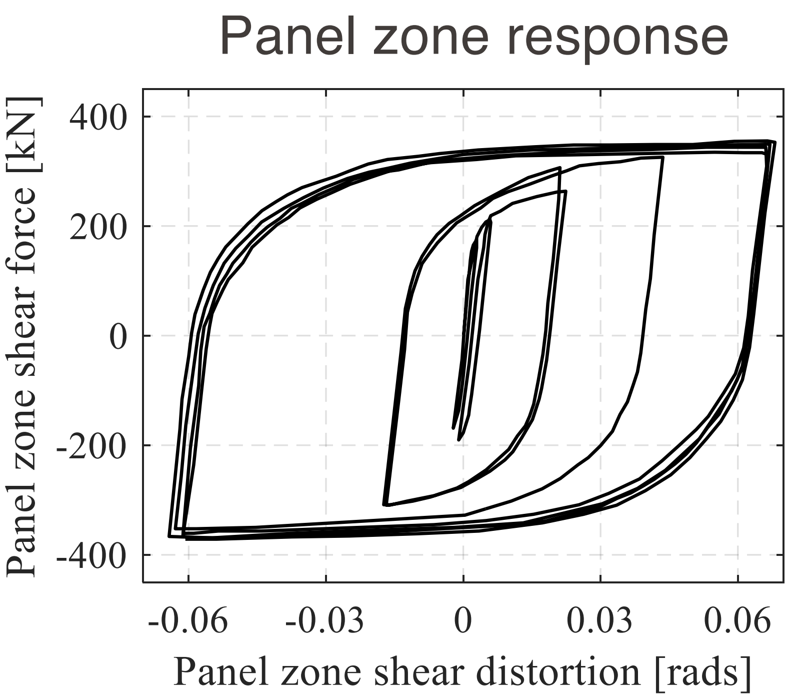


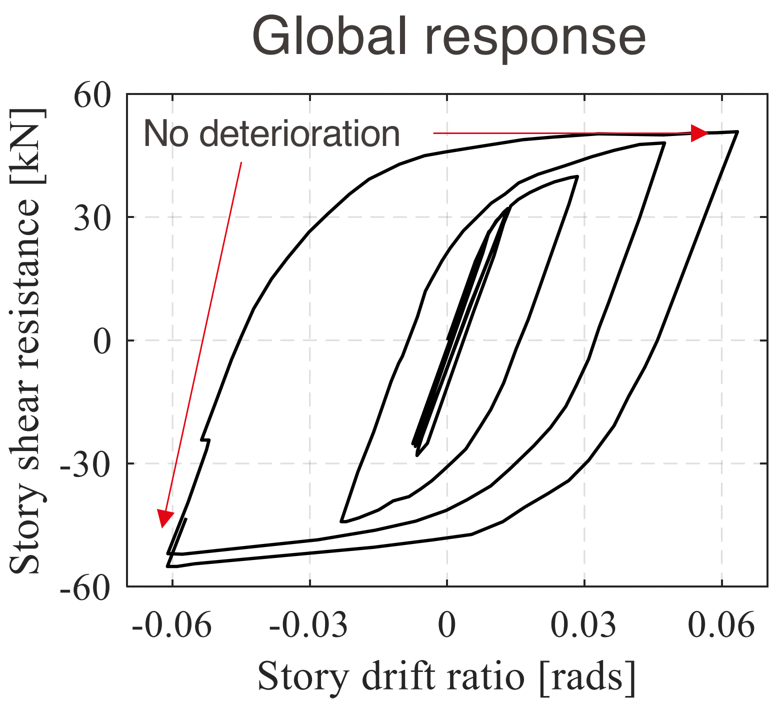
Farahi et al. (2022)

Panel zone joint: Behavior & design models

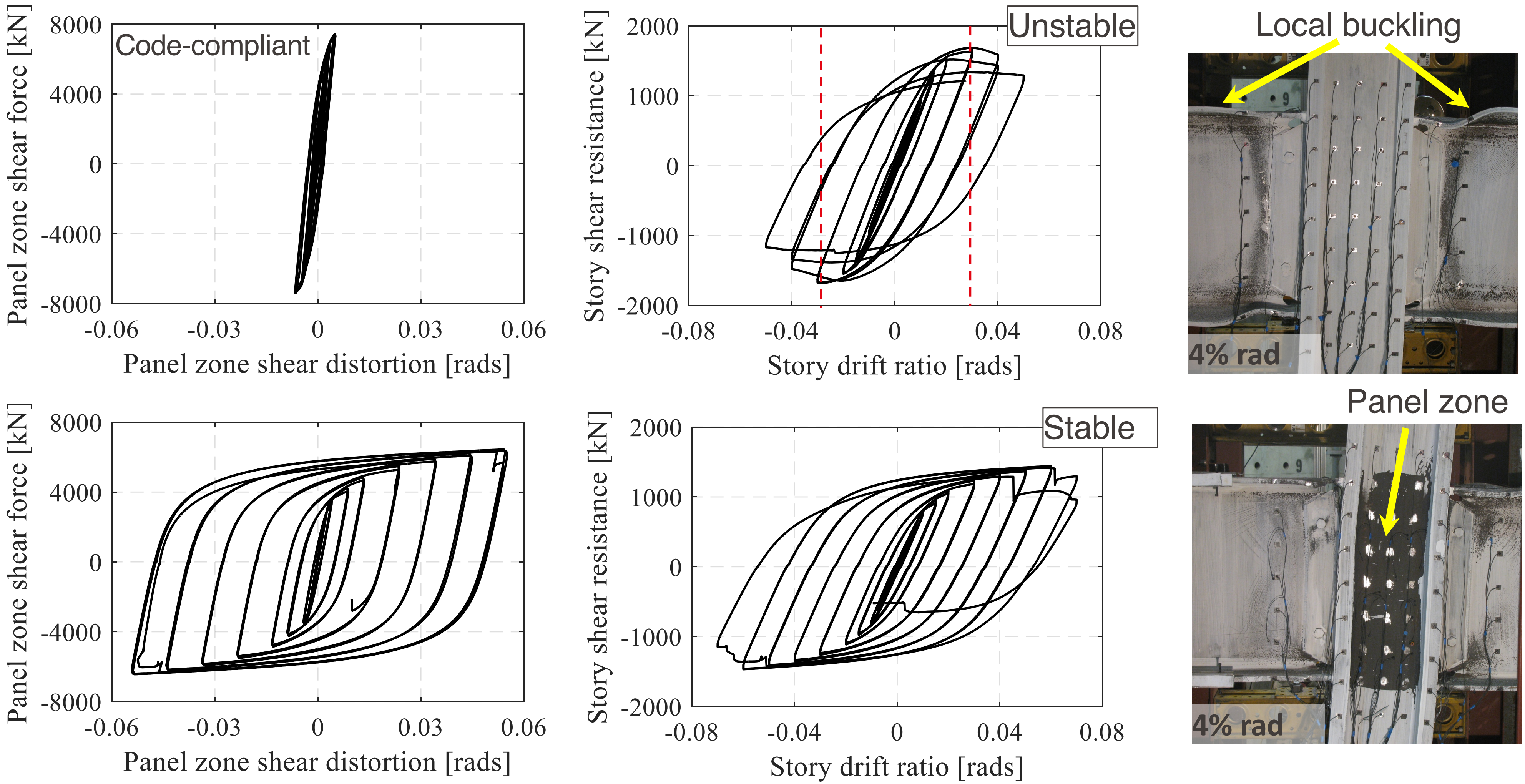


Krawinkler et al. (1971)





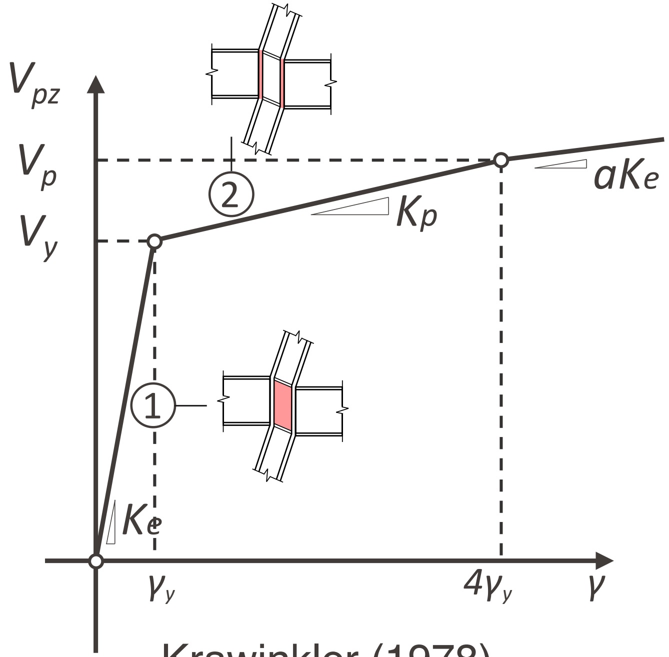
Elastic versus inelastic panel zone design



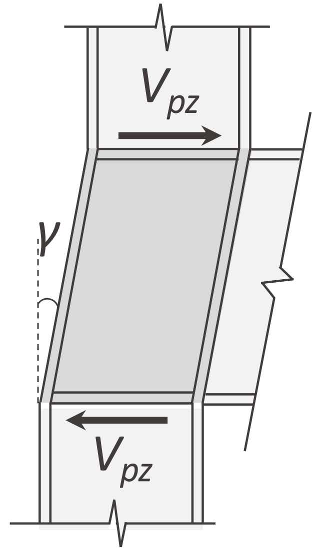
@ Shin and Engelhardt (2013), NSF funded project at University of Minnesota

Missing aspects to leverage shear yielding

-Limitations of current design models



Krawinkler (1978)



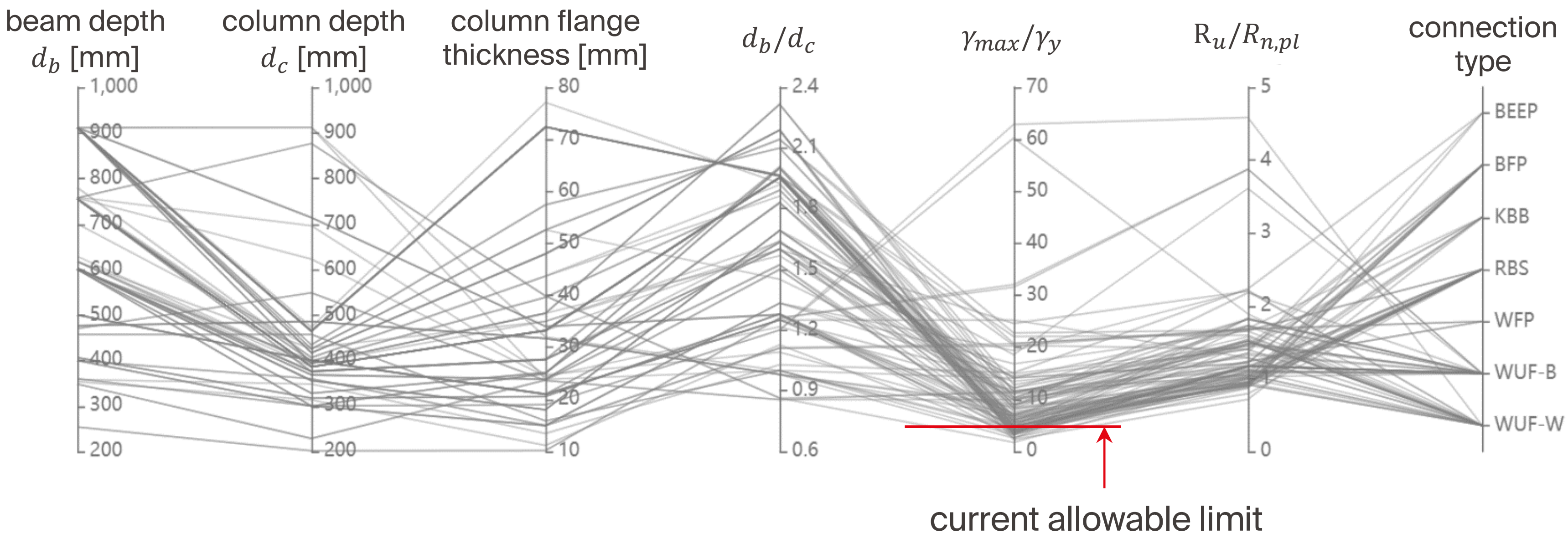
Main assumptions:

- K_e: Only shear deformations are considered
- V_y: Uniform panel zone yielding
- V_p: Valid until $4\gamma_y$ for flange thicknesses less than 40 mm

Review of test data on Post-Northridge connections

γ_y: panel zone shear distortions at yield

γ_{max}: maximum experimental panel zone shear distortions

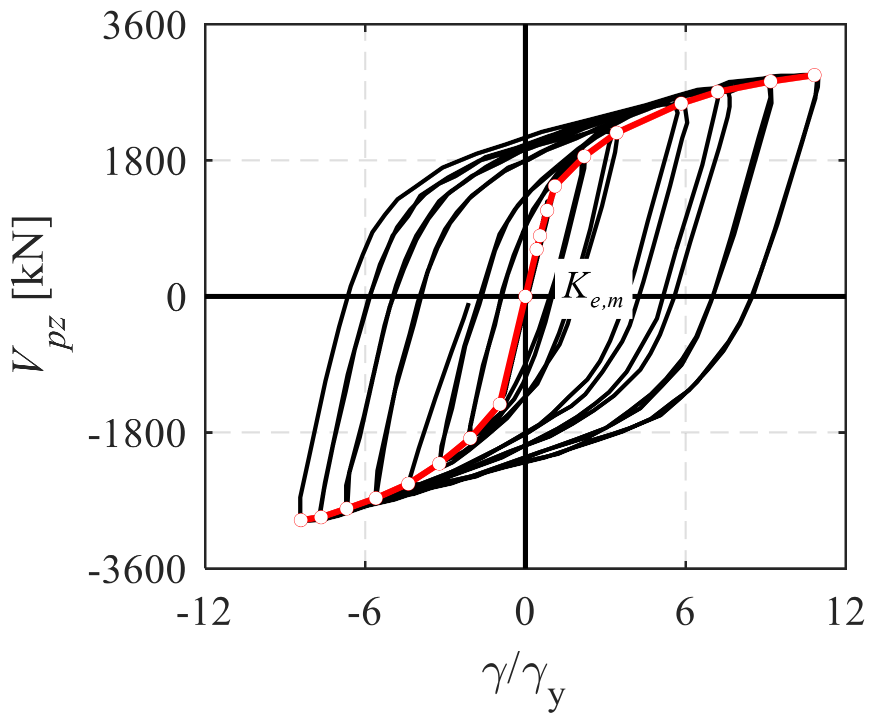


Data availability:
RESSLab Hub

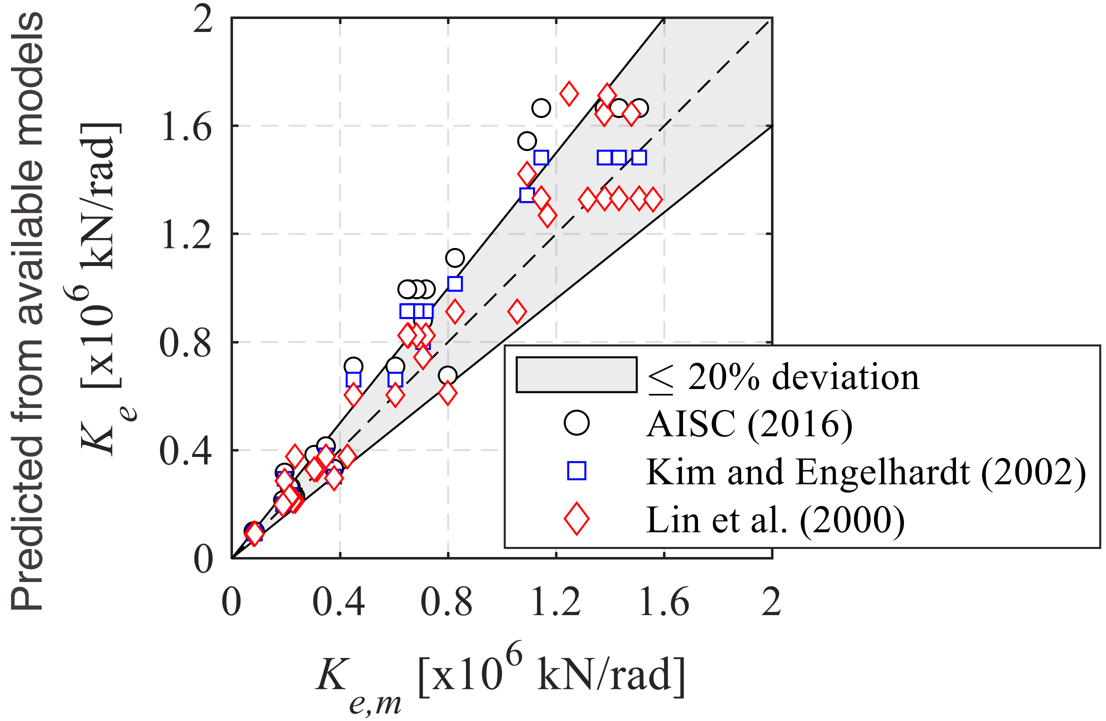
Skiadopoulos, A., and Lignos, D. G. (2021). "Development of inelastic panel zone database." *Journal of Structural Engineering*, American Society of Civil Engineers, 147(4), 04721001

Assessment of available panel zone models

-Panel zone stiffness



(Data source: Kim et al. 2015)

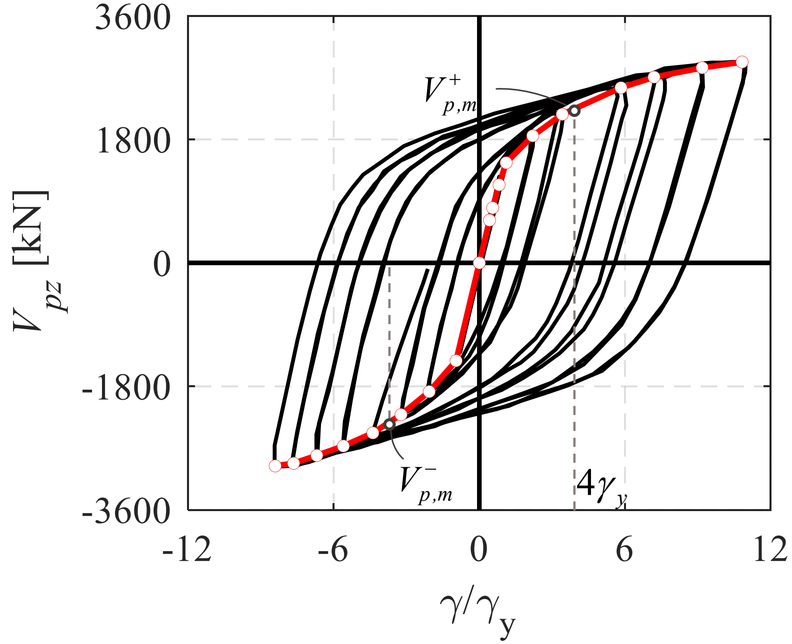


Measured from available tests

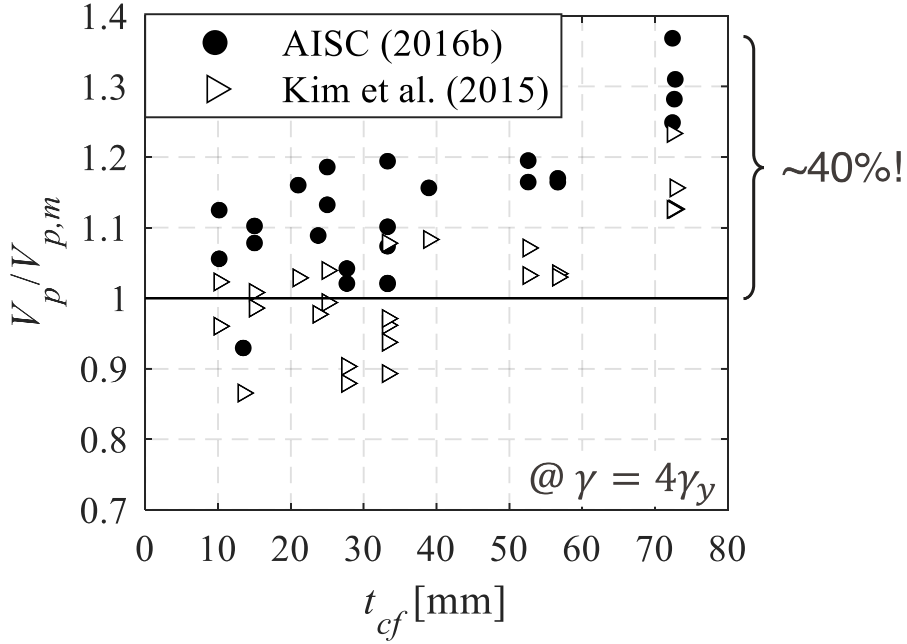
Skiadopoulos, A., Elkady, A., and Lignos, D. G. (2021). "Proposed panel zone model for seismic design of steel moment-resisting frames." *Journal of Structural Engineering*, American Society of Civil Engineers, 147(4), 04021006. DOI: https://doi.org/10.1061/(ASCE)ST.1943-541X.0002935.

Assessment of available panel zone models

-Panel zone shear resistance



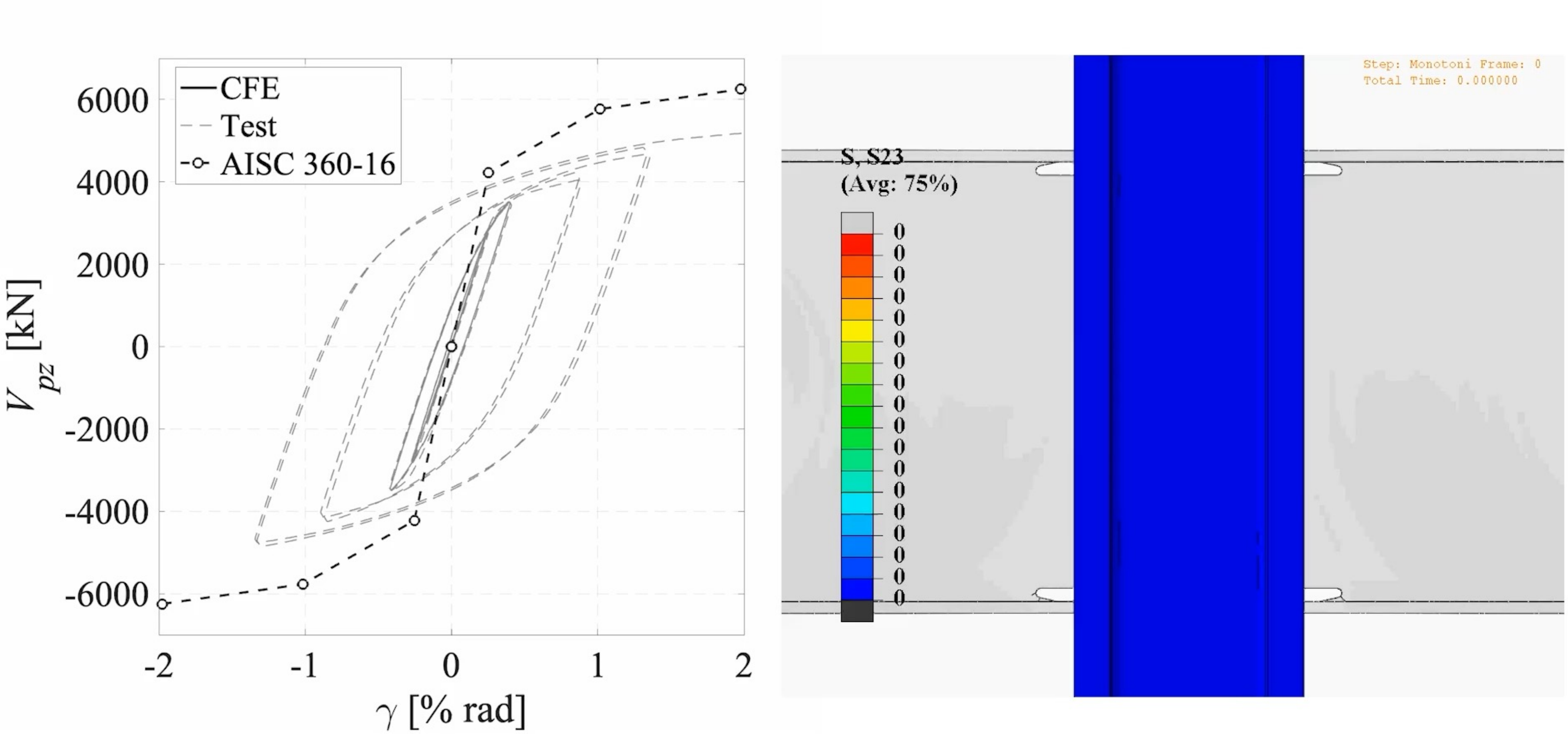
(Data source: Kim et al. 2015)



$V_{p,m}$: Measured from available tests

Skiadopoulos, A., Elkady, A., and Lignos, D. G. (2021). "Proposed panel zone model for seismic design of steel moment-resisting frames." *Journal of Structural Engineering*, American Society of Civil Engineers, 147(4), 04021006. DOI: https://doi.org/10.1061/(ASCE)ST.1943-541X.0002935.

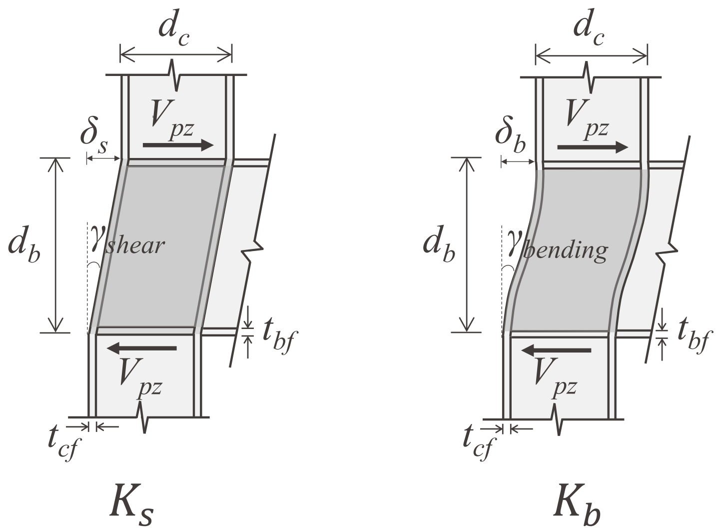
A fresh look into the problem



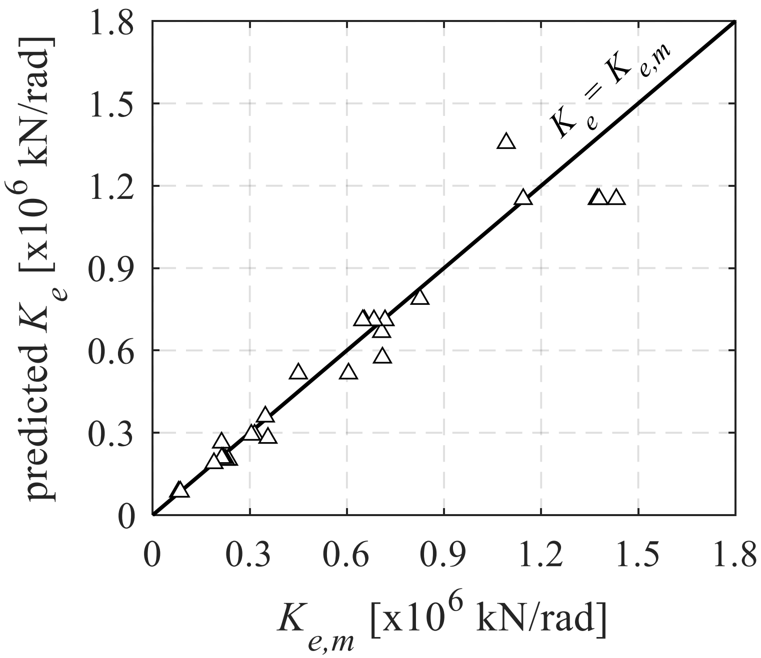
Skiadopoulos and Lignos (2021)
(Data source: Shin and Engelhardt 2013)

Proposed panel zone model

-Proposed expression for panel zone stiffness



$$K_e = \frac{V_{pz}}{\gamma} = \frac{K_s \cdot K_b}{K_s + K_b}$$

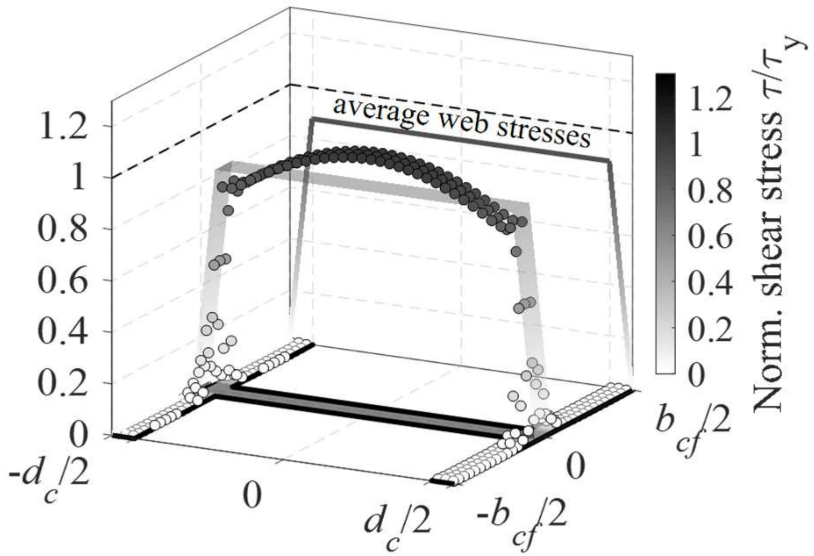


Skiadopoulos, A., Elkady, A., and Lignos, D. G. (2021). "Proposed panel zone model for seismic design of steel moment-resisting frames." *Journal of Structural Engineering*, American Society of Civil Engineers, 147(4), 04021006. DOI: https://doi.org/10.1061/(ASCE)ST.1943-541X.0002935.

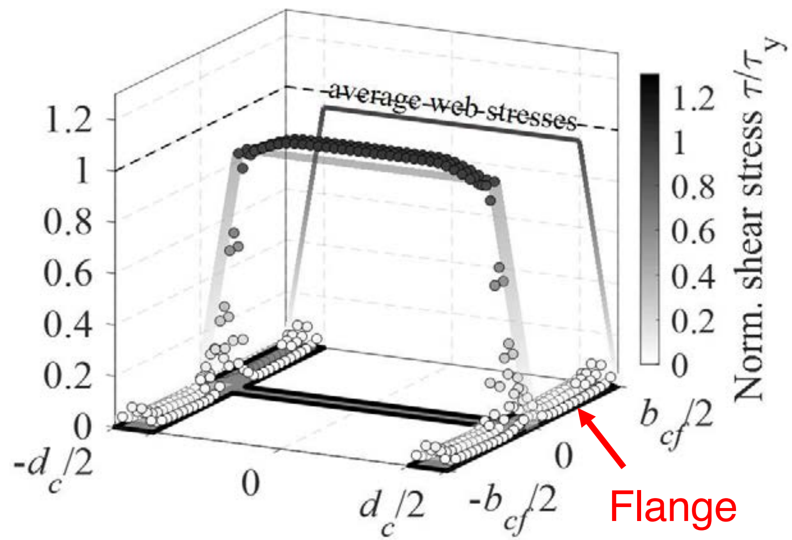
Proposed panel zone model

-Panel zone strength

Beam: W30x108, Column: W24x131



Beam: W36x150, Column: W14x398



$$V_{pz} = \frac{f_y}{\sqrt{3}} \cdot \left[a_{w,eff} \cdot \left(d_c - t_{cf}\right) \cdot t_{cw} + a_{f,eff} \cdot \left(b_{cf} - t_{cw}\right) \cdot 2t_{cf}\right]$$

web contribution — flange contribution

Skiadopoulos, A., Elkady, A., and Lignos, D. G. (2021). "Proposed panel zone model for seismic design of steel moment-resisting frames." *Journal of Structural Engineering*, American Society of Civil Engineers, 147(4), 04021006. DOI: https://doi.org/10.1061/(ASCE)ST.1943-541X.0002935.

Proposed model (adopted in CSA-S19 & EC8 Part 1)

-Proposed expression for panel zone shear strength

$$V_r = 0.61 a_c \phi f_{yc} \left\{ \underbrace{(d_c - t_{cf}) t_{pz}}_{\text{web contribution}} + \underbrace{\left[1.69 \left(\frac{K_f}{K_e} \right) + 0.027 \right] (b_{cf} - t_{pz}) t_{cf}}_{\text{flange contribution}} \right\}$$

For $\dfrac{P}{P_y} \leq 0.4 \qquad a_c = 1.0$

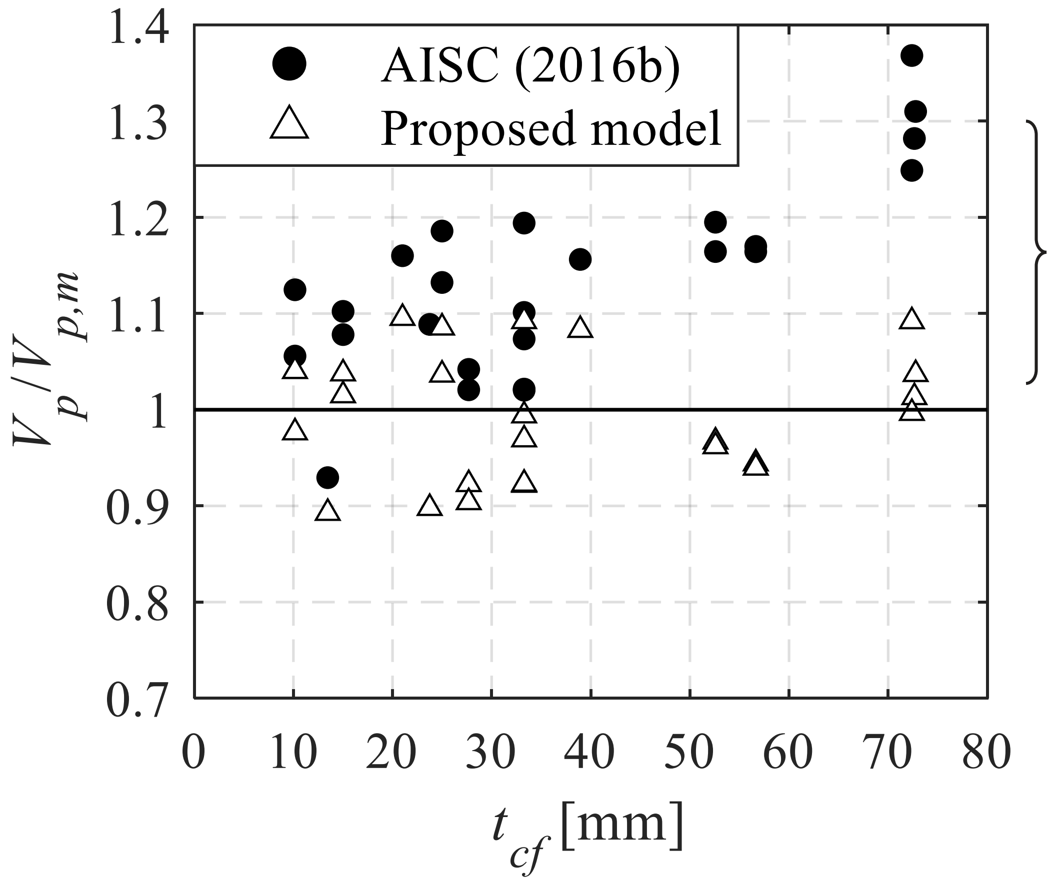
For $\dfrac{P}{P_y} > 0.4 \qquad a_c = \sqrt{1 - \left(\dfrac{P}{P_y} \right)^2}$

K_f / K_e = column flange-to-panel zone stiffness ratio and shall be computed as follows:

$$K_f = \frac{2 E b_{cf} t_{cf}^3}{d_b^2 + 2(1+v) t_{cf}^2}, \qquad K_e = \frac{12 E t_{pz} (d_c - t_{cf}) I_c}{t_{pz} (d_c - t_{cf}) d_b^2 + 24 (1+v) I_c}$$

Skiadopoulos, A., Elkady, A., and Lignos, D. G. (2021). "Proposed panel zone model for seismic design of steel moment-resisting frames." *Journal of Structural Engineering*, American Society of Civil Engineers, 147(4), 04021006. DOI: https://doi.org/10.1061/(ASCE)ST.1943-541X.0002935.

Comparisons with current model and available data



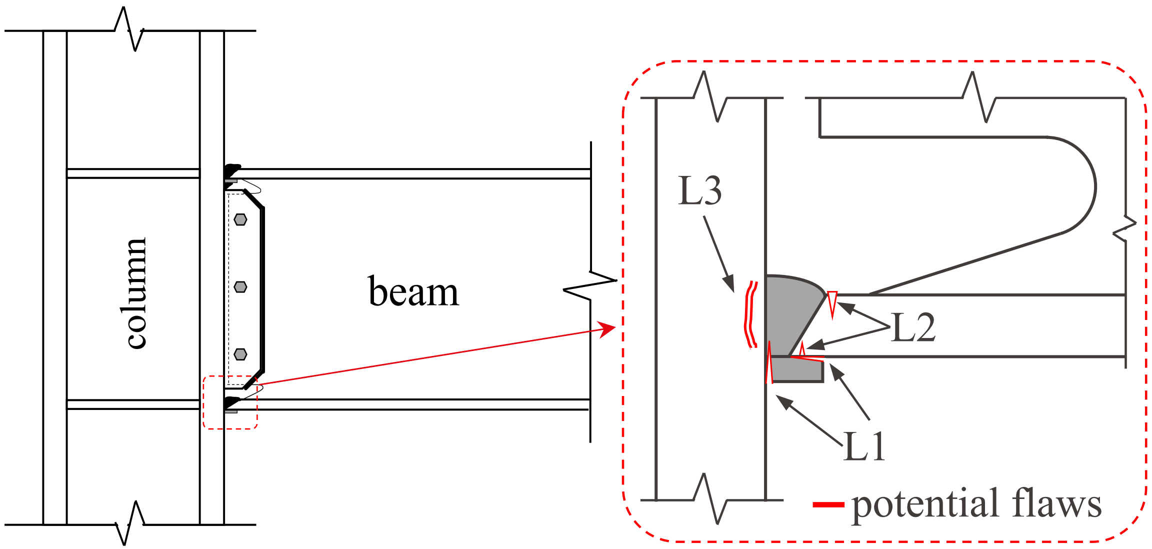
Reduction or error by 40%

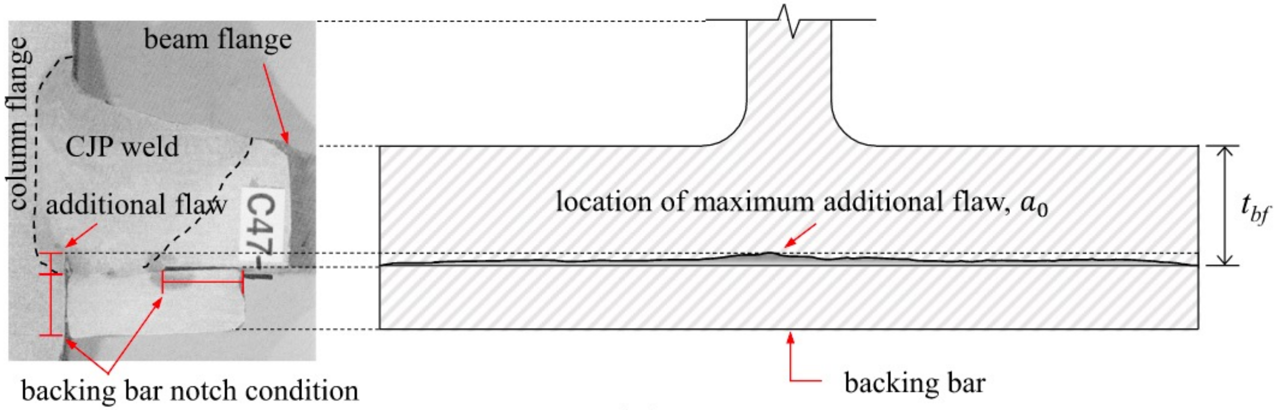
Skiadopoulos, A., Elkady, A., and Lignos, D. G. (2021). "Proposed panel zone model for seismic design of steel moment-resisting frames." *Journal of Structural Engineering*, American Society of Civil Engineers, 147(4), 04021006. DOI: https://doi.org/10.1061/(ASCE)ST.1943-541X.0002935.

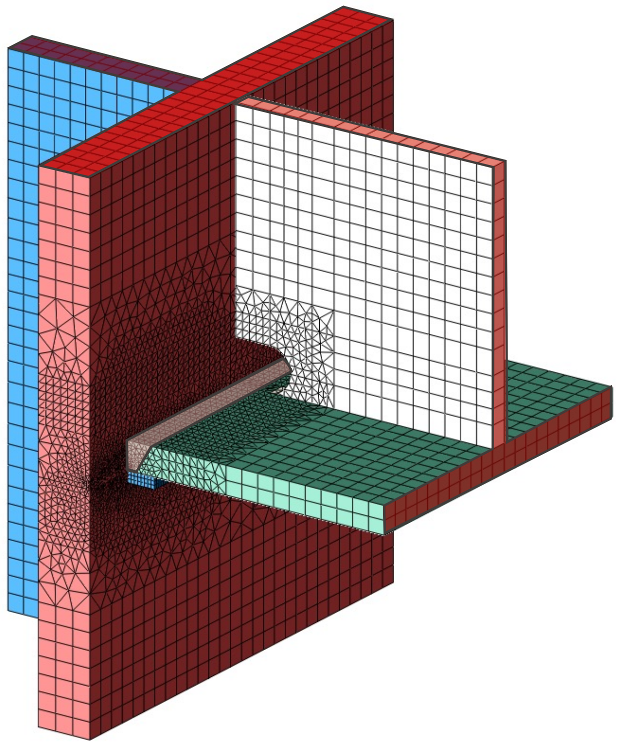
(2022 Raymond Reese Award, American Society of Civil Engineers)
Adopted in second revision of Eurocode 8 Part 3 and CSA S16

Development of simplified weld details

-Welded connections with highly inelastic panel zones



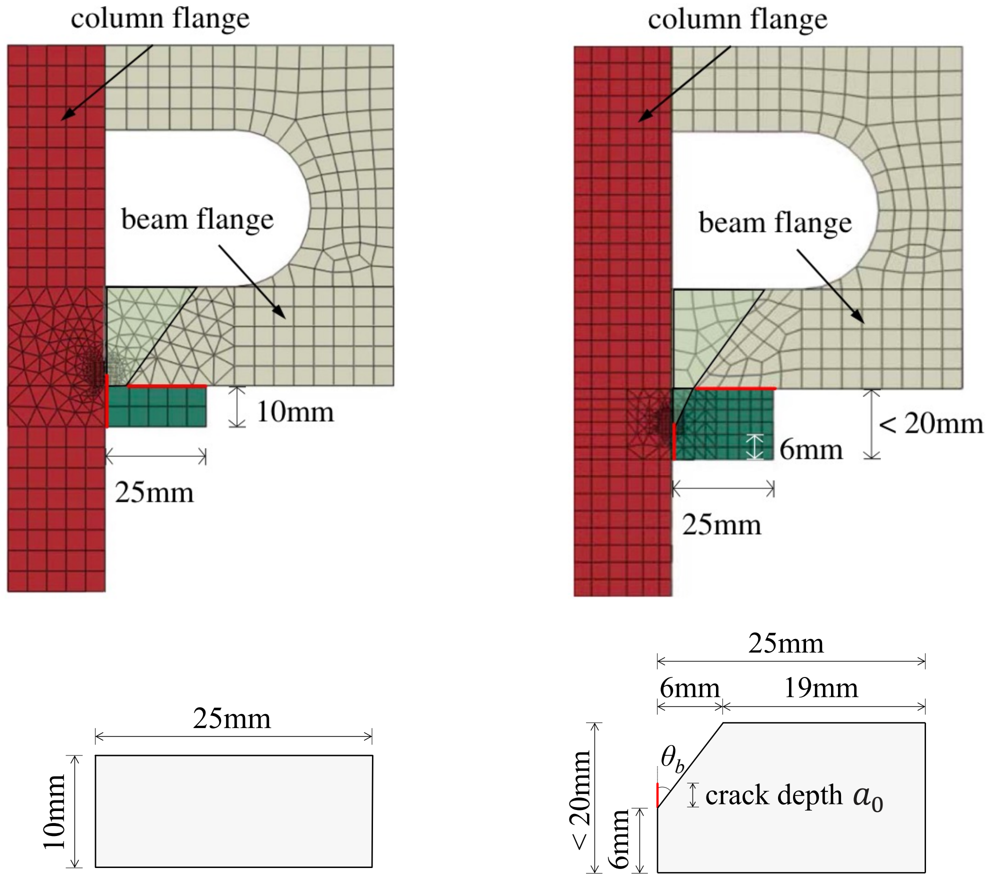




Skiadopoulos and Lignos (2022)

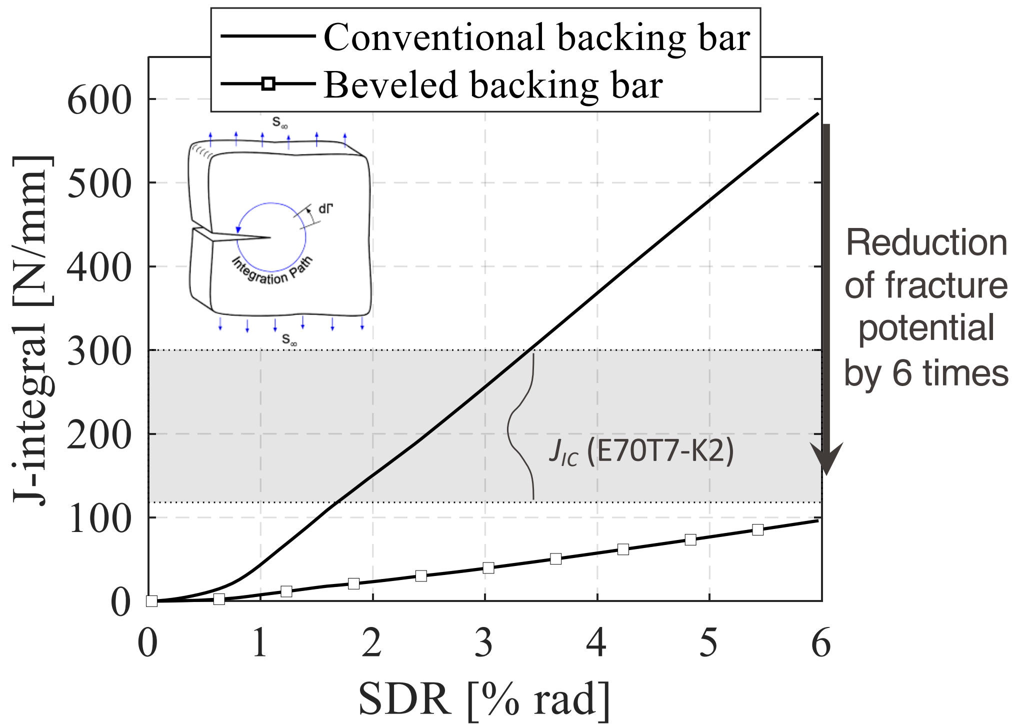
Development of simplified weld details

-Fracture mechanics-based simulations



conventional backing bar

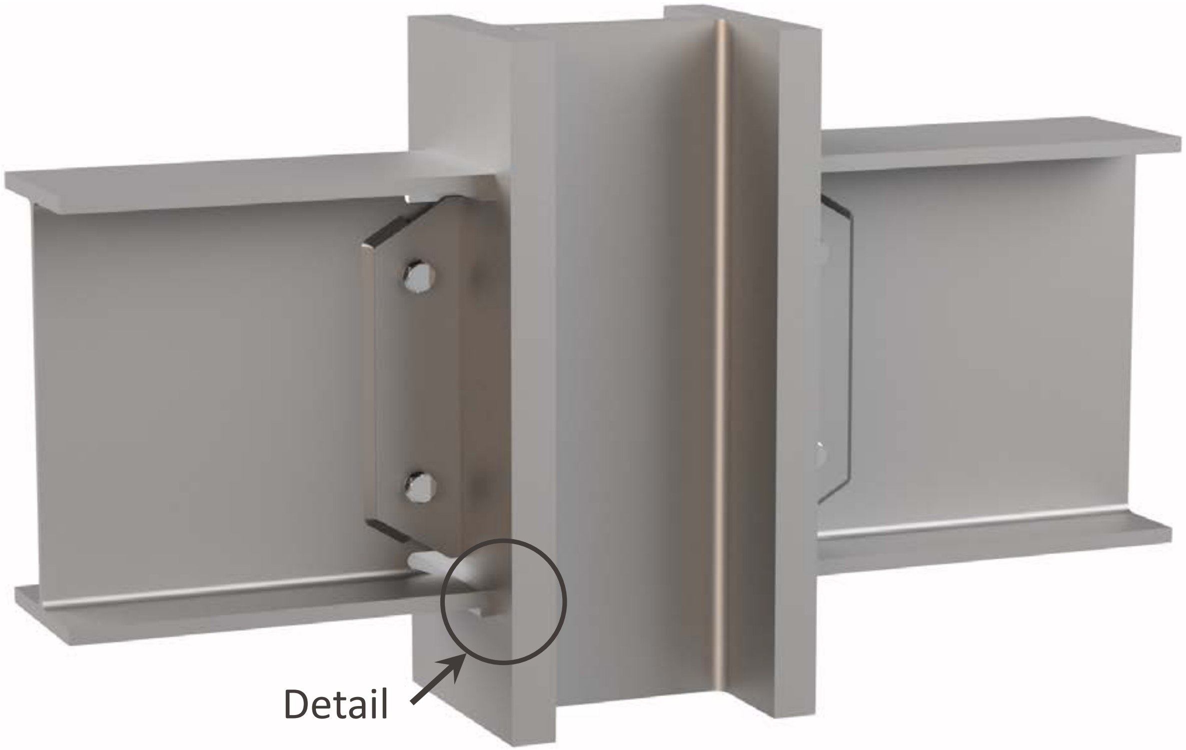
beveled backing bar



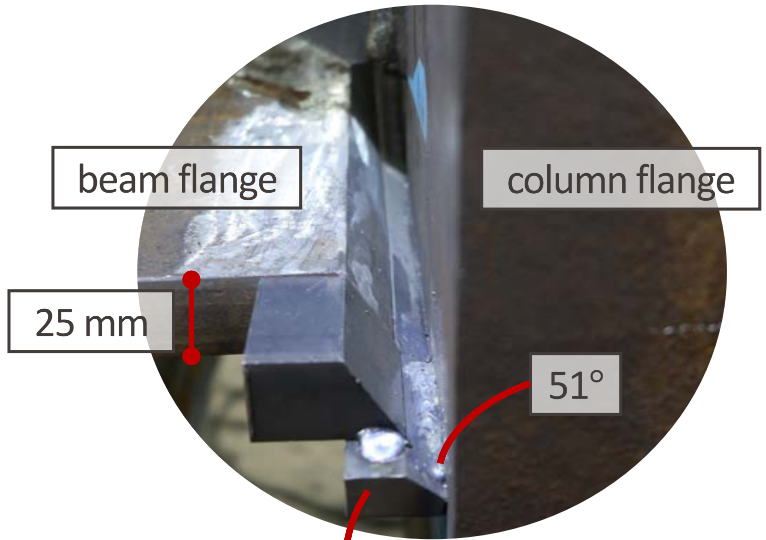
Skiadopoulos, A., and Lignos, D. G. (2022). "Proposed backing bar detail in welded beam-to-column connections for seismic applications." *Journal of Structural Engineering*, American Society of Civil Engineers, 148(8), 04022102. DOI: https://doi.org/10.1061/(ASCE)ST.1943-541X.0003374

Full-scale cyclic tests

-Welded connections with highly dissipative panel zones & simplified weld details



Detail

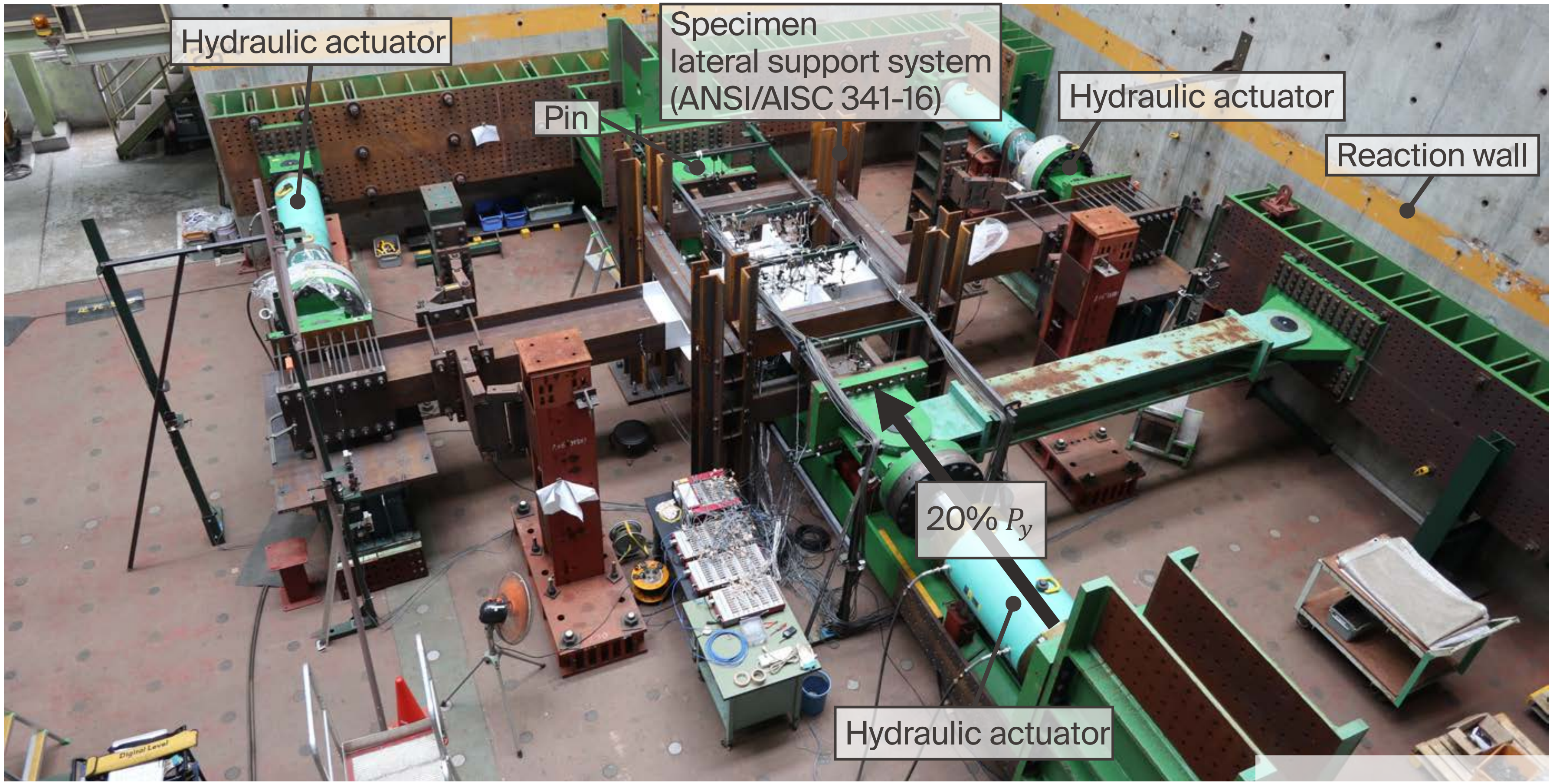


beveled backing bar
(Skiadopoulos and Lignos 2022)

- Stable hysteretic response through panel zone yielding - $\gamma_{max} = 15\gamma_y$ at 4% rad
- Prevent beam local buckling prior to lateral drift demands of 5% rad
- Beveled backing bars left intentionally in place

Full-scale cyclic tests

-Test setup

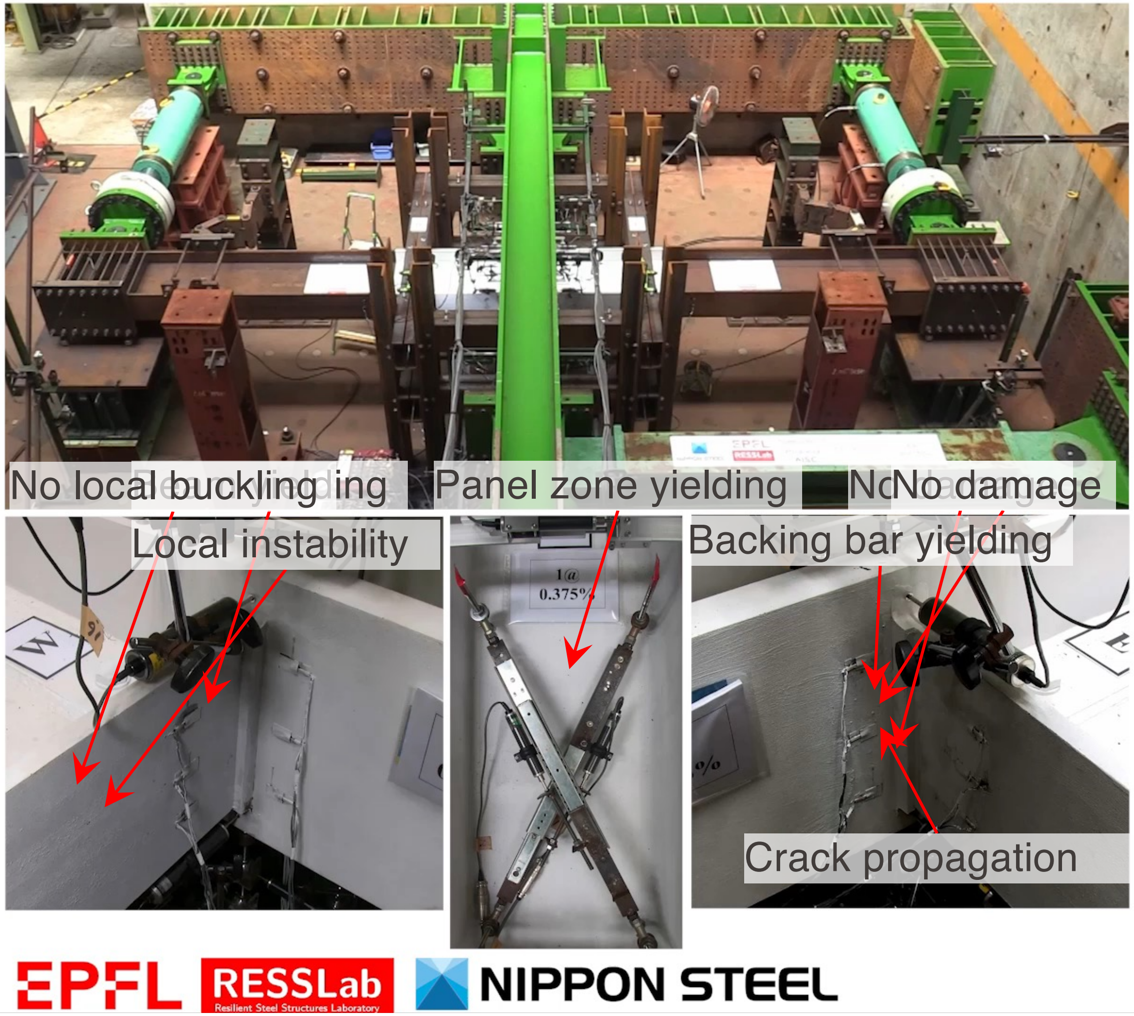


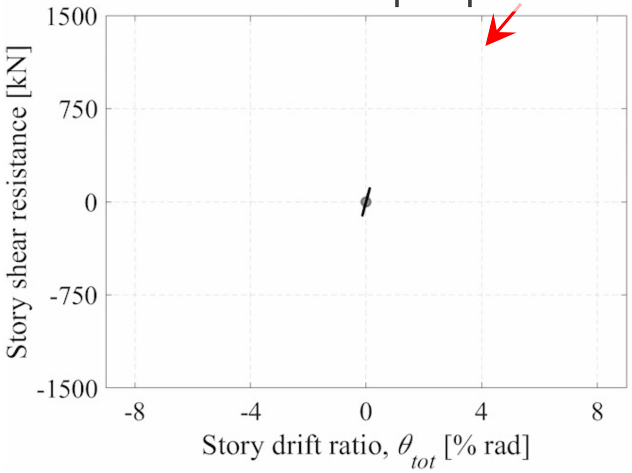
Skiadopoulos et al. (2023)

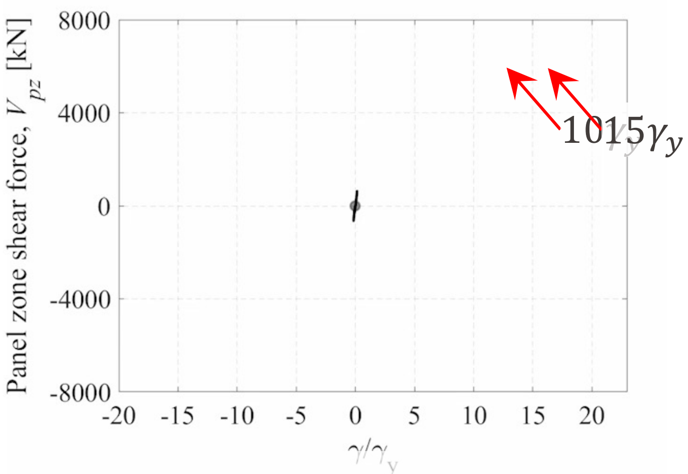
Skiadopoulos, A., Lignos, D. G., Arita., M., Hiroshima, S. (2023). "Full-scale experiments of cyclically loaded welded moment connections with highly dissipative panel zones and simplified weld details." *Journal of Structural Engineering*, American Society of Civil Engineers, 149(12), 04023167.

Full-scale cyclic tests

-Indicative test results

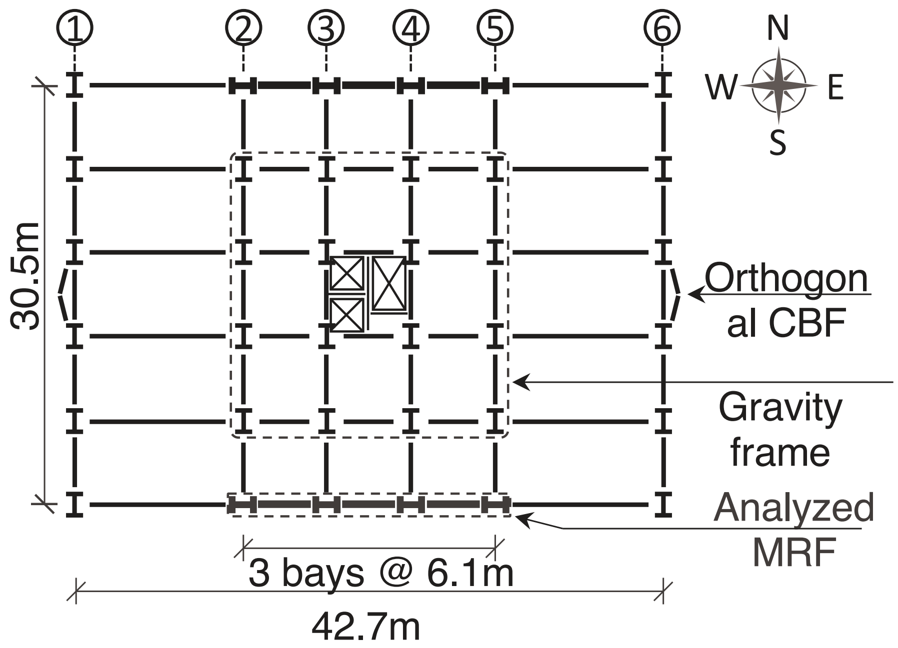


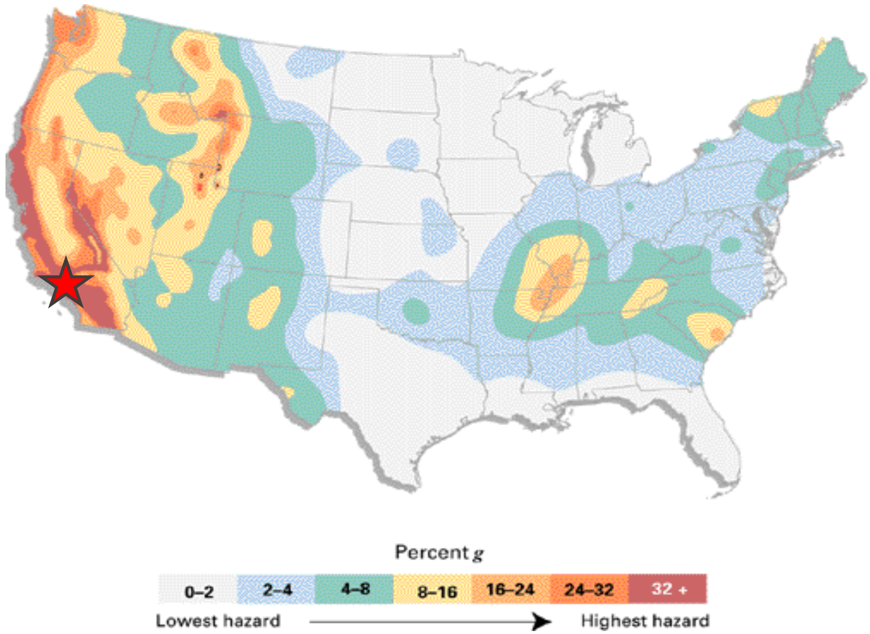




Skiadopoulos et al. (2023)

Archetype steel moment frames





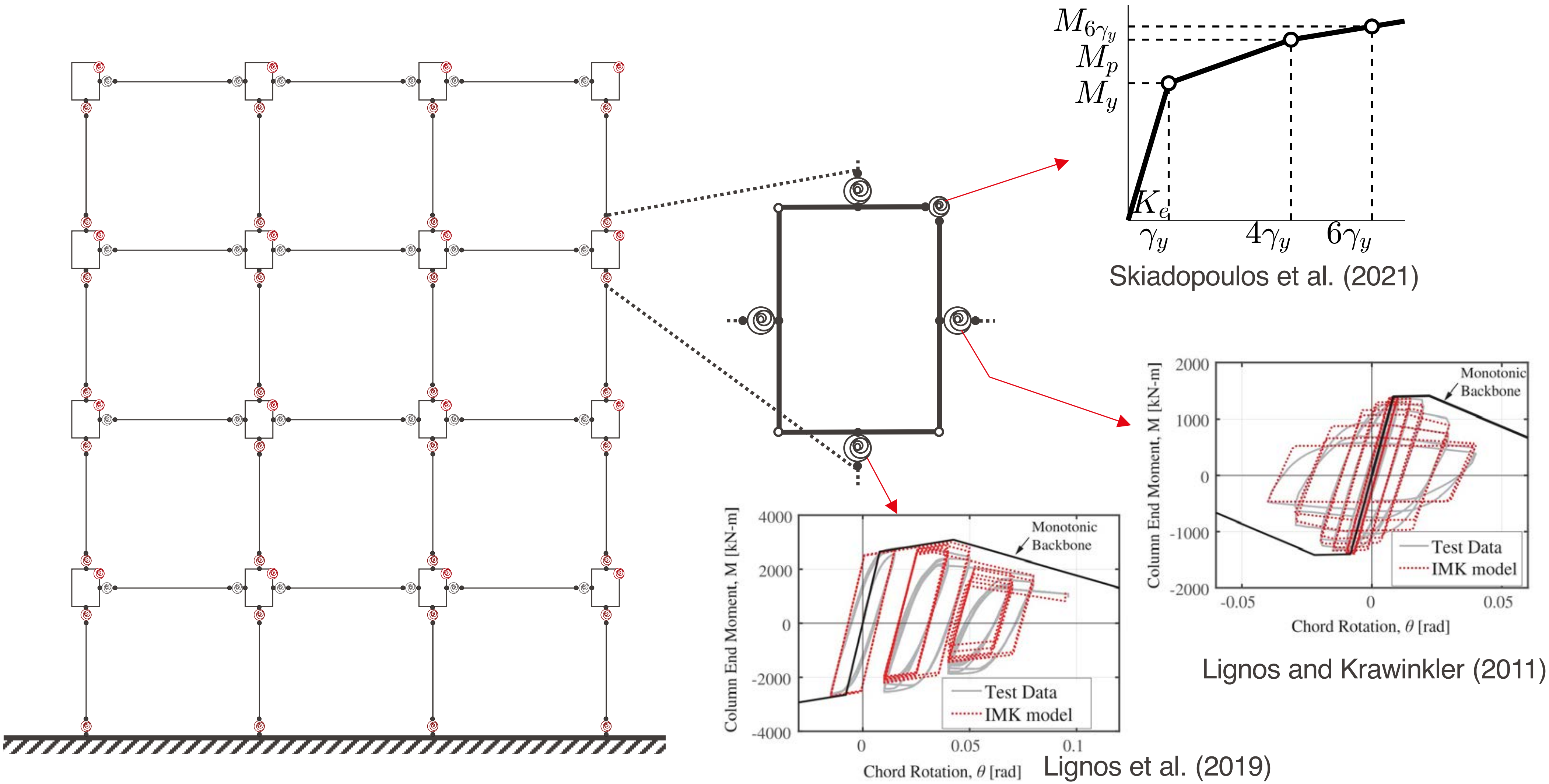
@USGS

- 32 steel buildings designed and analyzed
- Design location: Urban California
- 4-, 8-, 12-, and 20-story buildings
- Three-, and five-bays considered
- Variable panel zone targeted distortions: $\gamma_d = [1, 4, 10, 15]\gamma_y$

Elastic design

32 design summaries available at:

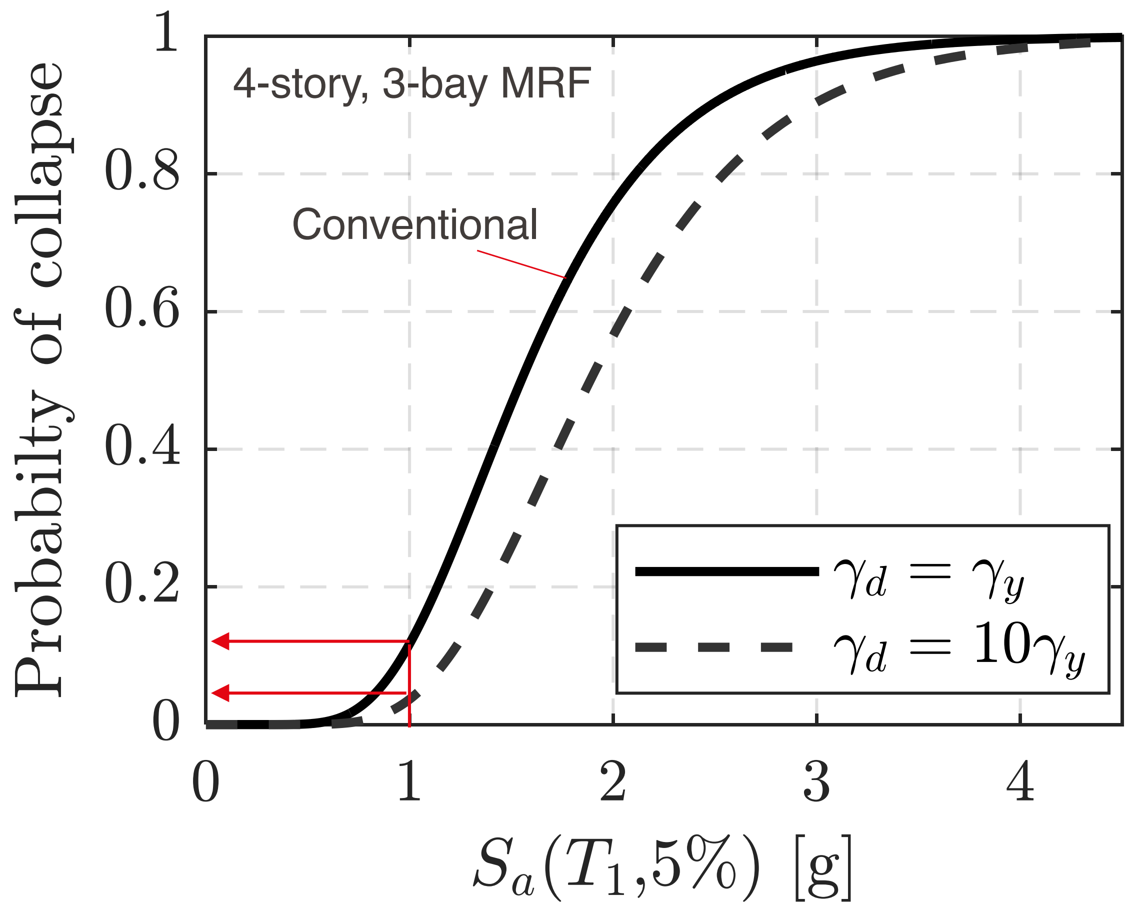
Modeling approach for steel moment frames



Skiadopoulos, A., and Lignos, D. G. (2022). “Seismic demands of steel moment resisting frames with inelastic beam-to-column web panel zones.” *Earthquake Engineering & Structural Dynamics*, Wiley. DOI: https://doi.org/10.1002/eqe.3629

Nonlinear dynamic analyses results

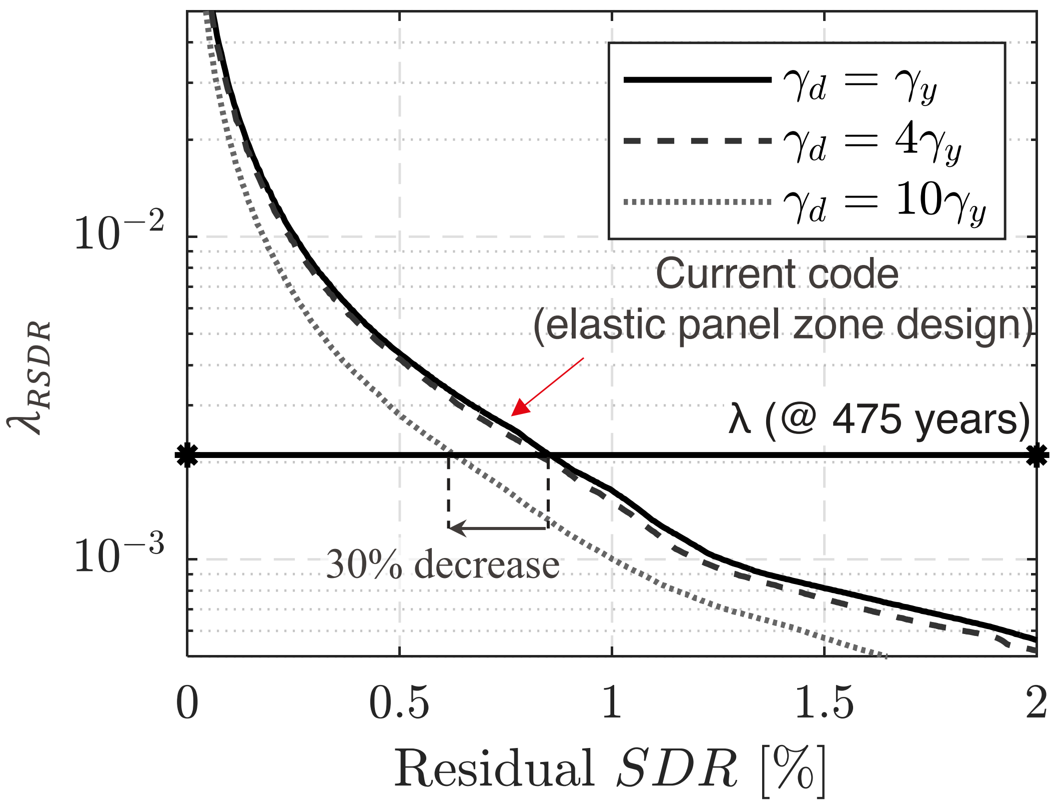
-Collapse risk evaluation: set of 44 far-field ground motions of FEMA P695



Skiadopoulos, A., and Lignos, D. G. (2022). “Seismic demands of steel moment resisting frames with inelastic beam-to-column web panel zones.” *Earthquake Engineering & Structural Dynamics*, Wiley. DOI: https://doi.org/10.1002/eqe.3629

Influence of panel zone design on residual story drift

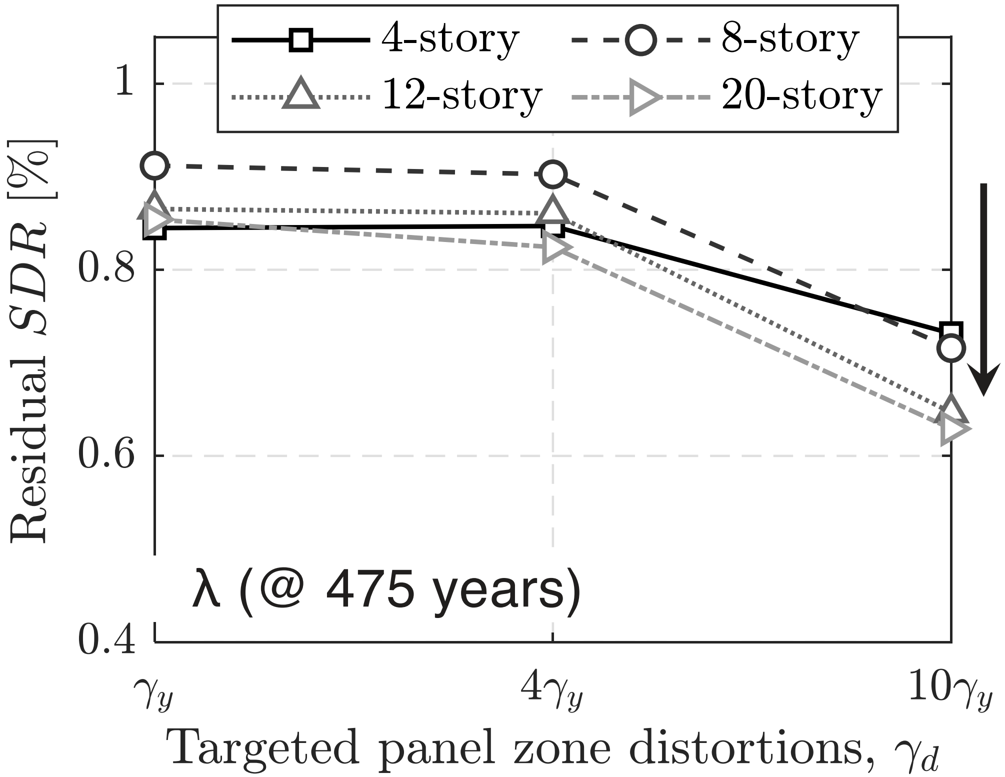
-Residual story drift hazard curves

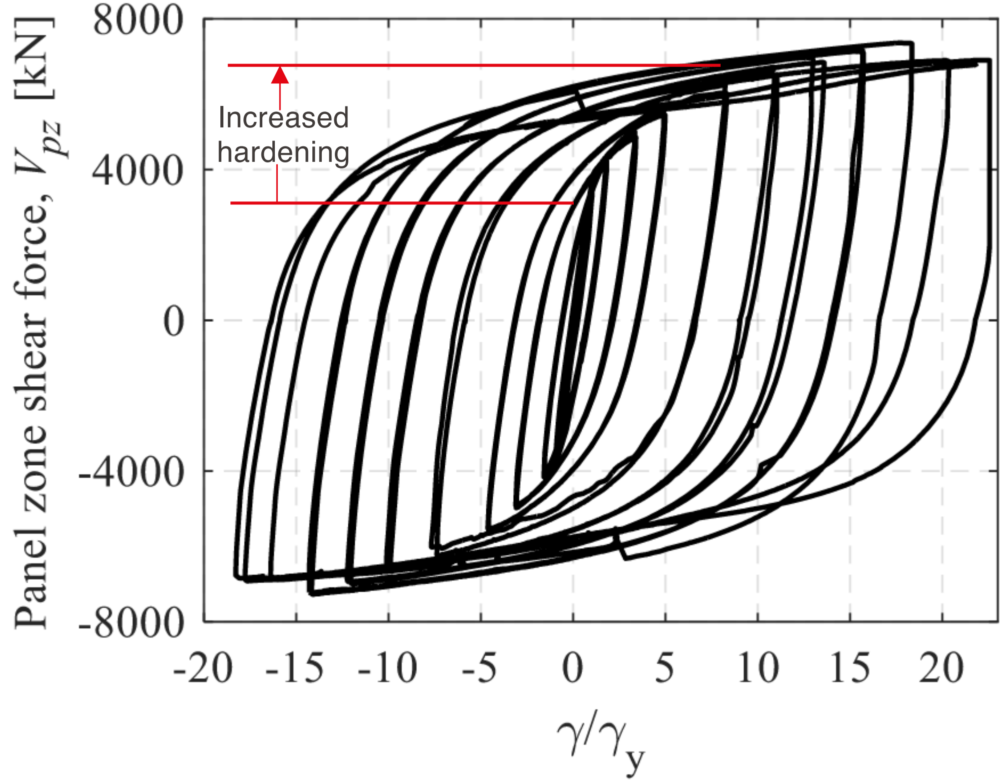


Skiadopoulos, A., and Lignos, D. G. (2022). "Seismic demands of steel moment resisting frames with inelastic beam-to-column web panel zones." *Earthquake Engineering & Structural Dynamics*, Wiley. DOI: https://doi.org/10.1002/eqe.3629

Influence of panel zone design on residual story drift

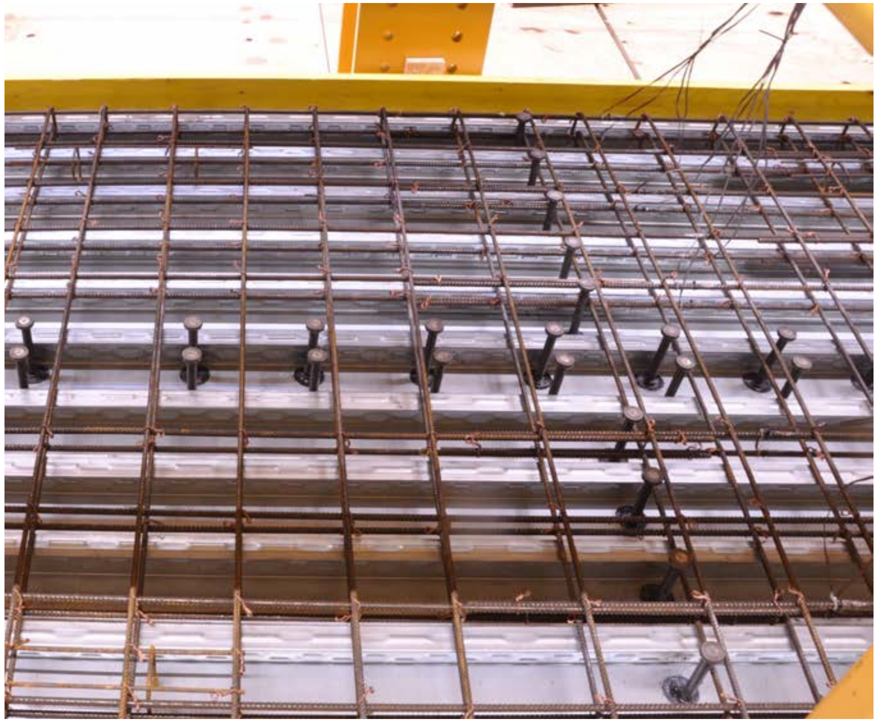
-Design basis earthquake (10% / 50 years)





Skiadopoulos, A., and Lignos, D. G. (2022). "Seismic demands of steel moment resisting frames with inelastic beam-to-column web panel zones." *Earthquake Engineering & Structural Dynamics*, Wiley. DOI: https://doi.org/10.1002/eqe.3629

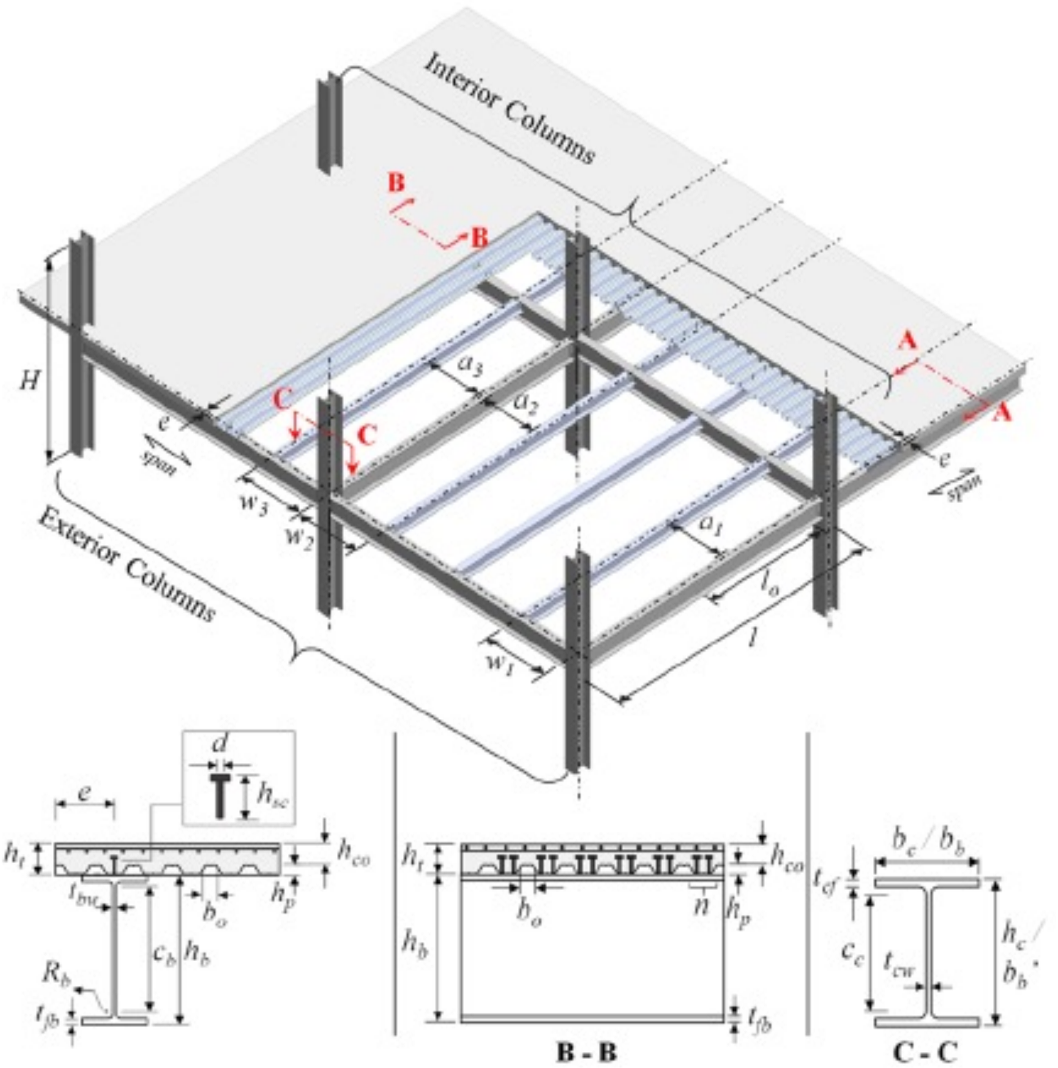
Composite-steel moment resisting frames



Images courtesy of D. Lignos

- Enhanced lateral stiffness and strength due to composite action
- Potentially lighter steel designs for higher degrees of composite action

What is the ‘right’ effective width?

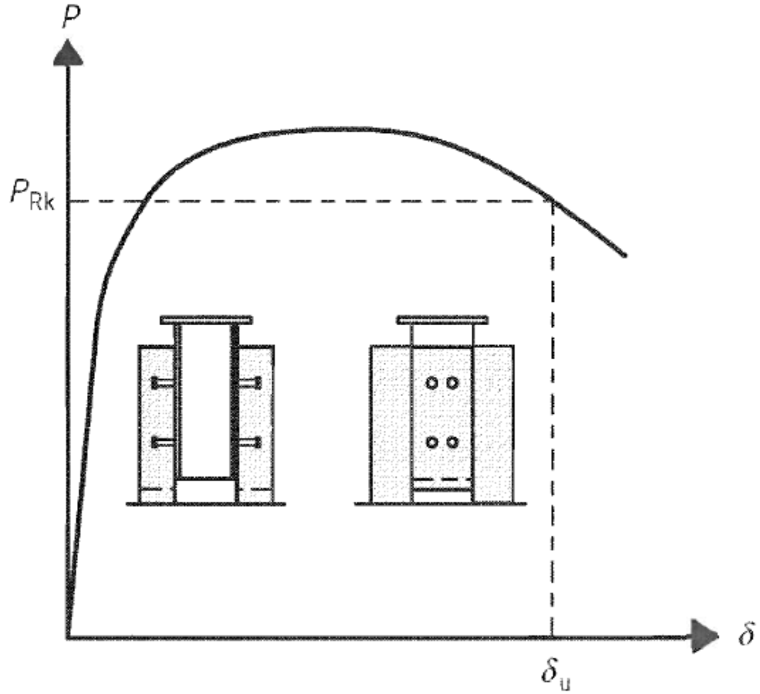


Code	Effective width*	
	Interior column	Exterior column
CEN (2004a)	$\min\begin{cases}0.075l\\ e\end{cases}$	$^{(1)}\min\begin{cases}0.075l\\ e\end{cases}$ $^{(2)}\min\begin{cases}0.5(b_b+0.7h_c)\\ e\end{cases}$ $^{(3)}\min\begin{cases}0.5(b_b'+0.7h_c)\\ 0.05l\\ e\end{cases}$
AISC (2016a)	$\min\begin{cases}0.0125l\\ w\\ e\end{cases}$	
AIJ (2010a)	b_b	

(El Jisr and Lignos, 2019)

Strength of shear connectors

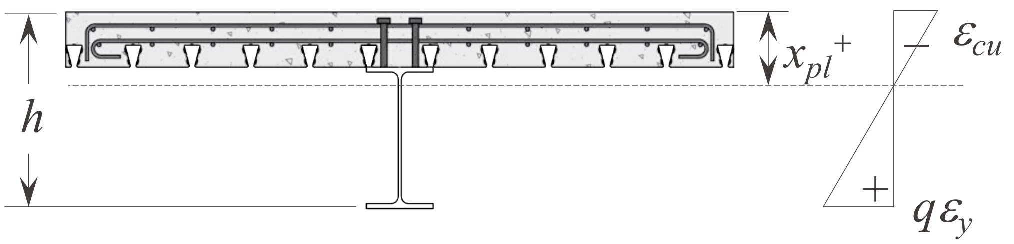
Image courtesy of Prof. D. Lignos



@EN 1994-1-1 Annex D (2004)

- In seismic applications: additional 25% reduction regardless of the beam depth
- Headed shear connectors to be ductile ($h_D \geq 4d$)
- Deformability $\delta_{uk} \geq 6mm$

Ductility requirements for controlling concrete crushing within a dissipative zone



$$\frac{x_{pl}^{+}}{h} \leq \frac{\varepsilon_{cu}}{\varepsilon_{cu} + q \cdot \varepsilon_y}$$

Current EN1998-1-1 (Chapter 11)

Not allowed

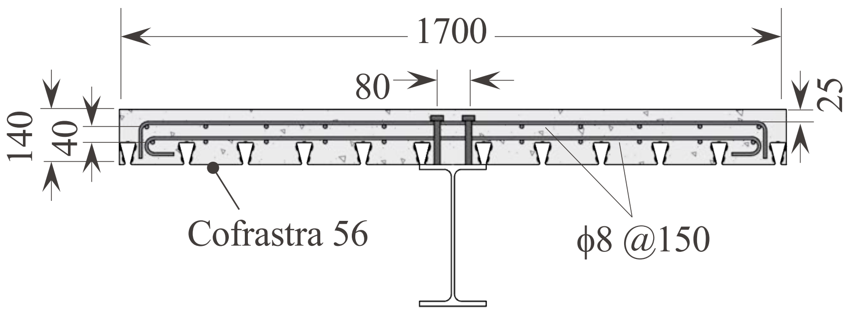
Fully Composite Beam										
	L [mm]	z/d								Limit
q		IPE270	IPE300	IPE330	IPE360	IPE400	IPE450	IPE500	IPE550	
1.5	5000	0.336	0.317	0.315	0.345	0.387	0.414	0.439	0.457	0.50
	6000	0.332	0.313	0.297	0.284	0.332	0.368	0.400	0.424	0.50
	7000	0.327	0.309	0.294	0.281	0.277	0.323	0.361	0.391	0.50
	8000	0.322	0.305	0.290	0.278	0.262	0.277	0.322	0.358	0.50
	9000	0.318	0.301	0.287	0.275	0.259	0.242	0.284	0.325	0.50
2	5000	0.336	0.317	0.315	0.345	0.387	0.414	0.439	0.457	0.43
	6000	0.332	0.313	0.297	0.284	0.332	0.368	0.400	0.424	0.43
	7000	0.327	0.309	0.294	0.281	0.277	0.323	0.361	0.391	0.43
	8000	0.322	0.305	0.290	0.278	0.262	0.277	0.322	0.358	0.43
	9000	0.318	0.301	0.287	0.275	0.259	0.242	0.284	0.325	0.43
3.5	5000	0.336	0.317	0.315	0.345	0.387	0.414	0.439	0.457	0.30
	6000	0.332	0.313	0.297	0.284	0.332	0.368	0.400	0.424	0.30
	7000	0.327	0.309	0.294	0.281	0.277	0.323	0.361	0.391	0.30
	8000	0.322	0.305	0.290	0.278	0.262	0.277	0.322	0.358	0.30
	9000	0.318	0.301	0.287	0.275	0.259	0.242	0.284	0.325	0.30
5.5	5000	0.336	0.317	0.315	0.345	0.387	0.414	0.439	0.457	0.21
	6000	0.332	0.313	0.297	0.284	0.332	0.368	0.400	0.424	0.21
	7000	0.327	0.309	0.294	0.281	0.277	0.323	0.361	0.391	0.21
	8000	0.322	0.305	0.290	0.278	0.262	0.277	0.322	0.358	0.21
	9000	0.318	0.301	0.287	0.275	0.259	0.242	0.284	0.325	0.21
6.5	5000	0.336	0.317	0.315	0.345	0.387	0.414	0.439	0.457	0.19
	6000	0.332	0.313	0.297	0.284	0.332	0.368	0.400	0.424	0.19
	7000	0.327	0.309	0.294	0.281	0.277	0.323	0.361	0.391	0.19
	8000	0.322	0.305	0.290	0.278	0.262	0.277	0.322	0.358	0.19
	9000	0.318	0.301	0.287	0.275	0.259	0.242	0.284	0.325	0.19

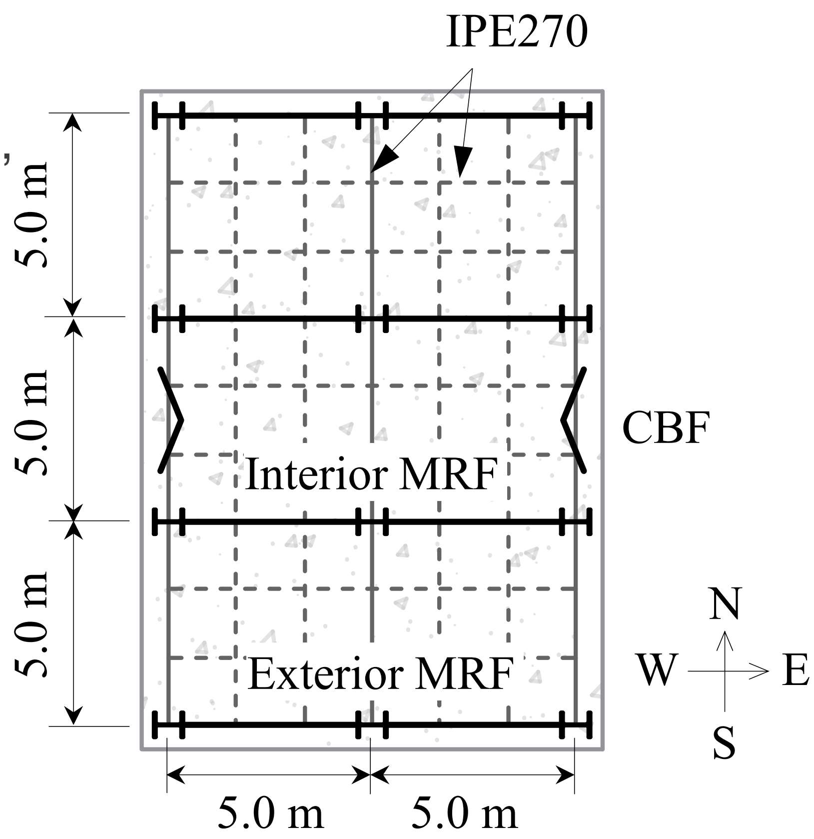
- In current provisions, assumed $\varepsilon_{cu} = 0{,}0025$

- Very restrictive for seismic designs of composite steel MRFs with $q > 2$

Overview of prototype building

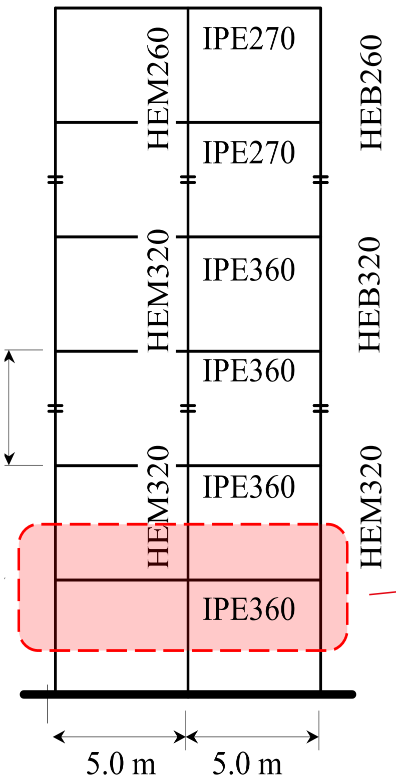
- 2-bay composite-steel MRFs (S355J2 steel, C25)
- Site Class D, a_g=0.22g
- Design location: Sion (CH), Katerini (GR), Braila (RO), Rimini (IT)
- Degree of composite action, $n = 80\%$
- Assumed, strength reduction factor, $q = 3$
- 25% reduction in shear stud resistance is waived
- Stiffened end-plate beam-to-column connections
- Fabrication: EXC2 according to EN1090-2

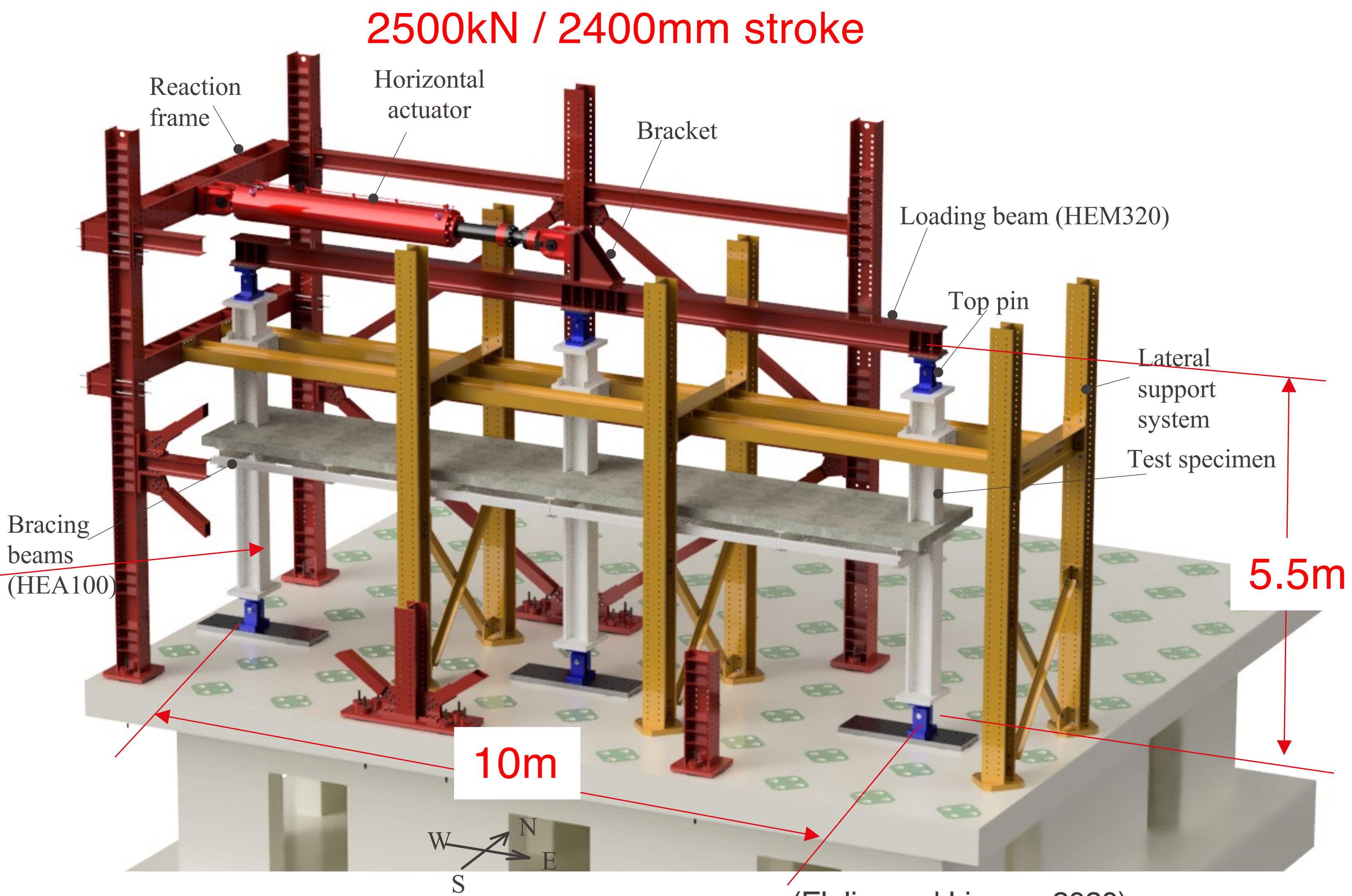




(El Jisr and Lignos, 2020)

Test structure @ EPFL





(El Jisr and Lignos, 2020)

Test structure after setup completion

Image courtesy of D. Lignos

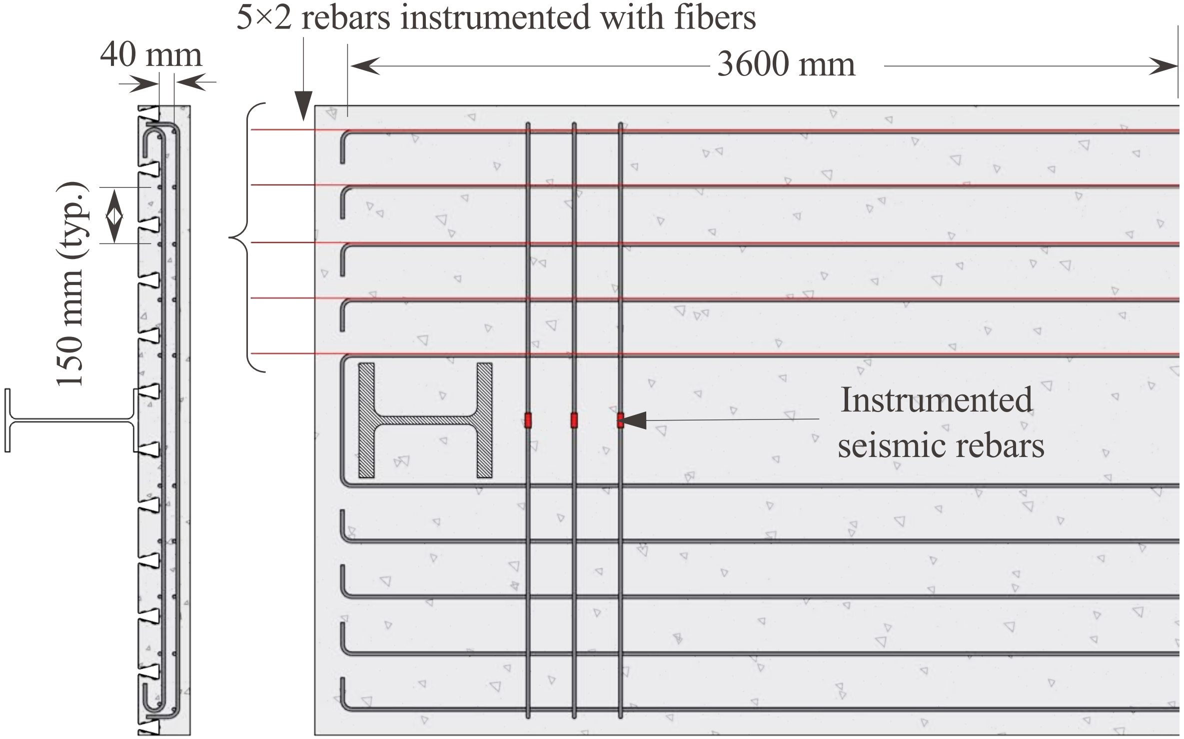
Test structure after setup completion (2)

Image courtesy of D. Lignos

Instrumentation

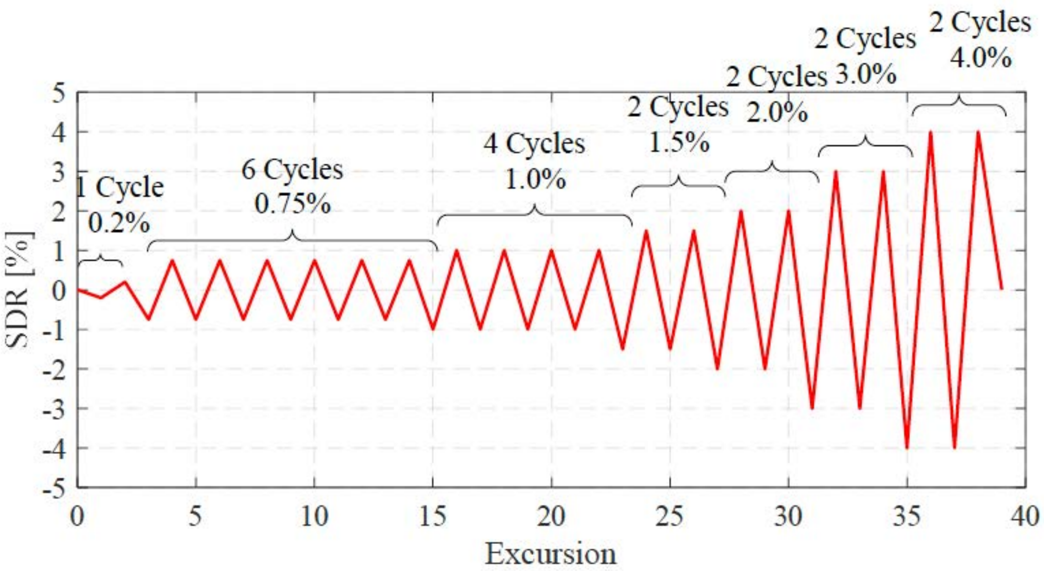
- Digital image correlation system (8 cameras) to track strains on the slab surface
- LUNA: fiber optic cables for continuous strain measurements on steel reinforcement
- LED Wireless tracking system for displacement tracking
- Conventional instrumentation: About 360 other sensors

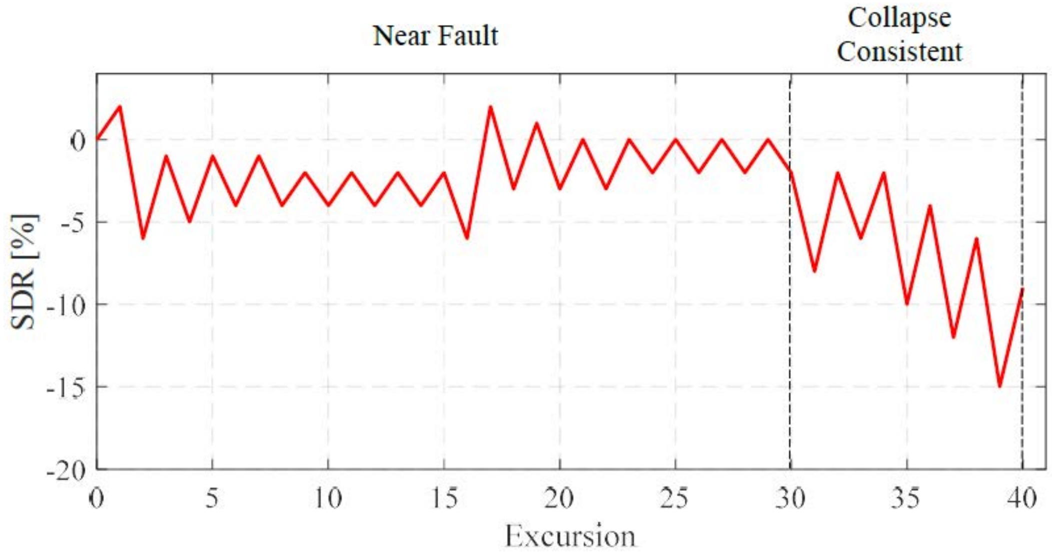
Digital image correlation system



(El Jisr and Lignos, 2020)

Employed loading protocols



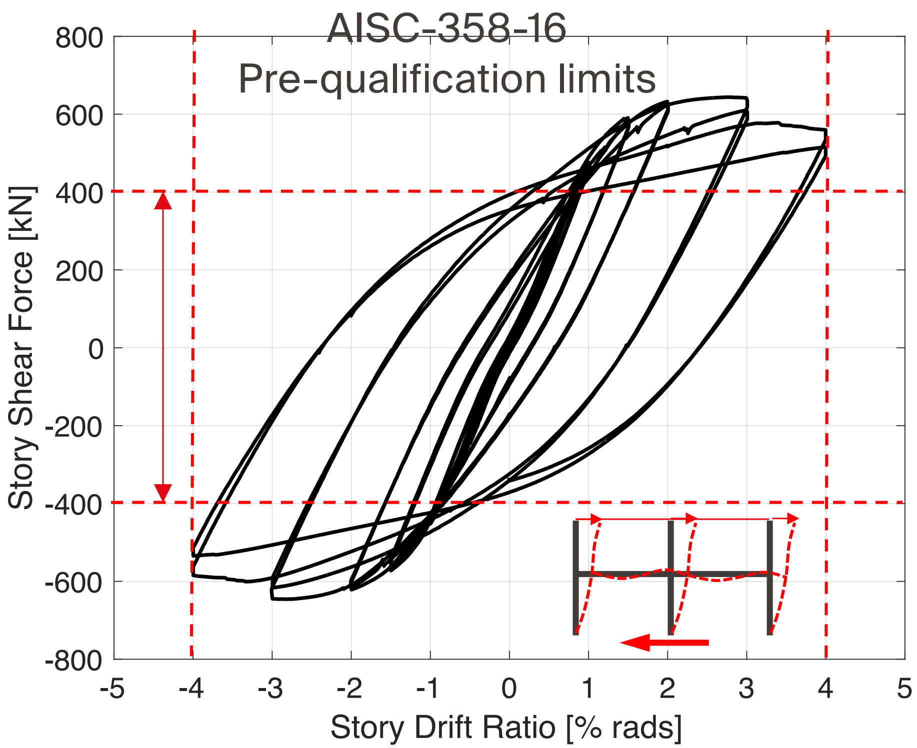


- AISC symmetric cyclic protocol (to evaluate pre-qualification)
- SAC near fault protocol (prototype design location near fault)
- Collapse-consistent protocol to mimic “ratcheting” prior to structural collapse

(El Jisr and Lignos, 2020)

Selected experimental findings

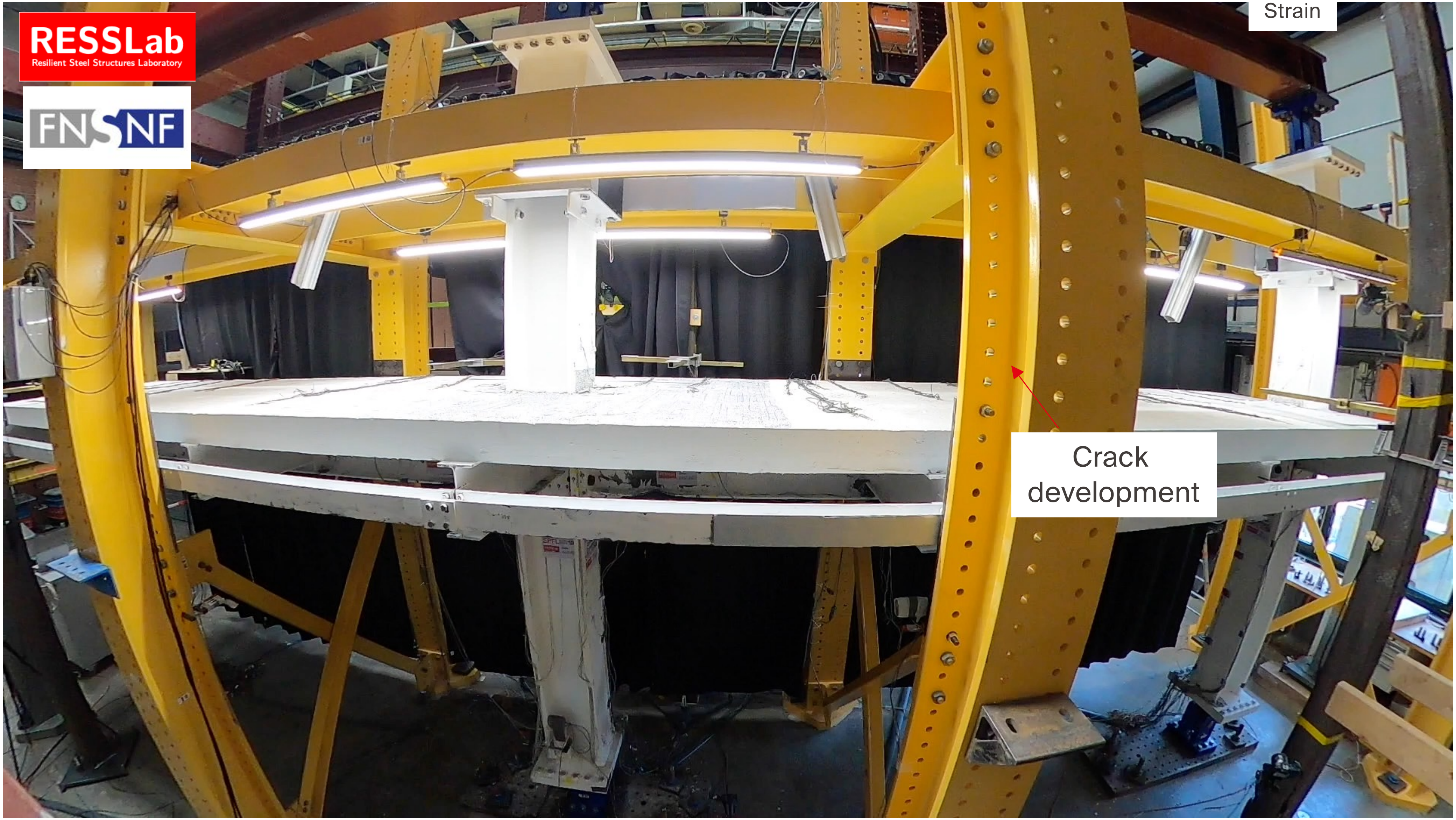
-AISC loading protocol



East beam: Exterior Joint

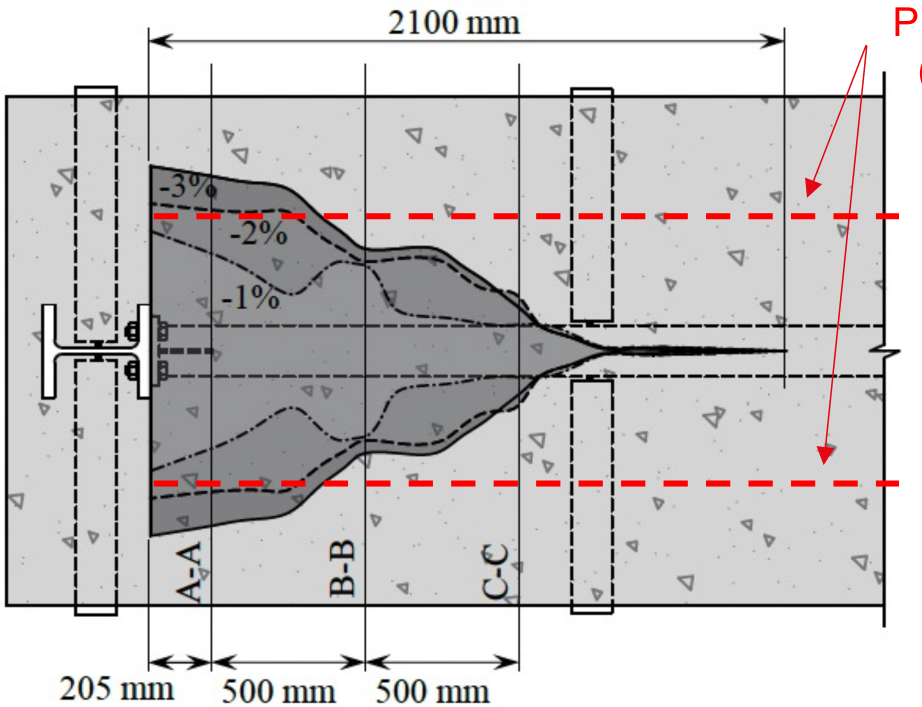
(El Jisr and Lignos, 2020)

Seismic performance of composite-steel moment frames



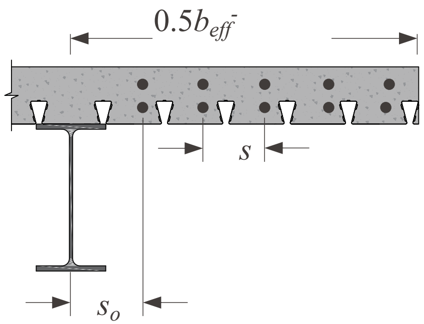
(El Jisr and Lignos, 2020)

Slab effective width



Proposed value according to AISC-2016
(adopted in New Eurocode 8 Part 1-2)

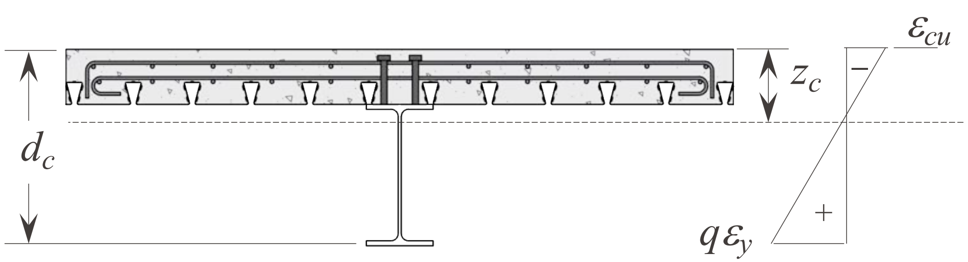
Digitized section cut in composite beams

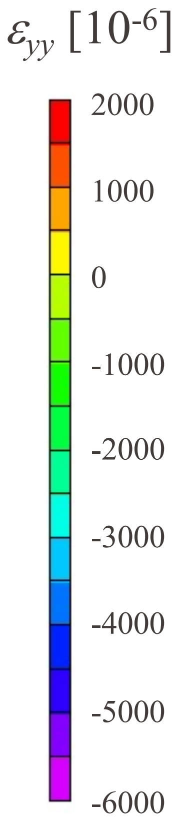


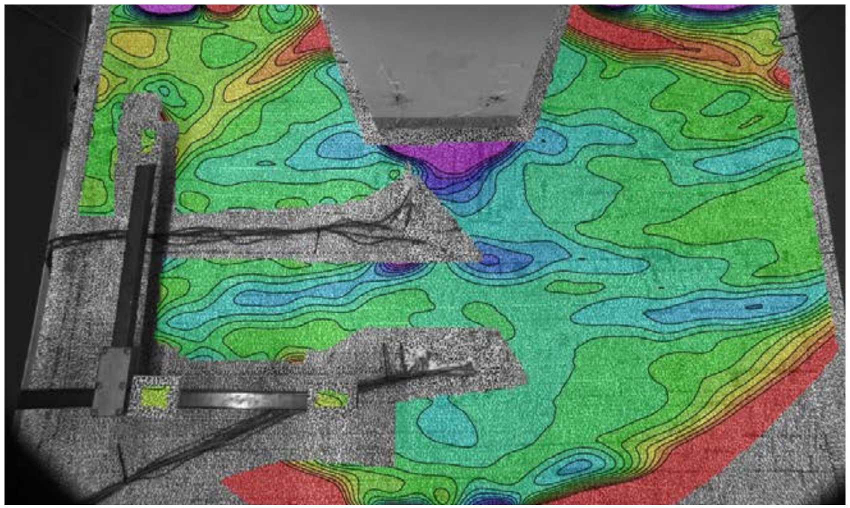
$$b_{eff}^{-} = 2\left[\left(\frac{\sum \sigma_{s,i}}{2f_{y,n}} - 1\right)\cdot s + s_0\right]$$

(El Jisr and Lignos, 2021)

Revised ductility requirements for composite-steel beams







@Story Drift Ratio = 2% rad
(significant damage limit state)

New Table 12.6 (EN1998-1-2: 2022)

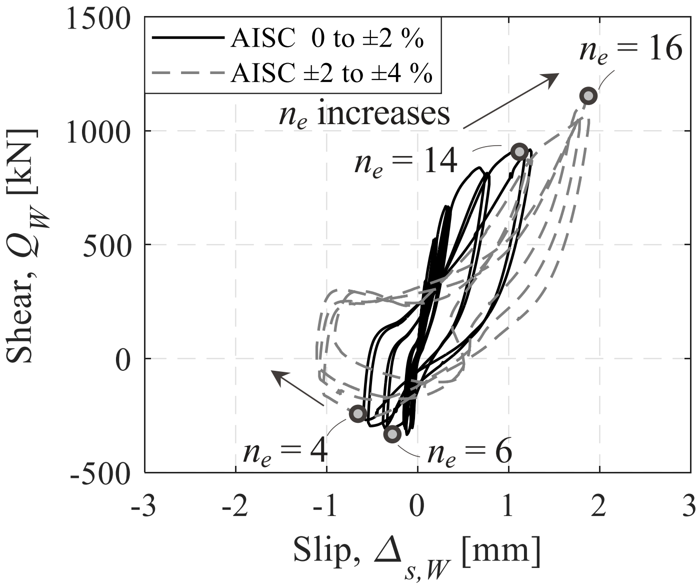
Ductility Class	q	f_y (N/mm²)	z_c/d_c upper limit
DC2	$1,5 < q \leq 3,5$	355	0,45
		275	0,50
		235	0,55
DC3	$3,5 < q \leq 5,0$	355	0,35
		275	0,40
		235	0,45
	$q > 5,0$	355	0,30
		275	0,35
		235	0,40

(El Jisr and Lignos, 2021)

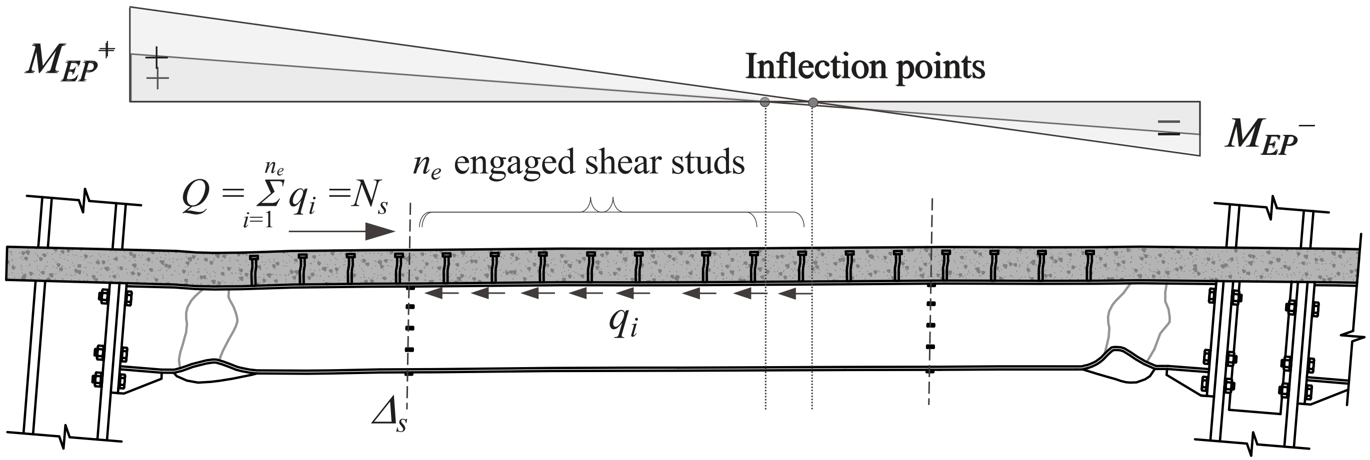
- Revised limit for confined concrete crushing, $\varepsilon_{cu} = 4,5‰$

Behavior of shear stud connectors

Beam-slab connection response



Shear stud participation during loading



(El Jisr and Lignos, 2021)

Behavior of shear stud connectors

-slab cut along the beam length (after the end of test)

Slab

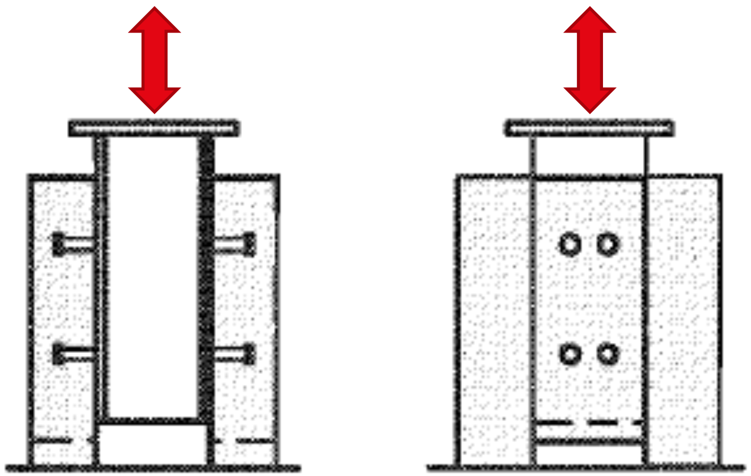
Steel beam

(El Jisr and Lignos, 2021)

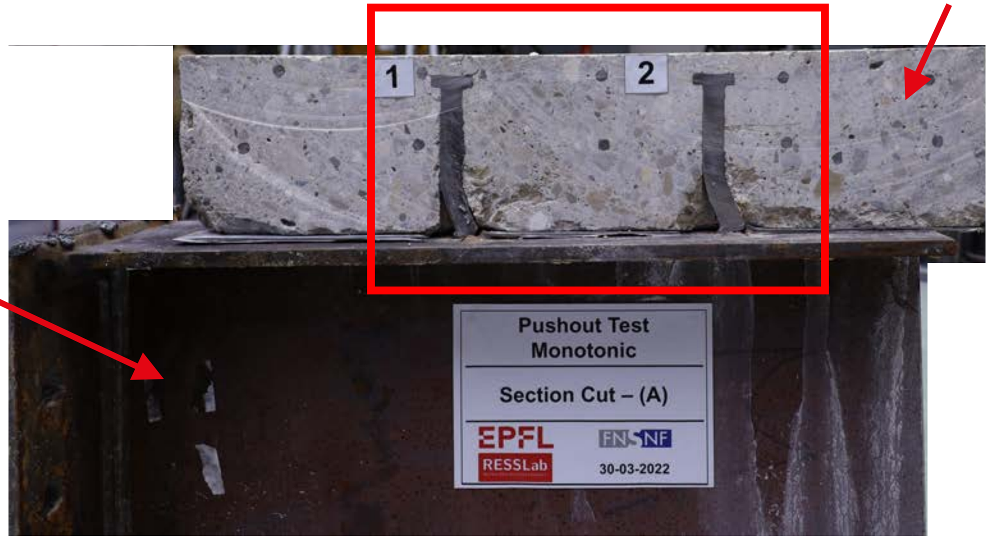
Behavior of shear stud connectors
-from typical push-out test

Slab

Steel beam



Push-out test



(El Jisr and Lignos, 2021)

Shear connectors in composite moment frames

-Section cuts after end of testing and design recommendations

Slab

Steel beam (IPE330)

(El Jisr and Lignos, 2021)

- For composite beams with $d_b \leq 500mm$:

→25% reduction on shear resistance is waived (adopted in EC8 Part 1-2)

EC8 Webinars program (https://ec8webinars.org/webinars/#webinars)

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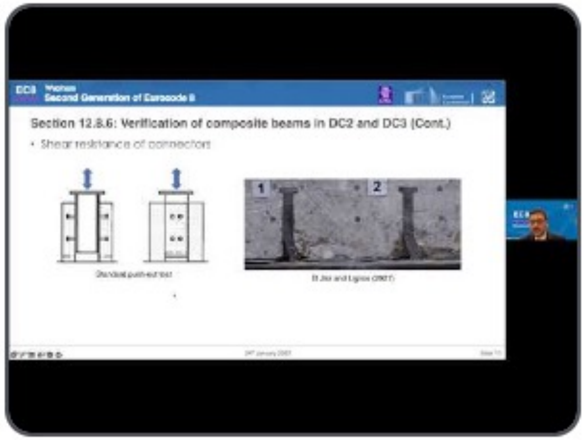
Webinar 1-2.4: **Composite Steel-Concrete Buildings**

Speaker: Dimitrios Lignos



Video: ***open in Youtube***

Presentation slides: ***open view***

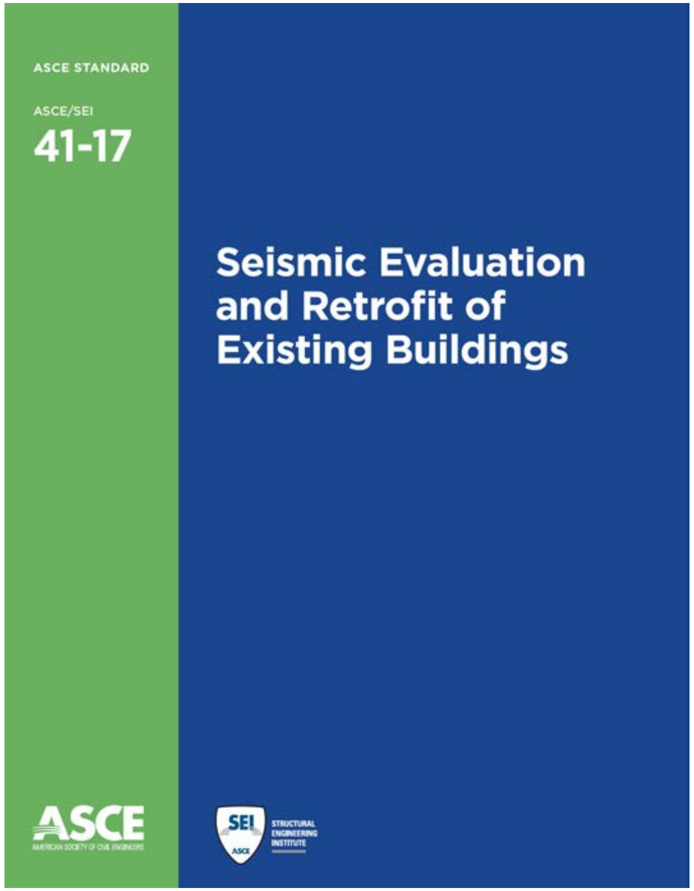
Link to all seminars

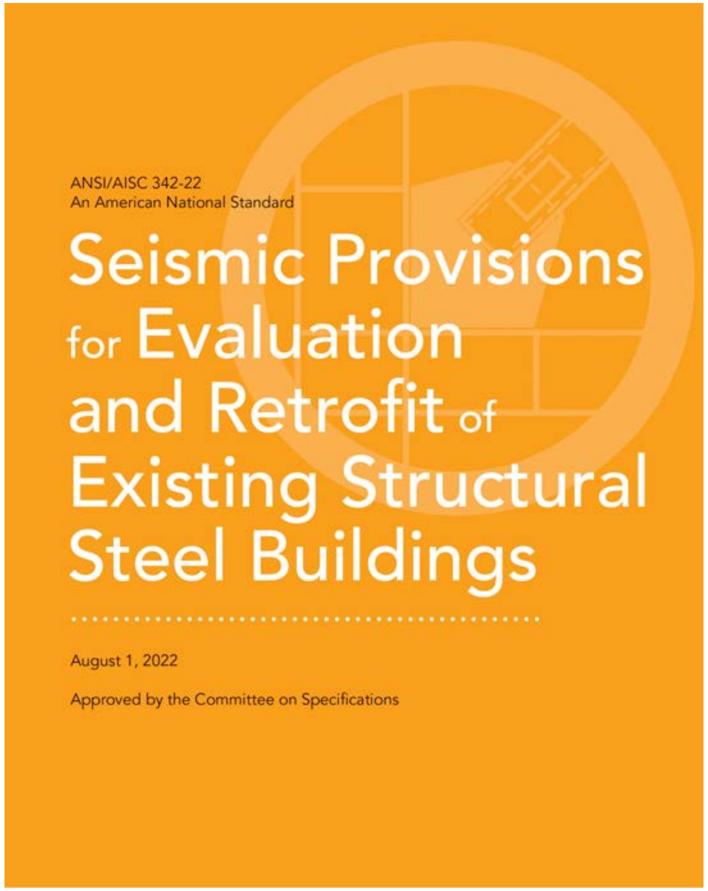
https://ec8webinars.org/recorded-webinars/#webinar-1-2

Link for composite-steel concrete frames

https://www.youtube.com/watch?v=5S3dCmijtg4

Practice-oriented models for seismic assessment





EUROPEAN STANDARD
NORME EUROPÉENNE
EUROPÄISCHE NORM

EN 1998-3

June 2005

ICS 91.120.25

Supersedes ENV 1998-1-4:1996

English version

Eurocode 8: Design of structures for earthquake resistance - Part 3: Assessment and retrofitting of buildings

Eurocode 8: Calcul des structures pour leur résistance aux séismes - Partie 3: Evaluation et renforcement des bâtiments

Eurocode 8: Auslegung von Bauwerken gegen Erdbeben - Teil 3: Beurteilung und Ertüchtigung von Gebäuden

This European Standard was approved by CEN on 15 March 2005.

CEN members are bound to comply with the CEN/CENELEC Internal Regulations which stipulate the conditions for giving this European Standard the status of a national standard without any alteration. Up-to-date lists and bibliographical references concerning such national standards may be obtained on application to the Central Secretariat or to any CEN member.

This European Standard exists in three official versions (English, French, German). A version in any other language made by translation under the responsibility of a CEN member into its own language and notified to the Central Secretariat has the same status as the official versions.

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Ref. No. EN 1998-3:2005: E

Current Eurocode 8 – Part 3

Resistance & deformation models for assessment (steel beams & columns)

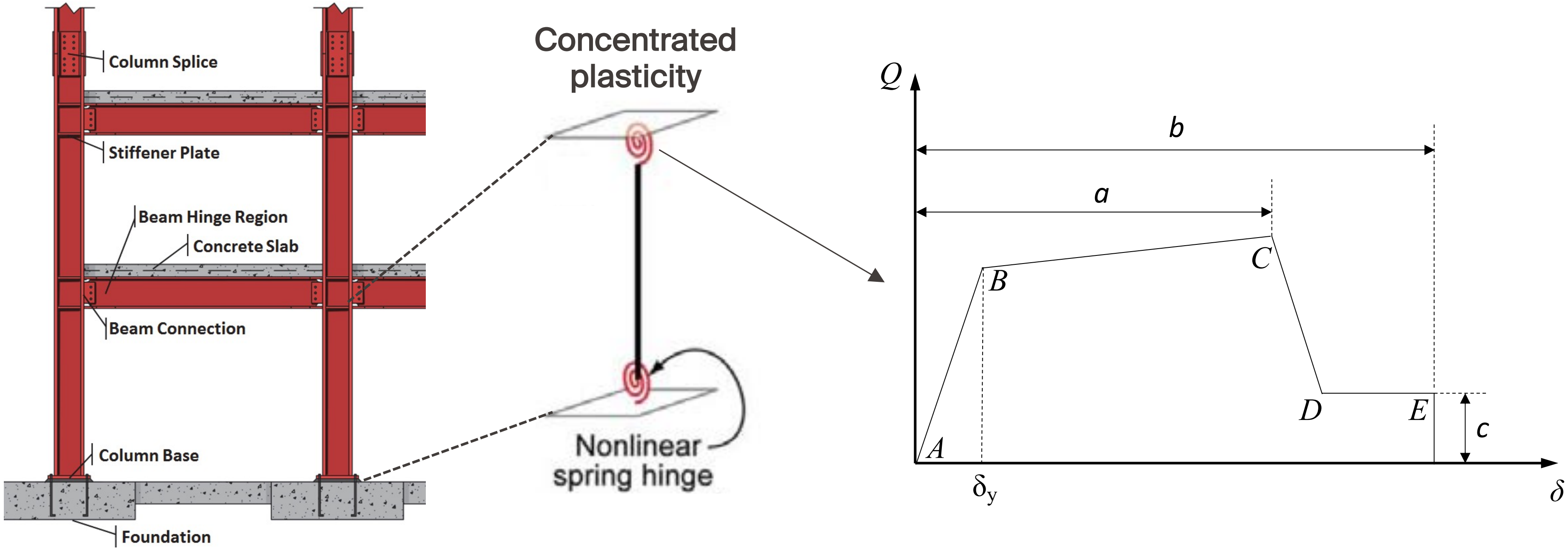


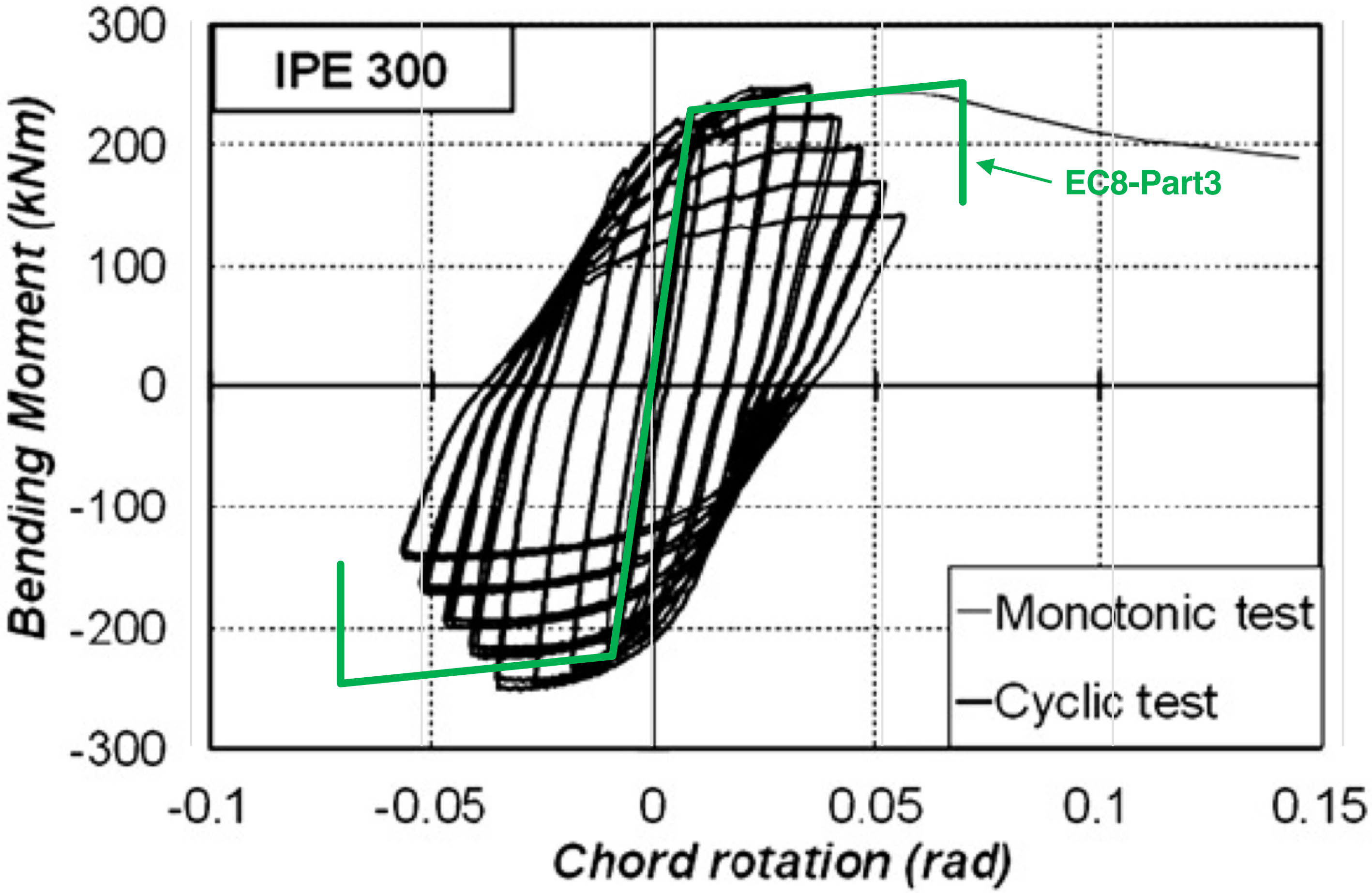
Table B.1: Plastic rotation capacity at the end of beams or columns with dimensionless axial load ν not greater than 0,30.

	Limit State		
Class of cross section	DL	SD	NC
1	1,0 θ_y	6,0 θ_y	8,0 θ_y
2	0,25 θ_y	2,0 θ_y	3,0 θ_y

@EN1998-3

Current Eurocode 8 – Part 3

Evaluation of deformation models for assessment (steel beams)

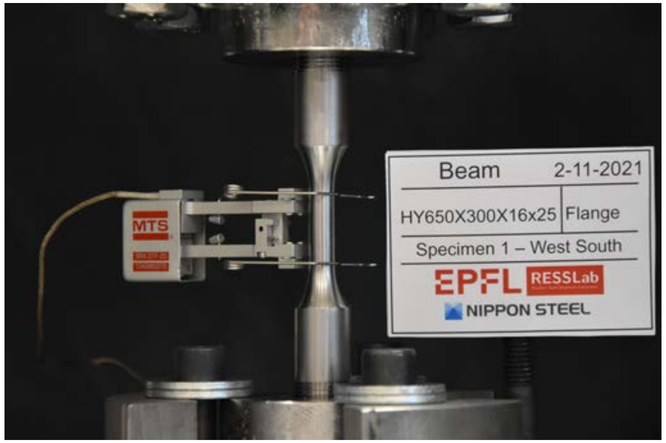


Cantilever steel beam test data (D'Aniello et al. 2012)

Material and structural performance databases

-over 1500 collected experiments

Material scale (over 10 steel grades)

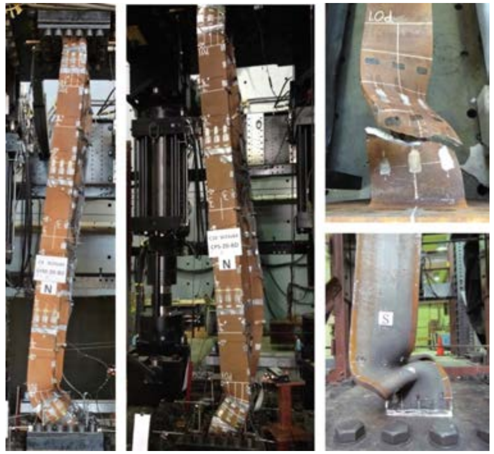


Hartloper et al. (2023)

Steel beam-to-column joints

Lignos and Krawinkler (2011)

I- H- shaped steel columns

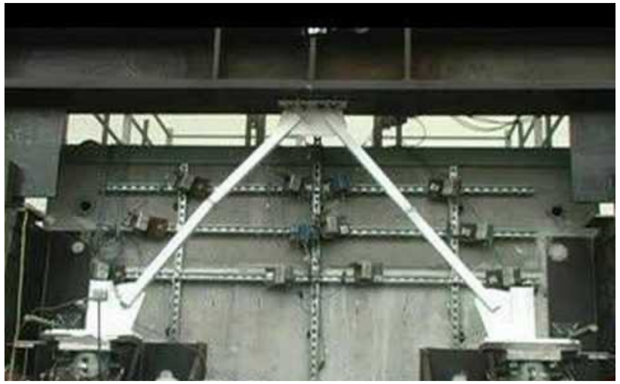


Elkady and Lignos (2018)

Hollow structural steel columns

Lignos and Krawinkler (2010)

Steel braces (HSS, round HSS, I-shaped)

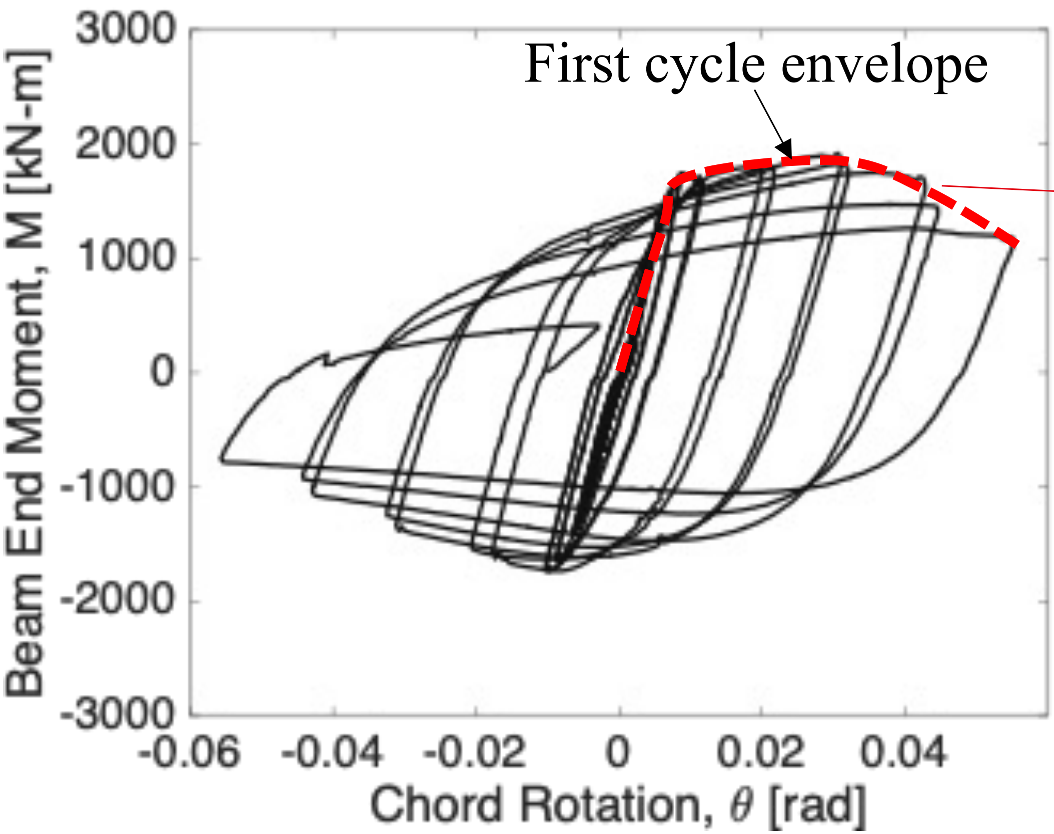


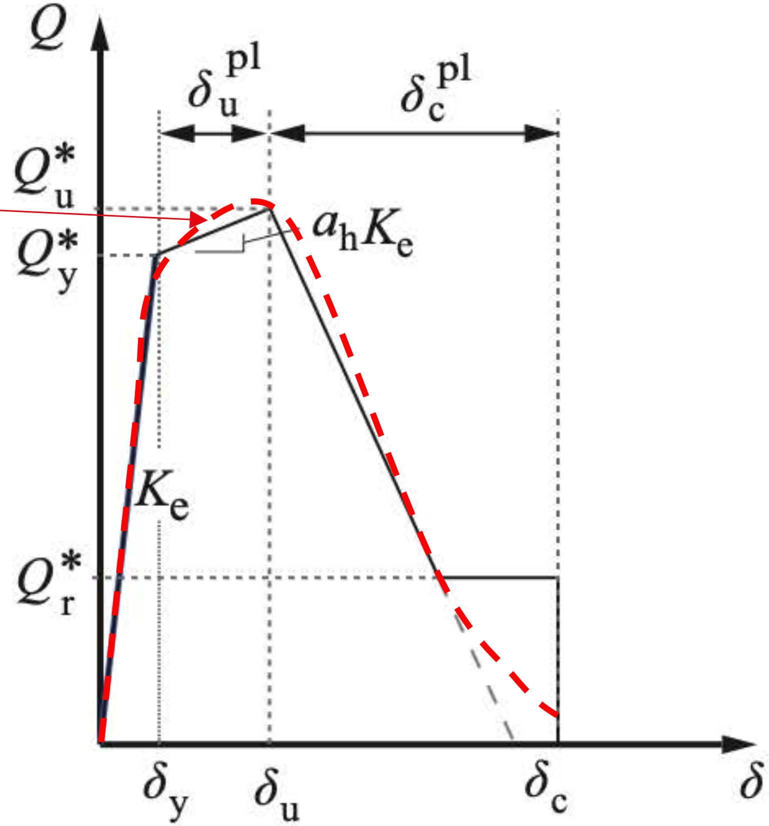
Karamanci and Lignos (2014)

Beam-to-column web panel

Skiadopoulos and Lignos (2021)

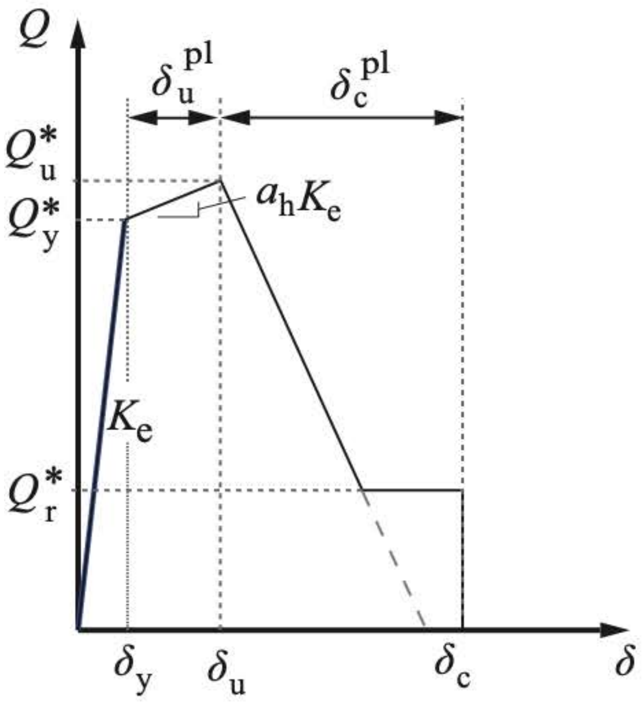
Resistance & deformation models for assessment





Resistance & deformation models

I- and H-shaped steel columns



Resistance models

$$M_y^* = 1{,}15\omega_{rm}\left(1 - \frac{N_{Ed,G}}{\chi_z N_{Rk}/\gamma_{M1}}\right)\chi_{LT} M_{y,Rk}/\gamma_{M1}$$

$$M_u^* = M_y^* + a_h K_e \theta_u^{pl}$$

Deformation models

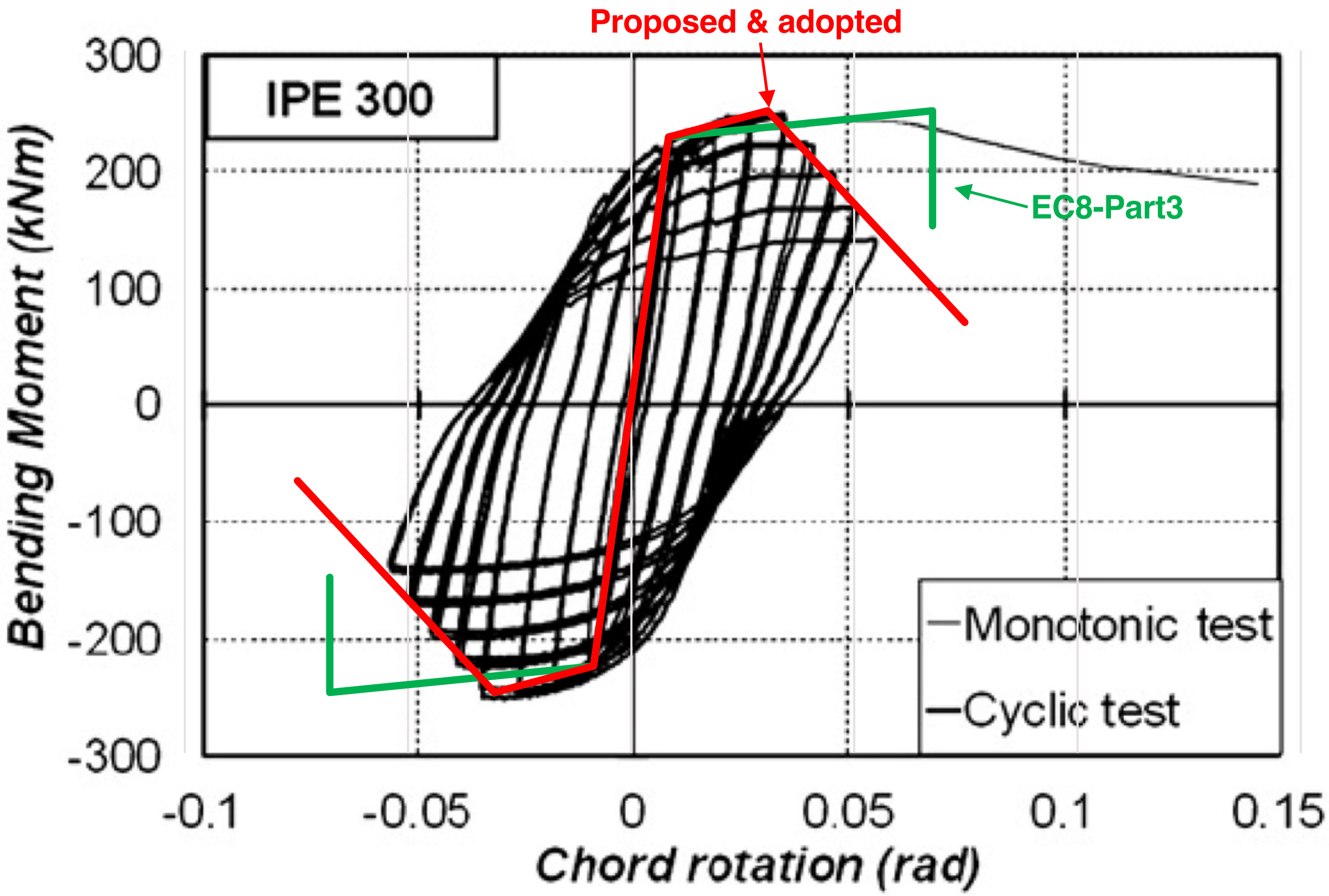
$$\delta_u^{pl} = 7{,}37\left(\frac{h}{t_w}\right)^{-0{,}95}\left(\frac{L_b}{i_z}\right)^{-0{,}5}\left(1 - \frac{N_{Ed,G}}{N_{pl,e}}\right)^{2{,}4} \leq 0{,}15\text{rad} \quad (COV = 0.38)$$

$$\delta_c^{pl} = 20\left(\frac{h}{t_w}\right)^{-0{,}9}\left(\frac{L_b}{i_z}\right)^{-0{,}5}\left(1 - \frac{N_{Ed,G}}{N_{pl,e}}\right)^{3{,}4} \leq 0{,}07\text{rad} \quad (COV = 0.42)$$

Source: Lignos et al. (2019)

Proposed resistance models for assessment

-Typical steel beam



Cantilever steel beam test data (D'Aniello et al. 2012)

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WEBINAR 3: Assessment and retrofitting of buildings and bridges

Webinar 3.4: Assessment and retrofit of steel structures

Speaker: Dimitrios Lignos

Video: ***open in Youtube***

Presentation slides: ***open view***

Link to all seminars

https://ec8webinars.org/recorded-webinars/#webinar-1-2

Link for Clause 9 of EC8 Part 3 (seismic assessment)

https://www.youtube.com/watch?v=3lZPHQfkB0Y

Summary and conclusions

- **Seismic stability of steel columns:**
  - Axial shortening, coupled instabilities
  - Proposed limits on compressive axial load ratio & web slenderness
  - Influence of loading history (subduction versus near fault)
- **Beam-to-column web panel zone joints:**
  - Existing models do not work well with columns having thick flanges (>40mm) and beam-to-column depth ratios larger than 1
  - Proposed model for design: addresses all previous limitations
- **Composite effects for seismic action:**
  - Additional 25% reduction on shear resistance can be waived for beam depths smaller than 500mm
  - New recommendations for effective width
- **Practice-oriented models for seismic assessment of existing structures:**
  - New Chapter 9 in Eurocode 8 Part 3 (second generation of Eurocodes)
  - Revisions on columns in ASCE-41 (2023 version)

EPFL

Thank you very much for your kind attention!

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