Innovations in Earthquake Resistant Steel Structures

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Selected Recent Innovations

- Expanding range of applicability of a number of new and emerging structural steel systems that can provide effective seismic performance.
  - Buckling Restrained Braced
    - Designed to meet Structural Fuse objectives
  - Rocking braced frames.
  - Tubular Eccentrically Braced Frames
  - Steel Plate Shear Walls

Acknowledgments

- Ph.D. Students:
  - Bing Qu – *Seismic Performance of Buildings with Steel Plate Shear Walls*
  - Michael Pollino – *Rocking Steel Framed Systems*
  - Jeffrey Berman – *Seismic Retrofit of Large Bridges Braced Bent*
  - Ramiro Vargas – *Enhancing Resilience using Passive Energy Dissipation Systems*
  - Darren Vian – *Passive Energy Dissipation using Metallic In-fills*
  - Shuichi Fujikura – *Multi-Hazard Resilient Bridges*
- M.Sc. Students:
  - Ronny Purba – *Design of Perforated Steel Plate Shear Walls*
  - Jeffrey Berman – *Thin Steel Infill Walls as Passive Energy Dissipators for the Seismic Retrofit of Hospitals*
- Post-Doc: Gordon Warn – *Blast Resistance of Steel Plate Shear Walls*
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  - National Science Foundation
  - Federal Highway Administration

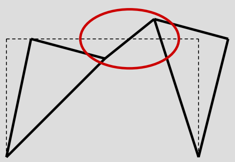
Buckling Restrained Braces in Structural Fuse Application

Structural Fuses

- Earthquake-resistant design has long relied on hysteretic energy dissipation to provide life-safety level of protection
- Advantages of yielding steel
  - Stable material properties well known to practicing engineers
  - Not a mechanical device (no special maintenance)
  - Reliable long term performance (resistance to aging)
- For traditional structural systems, ductile behavior achieved by stable plastic deformation of structural members = damage to those members
- In conventional structural configurations, serves life-safety purposes, but translates into property loss, and need substantial repairs
- Researchers have proposed that hysteretic energy dissipation should instead occur in "disposable" structural elements (i.e., structural fuses)

Roeder and Popov (1977)

"Ductile Fuse"

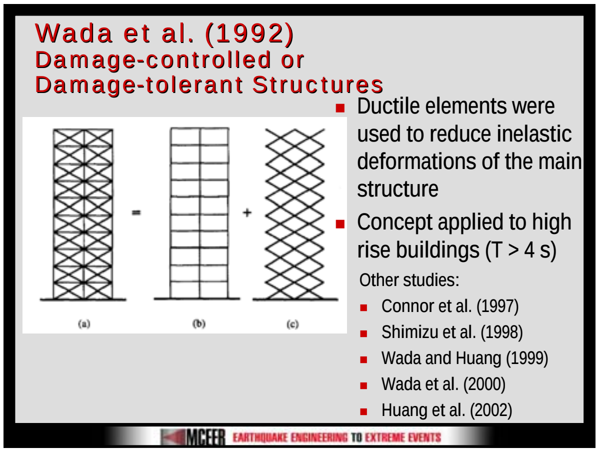


Eccentrically Braced Frame

- Ductile seismic behavior
- Concentrating energy dissipation in special elements + capacity design
- Links not literally disposable

Other studies:

- Fintel and Ghosh (1981)
- Aristizabal-Ochoa (1986)
- Basha and Goel (1996)
- Carter and Iwankiw (1998)
- Sugiyama (1998)
- Rezai et al. (2000)

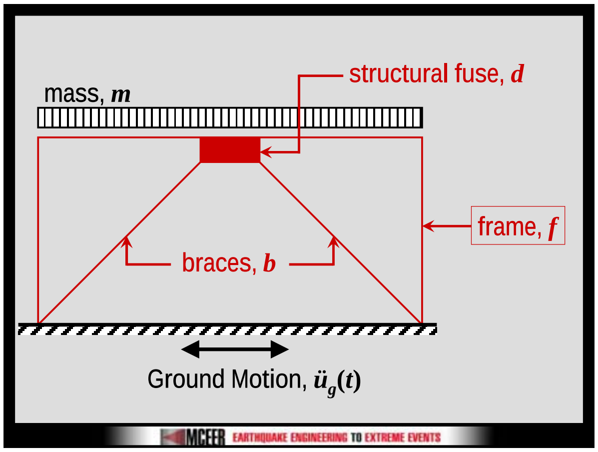


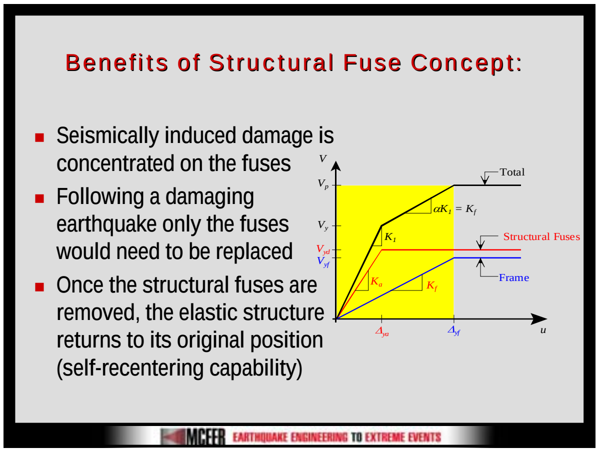
Wada et al. (1992)
Damage-controlled or Damage-tolerant Structures

- Ductile elements were used to reduce inelastic deformations of the main structure
- Concept applied to high rise buildings (T > 4 s)

Other studies:

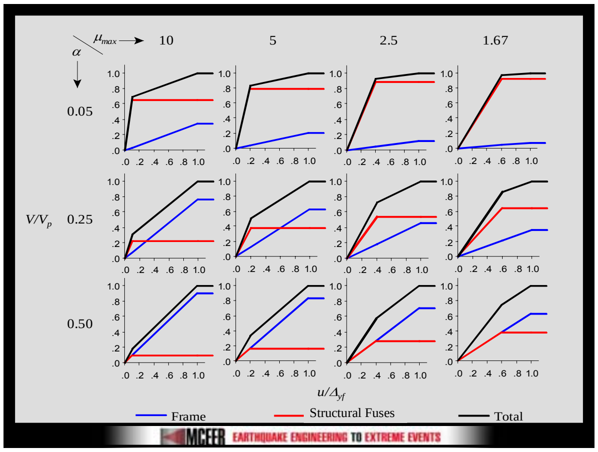
- Connor et al. (1997)
- Shimizu et al. (1998)
- Wada and Huang (1999)
- Wada et al. (2000)
- Huang et al. (2002)

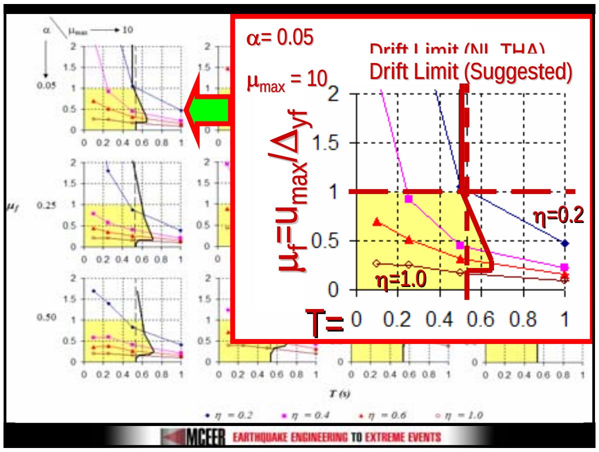


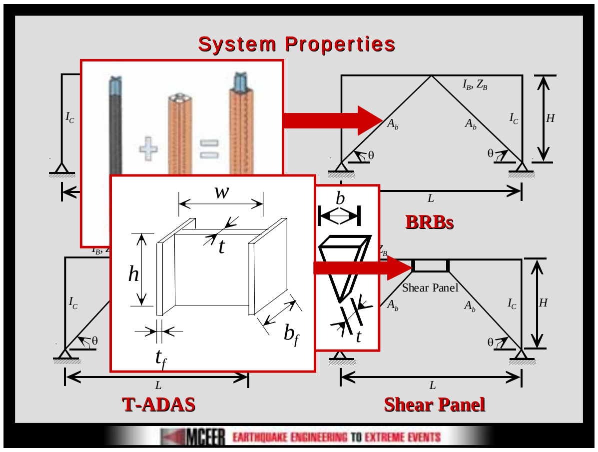


Benefits of Structural Fuse Concept:

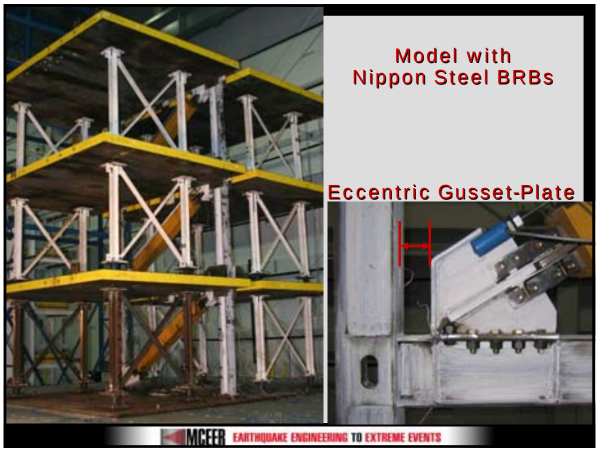
- Seismically induced damage is concentrated on the fuses
- Following a damaging earthquake only the fuses would need to be replaced
- Once the structural fuses are removed, the elastic structure returns to its original position (self-recentering capability)





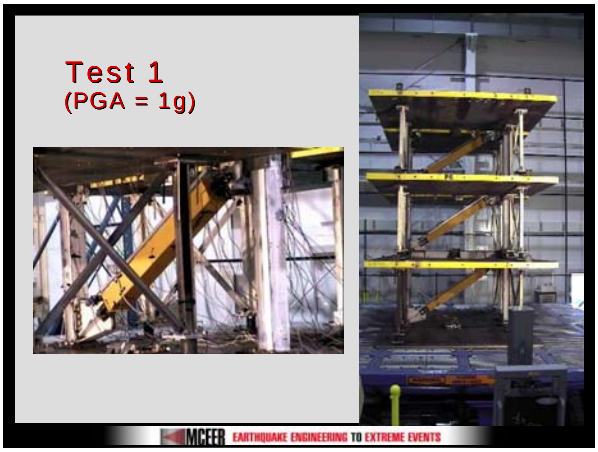


System Properties

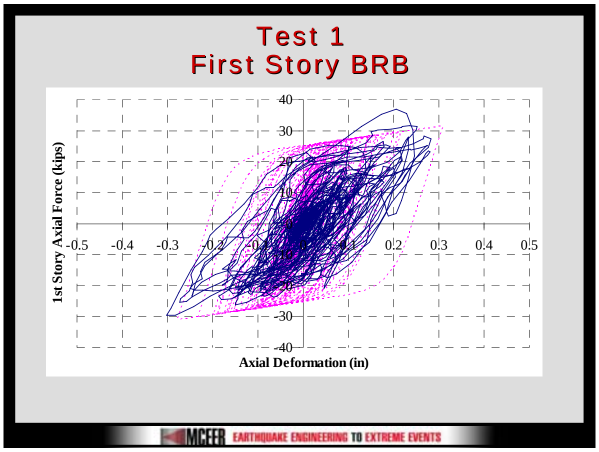


Model with
Nippon Steel BRBs

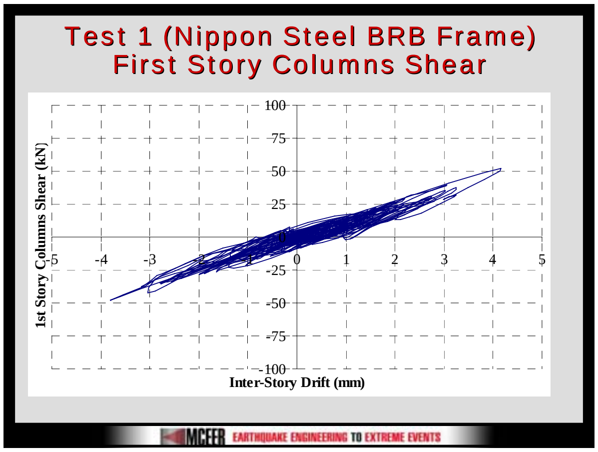
Eccentric Gusset-Plate



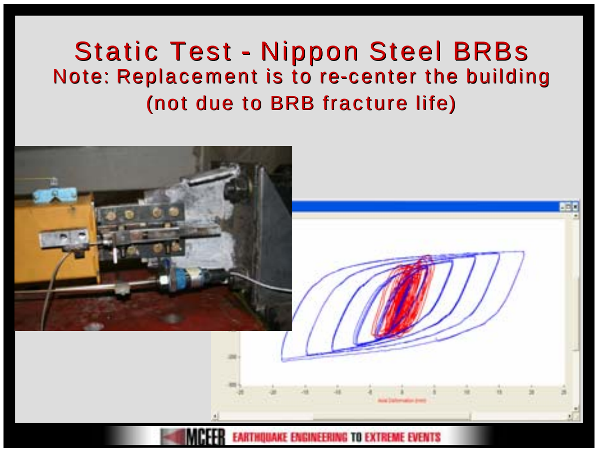
Test 1
(PGA = 1g)



Test 1
First Story BRB



Test 1 (Nippon Steel BRB Frame)
First Story Columns Shear

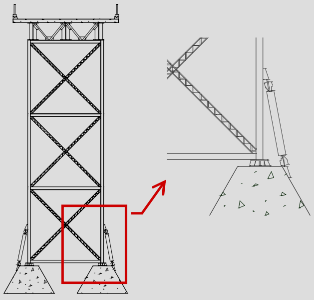


Static Test - Nippon Steel BRBs
Note: Replacement is to re-center the building
(not due to BRB fracture life)

Rocking Trusses
(Rocking Braced Frames)

Controlled Rocking/Energy Dissipation System

- Absence of base of leg connection creates a rocking bridge pier system partially isolating the structure
- Installation of steel yielding devices (buckling-restrained braces) at the steel/concrete interface controls the rocking response while providing energy dissipation



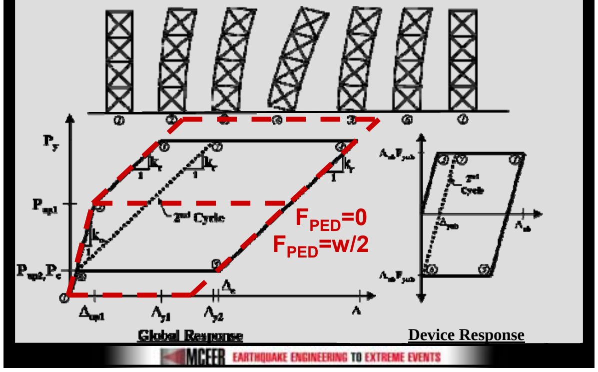
Retrofitted Tower

Existing Rocking Bridges

South Rangitikei Rail Bridge

Lions Gate Bridge North Approach

Static, Hysteretic Behavior of Controlled Rocking Pier



General Design Constraints for Controlled Rocking System

- (1) Deck-level displacement limits need to be established on a case-by-case basis
  - Maintain pier stability
  - Bridge serviceability requirements
- (2) Strains on buckling-restrained brace (uplifting displacements) need to be limited such that it behaves in a stable, reliable manner
- (3) Capacity Protection of existing, vulnerable resisting elements considering 3-components of excitation and dynamic forces developed during impact and uplift
- (4) Allow for self-centering of pier

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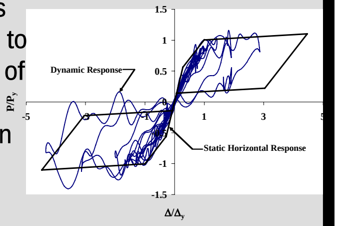
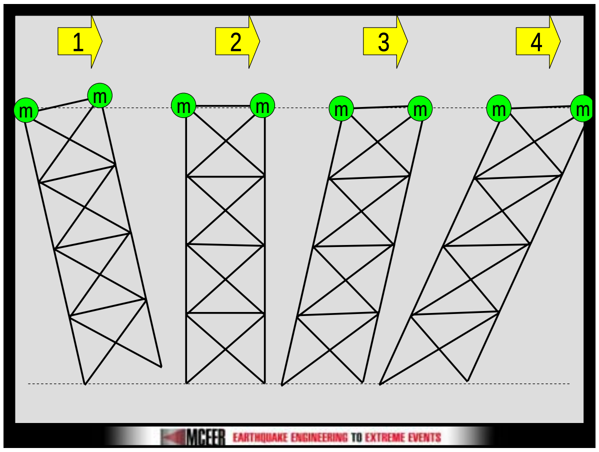
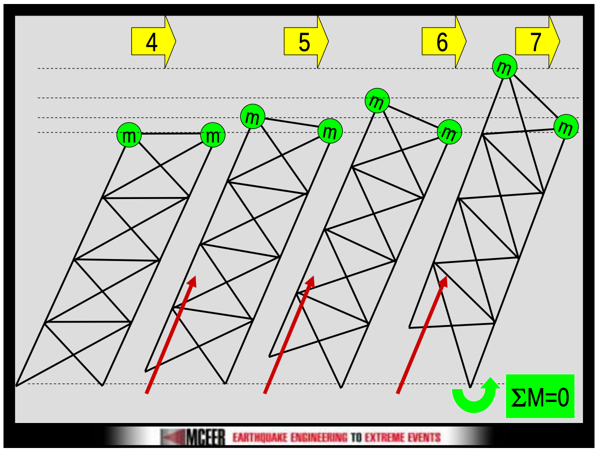
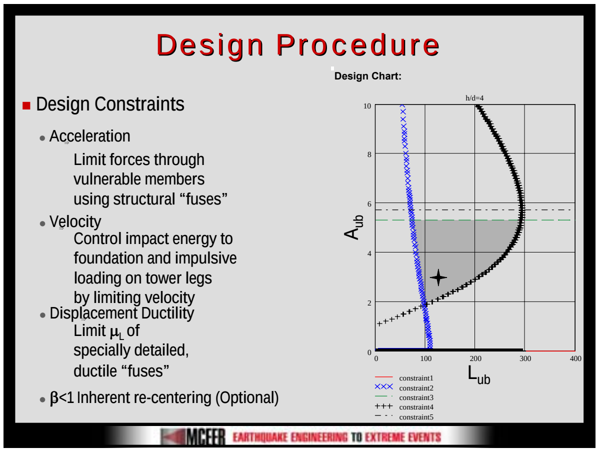
(3) Capacity Protection (cont.)

- An increase from the static response has been observed due to dynamic excitation of vertical modes of vibration even when subjected solely to horizontal base accelerations





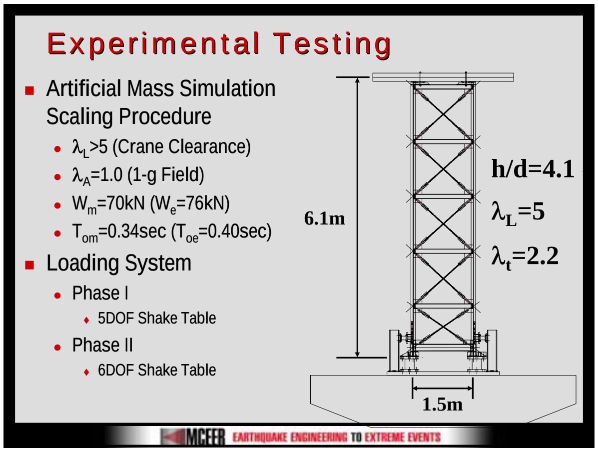




Design Procedure

- Design Constraints
  - Acceleration
    Limit forces through vulnerable members using structural “fuses”
  - Velocity
    Control impact energy to foundation and impulsive loading on tower legs by limiting velocity
  - Displacement Ductility
    Limit μ_L of specially detailed, ductile “fuses”
  - $\beta<1$ Inherent re-centering (Optional)

Design Chart:

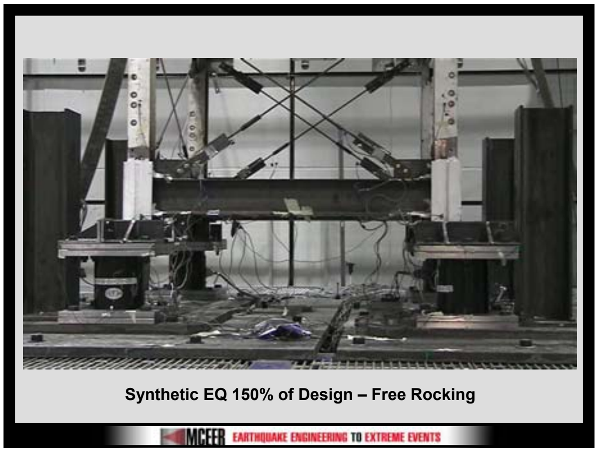


Experimental Testing

- Artificial Mass Simulation Scaling Procedure
  - $\lambda_L>5$ (Crane Clearance)
  - $\lambda_A=1.0$ (1-g Field)
  - $W_m=70kN$ ($W_e=76kN$)
  - $T_{om}=0.34sec$ ($T_{oe}=0.40sec$)
- Loading System
  - Phase I
    - 5DOF Shake Table
  - Phase II
    - 6DOF Shake Table

Synthetic EQ 150% of Design
Free Rocking

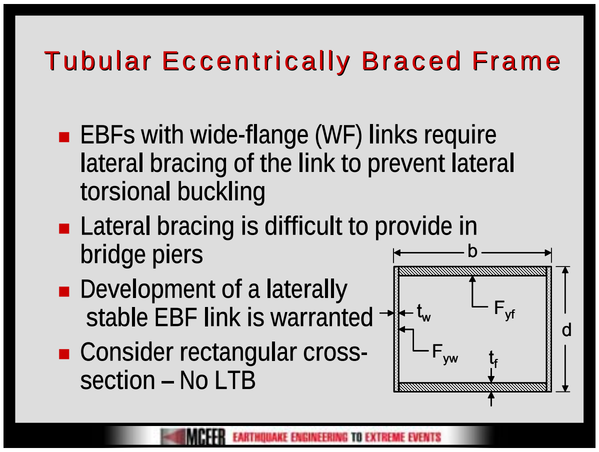
Synthetic EQ 150% of Design
TADAS Case $\eta_L=1.0$



Synthetic EQ 150% of Design – Free Rocking

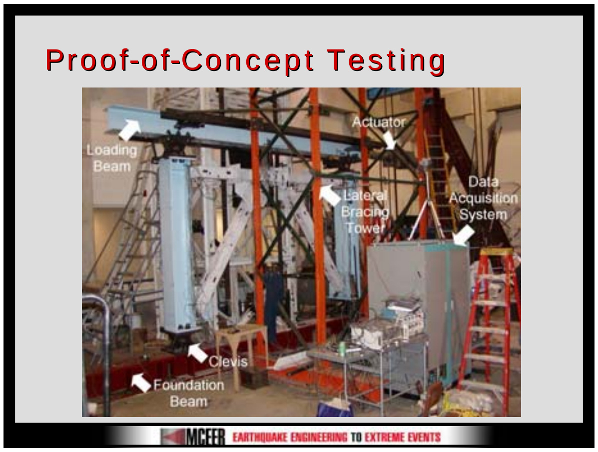
Synthetic EQ 175% of Design - Viscous Dampers

Eccentrically Braced Frames with Tubular Links

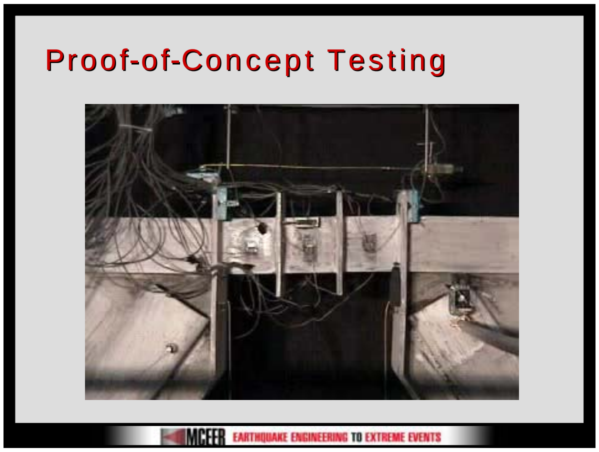


Tubular Eccentrically Braced Frame

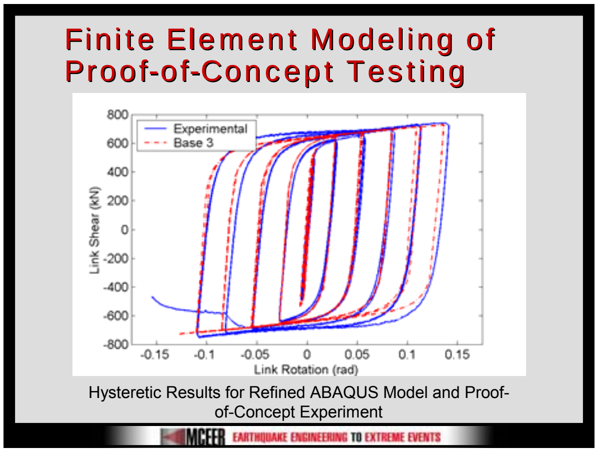
- EBFs with wide-flange (WF) links require lateral bracing of the link to prevent lateral torsional buckling
- Lateral bracing is difficult to provide in bridge piers
- Development of a laterally stable EBF link is warranted
- Consider rectangular cross-section – No LTB



Proof-of-Concept Testing

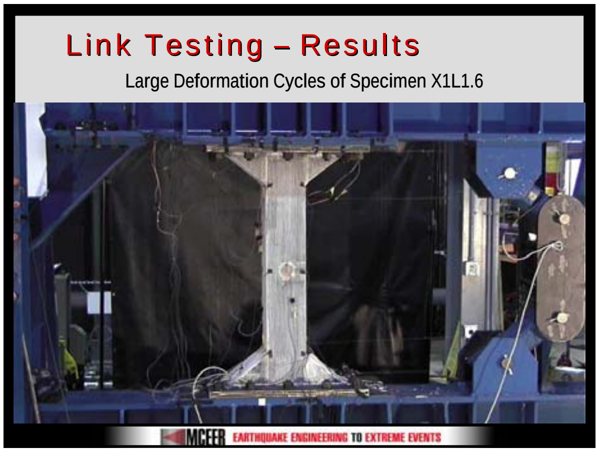


Proof-of-Concept Testing



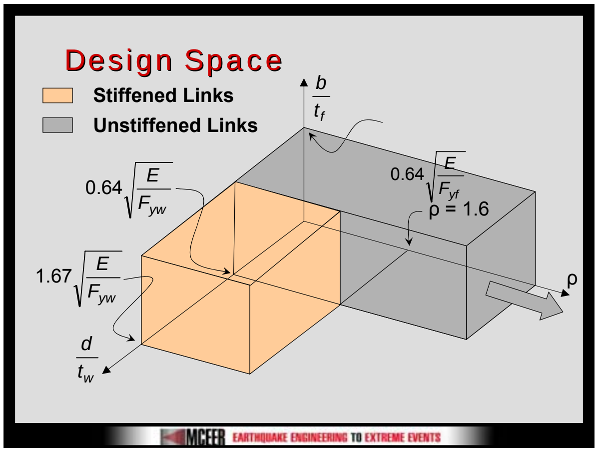
Finite Element Modeling of Proof-of-Concept Testing

Hysteretic Results for Refined ABAQUS Model and Proof-of-Concept Experiment



Link Testing – Results

Large Deformation Cycles of Specimen X1L1.6

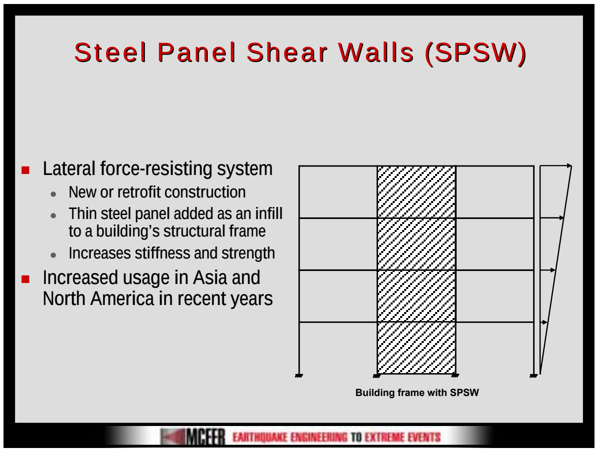


Design Space

🟧 Stiffened Links

⬜ Unstiffened Links

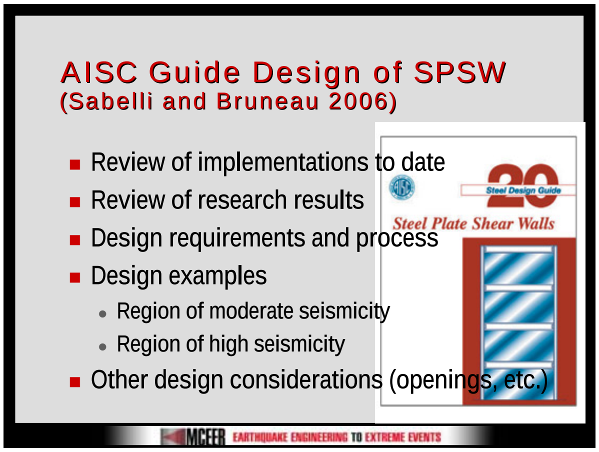
Steel Plate Shear Walls



Steel Panel Shear Walls (SPSW)

- Lateral force-resisting system
  - New or retrofit construction
  - Thin steel panel added as an infill to a building's structural frame
  - Increases stiffness and strength
- Increased usage in Asia and North America in recent years

Building frame with SPSW



AISC Guide Design of SPSW
(Sabelli and Bruneau 2006)

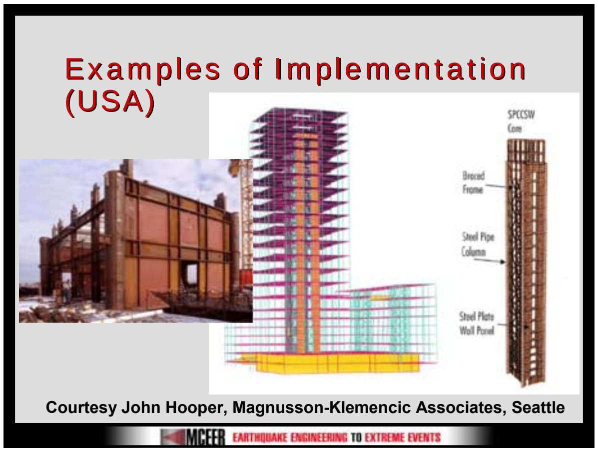
- Review of implementations to date
- Review of research results
- Design requirements and process
- Design examples
  - Region of moderate seismicity
  - Region of high seismicity
- Other design considerations (openings, etc.)

Examples of Implementation (Canada)

Courtesy Louis Crepeau, Groupe Teknika, Montreal, Canada

Examples of Implementation (Mexico)

Courtesy Martinez Romero, Mexico



Examples of Implementation (USA)

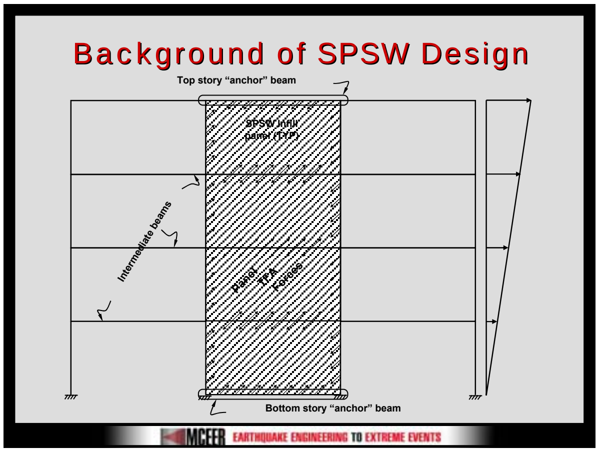
Courtesy John Hooper, Magnusson-Klemencic Associates, Seattle

Examples of Implementation (USA)

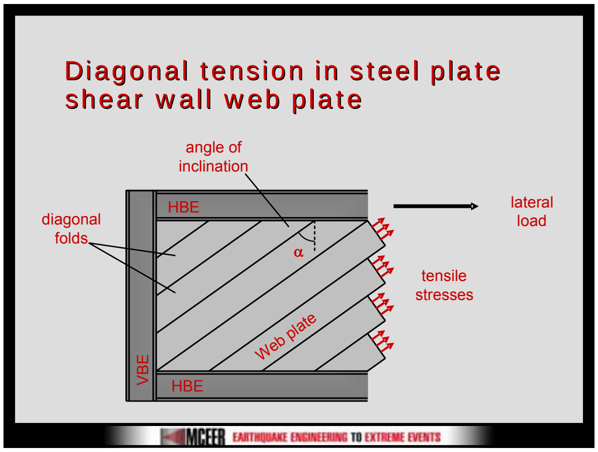
Courtesy Matthew Eatherton – GFDS Engineers San Francisco, CA

Example of Implementation (USA) – Hospital Retrofit

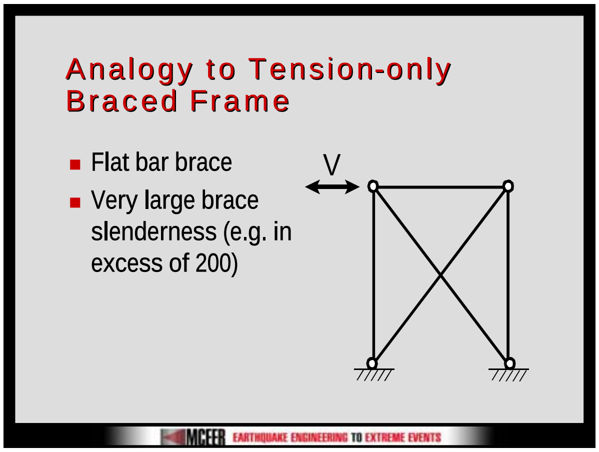
Courtesy Jay Love, Degenkolb Engineers, San Francisco



Background of SPSW Design

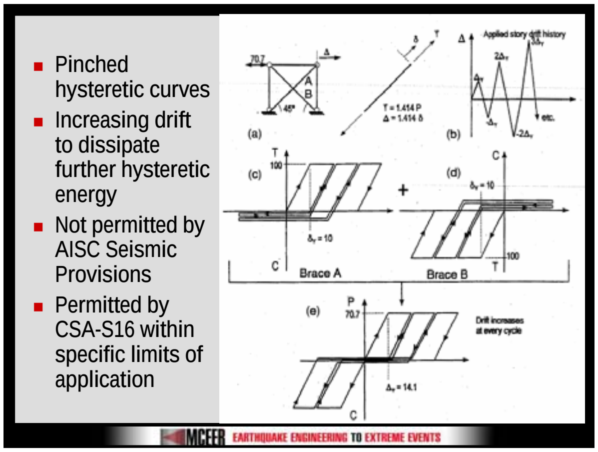


Diagonal tension in steel plate shear wall web plate

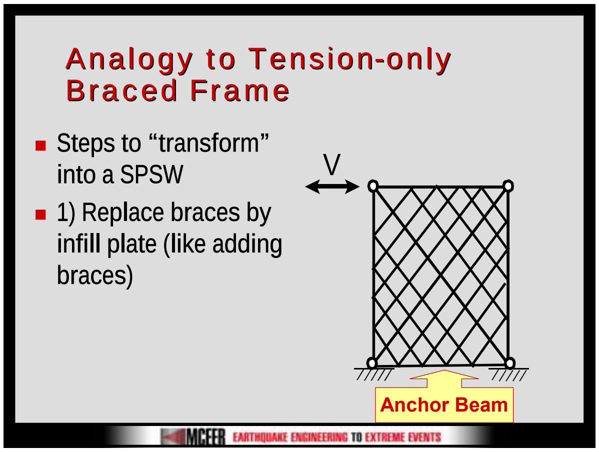


Analogy to Tension-only Braced Frame

- Flat bar brace
- Very large brace slenderness (e.g. in excess of 200)

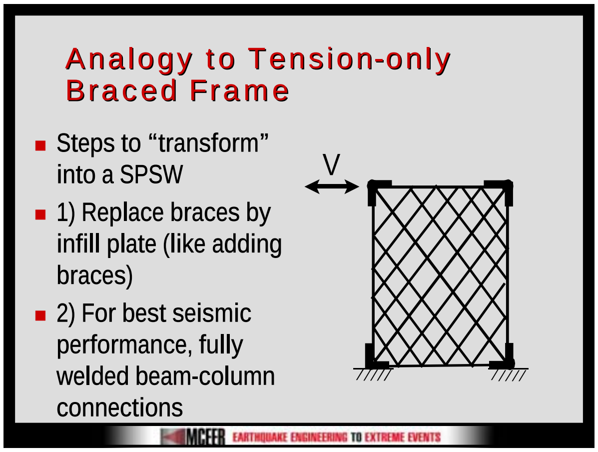


- Pinched hysteretic curves
- Increasing drift to dissipate further hysteretic energy
- Not permitted by AISC Seismic Provisions
- Permitted by CSA-S16 within specific limits of application



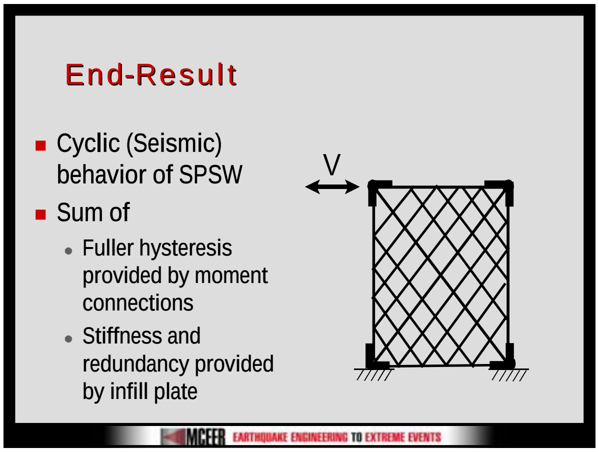
Analogy to Tension-only Braced Frame

- Steps to “transform” into a SPSW
- 1) Replace braces by infill plate (like adding braces)



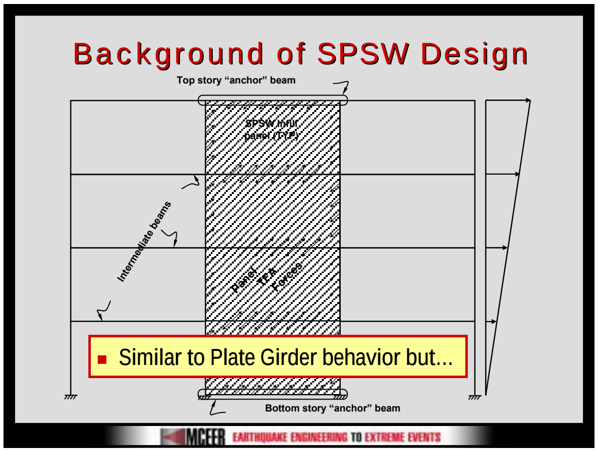
Analogy to Tension-only Braced Frame

- Steps to “transform” into a SPSW
- 1) Replace braces by infill plate (like adding braces)
- 2) For best seismic performance, fully welded beam-column connections



End-Result

- Cyclic (Seismic) behavior of SPSW
- Sum of
  - Fuller hysteresis provided by moment connections
  - Stiffness and redundancy provided by infill plate

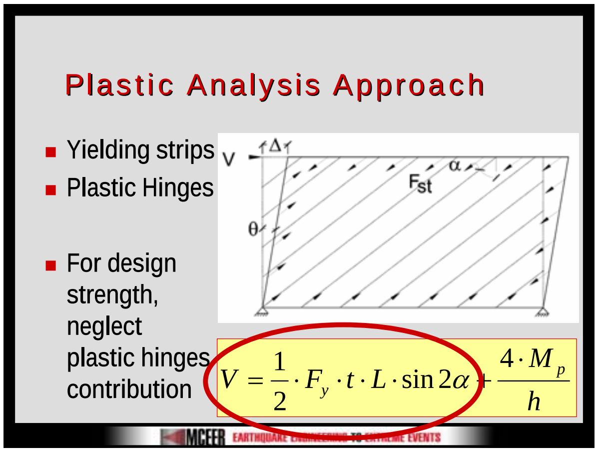


Background of SPSW Design

- Similar to Plate Girder behavior but…

But…
SPSWs are NOT Plate Girders

- Berman, J., Bruneau, M., (2004). "**Steel Plate Shear Walls are not Plate Girders**", AISC Engineering Journal.
- Seismic design provisions specifically developed for SPSW must provide:
  - Design procedure (and, in commentary, modeling guidance) based on
  - Capacity design approach with clear hierarchy of yielding

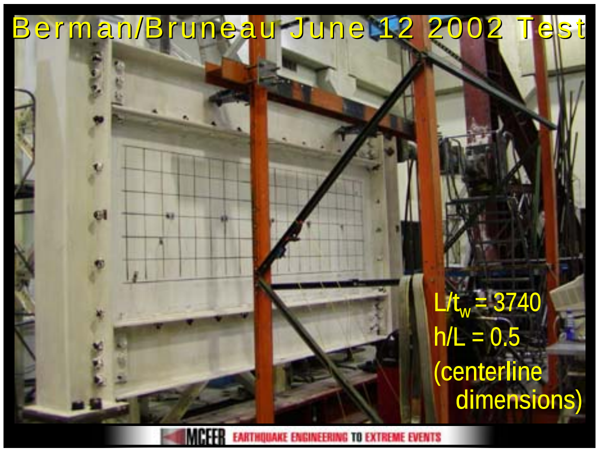


Plastic Analysis Approach

- Yielding strips
- Plastic Hinges

- For design strength, neglect plastic hinges contribution

$$V = \frac{1}{2}\cdot F_y \cdot t \cdot L \cdot \sin 2\alpha + \frac{4 \cdot M_p}{h}$$

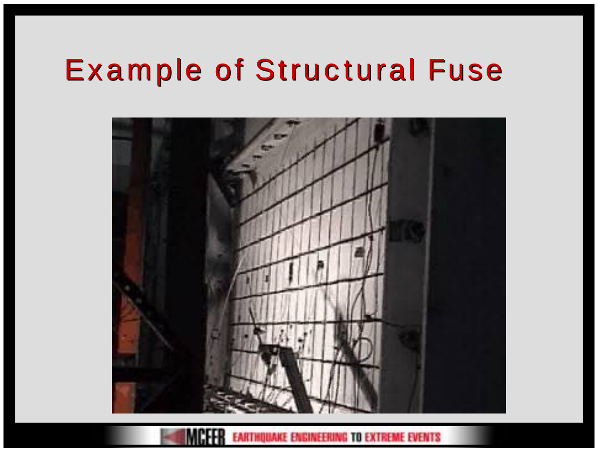


Berman/Bruneau June 12 2002 Test

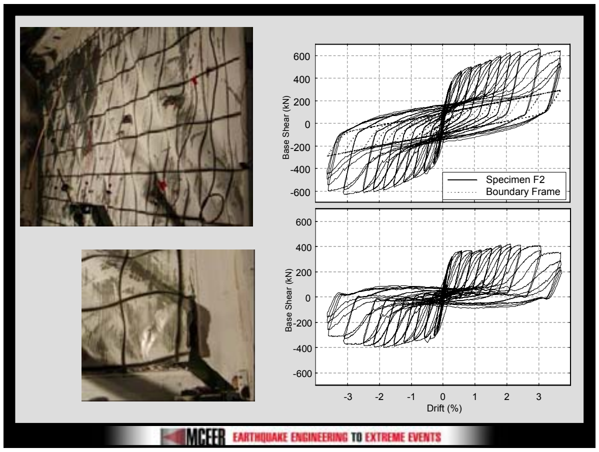
$L/t_w = 3740$

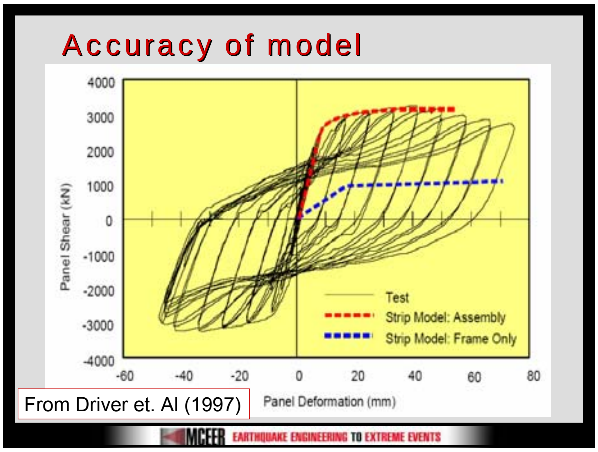
$h/L = 0.5$

(centerline dimensions)



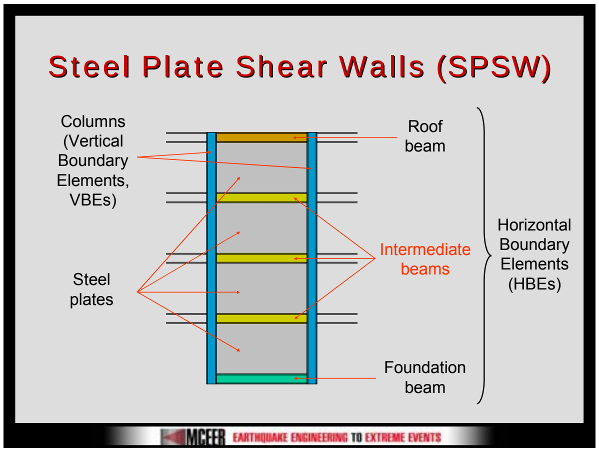
Example of Structural Fuse





Accuracy of model

From Driver et. Al (1997)



Steel Plate Shear Walls (SPSW)

Columns (Vertical Boundary Elements, VBEs)

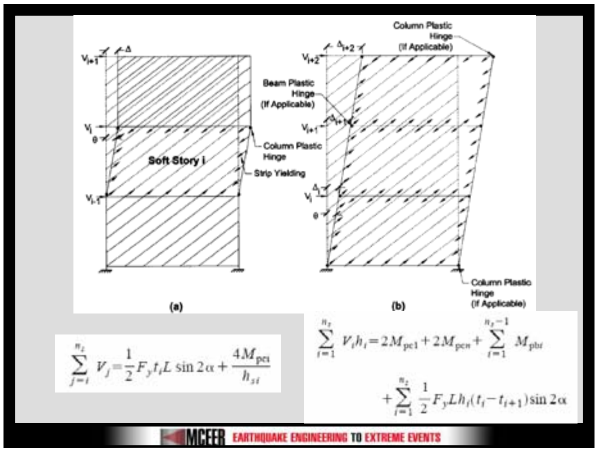
Roof beam

Intermediate beams

Horizontal Boundary Elements (HBEs)

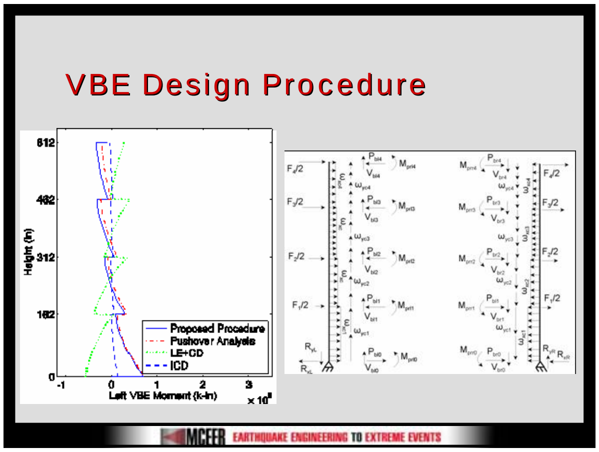
Steel plates

Foundation beam

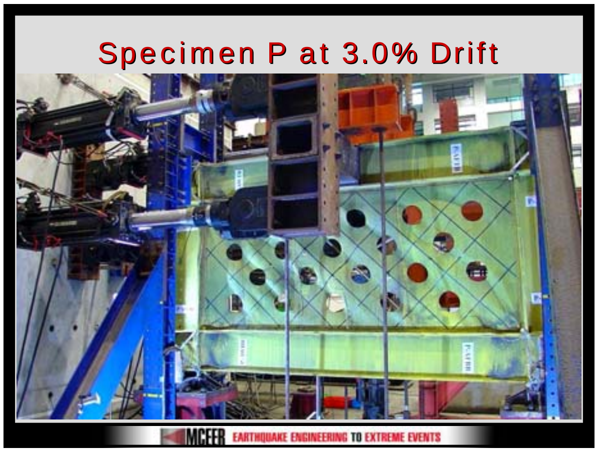


$$\sum_{j=i}^{n_s} V_j = \frac{1}{2} F_y t_i L \sin 2\alpha + \frac{4M_{pci}}{h_{si}}$$

$$\sum_{i=1}^{n_s} V_i h_i = 2M_{pc1} + 2M_{pcn} + \sum_{i=1}^{n_s-1} M_{pbi} + \sum_{i=1}^{n_s} \frac{1}{2} F_y L h_i (t_i - t_{i+1}) \sin 2\alpha$$

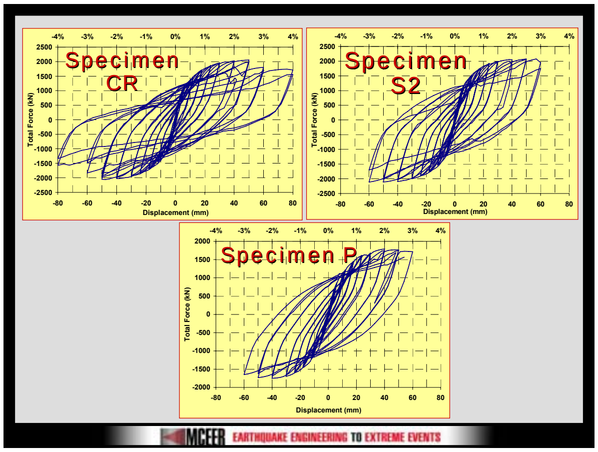


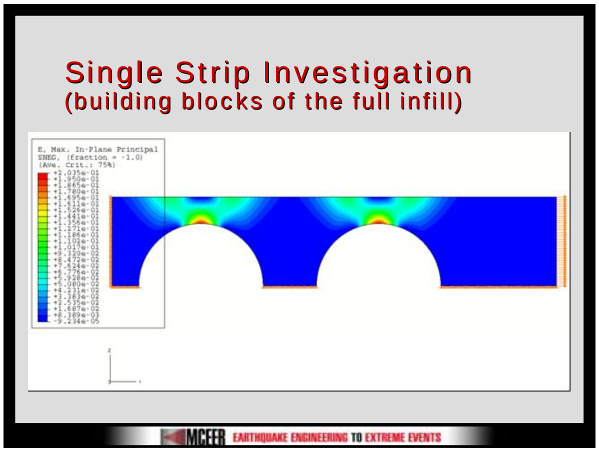
VBE Design Procedure



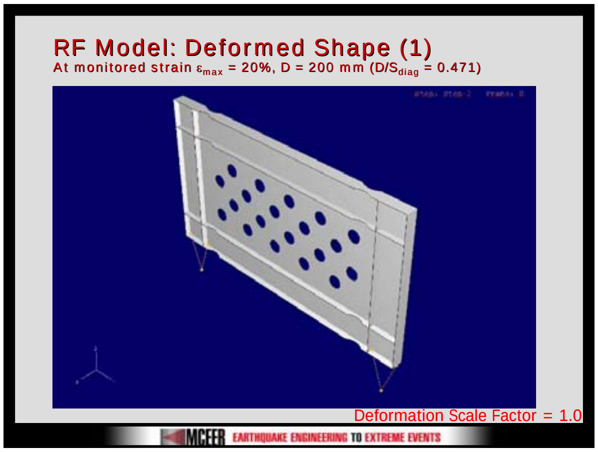
Specimen P at 3.0% Drift

Specimen CR at 4.0% Drift



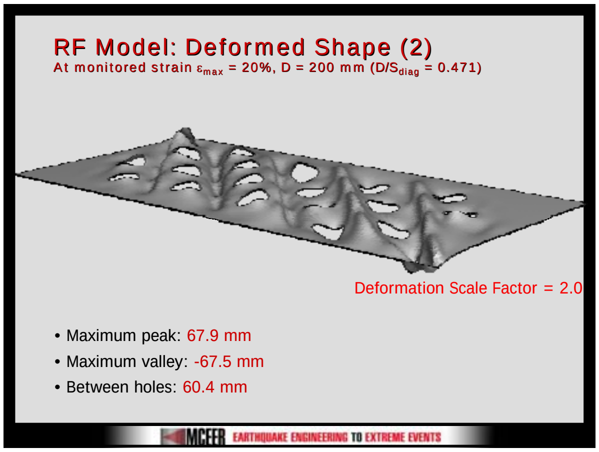


Single Strip Investigation
(building blocks of the full infill)



RF Model: Deformed Shape (1)
At monitored strain ε_{max} = 20%, D = 200 mm (D/S_{diag} = 0.471)

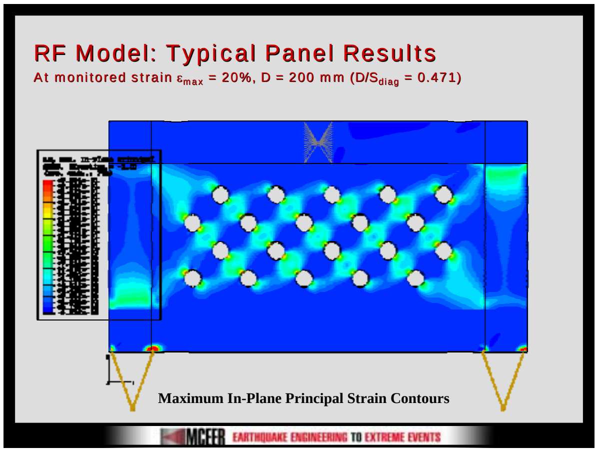
Deformation Scale Factor = 1.0



RF Model: Deformed Shape (2)
At monitored strain ε_{max} = 20%, D = 200 mm (D/S_{diag} = 0.471)

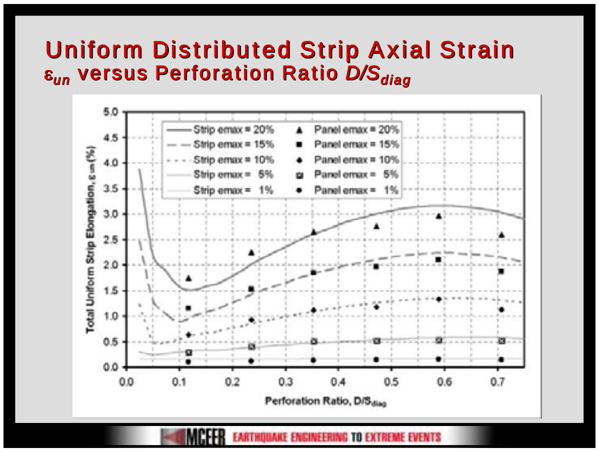
Deformation Scale Factor = 2.0

- Maximum peak: 67.9 mm
- Maximum valley: -67.5 mm
- Between holes: 60.4 mm

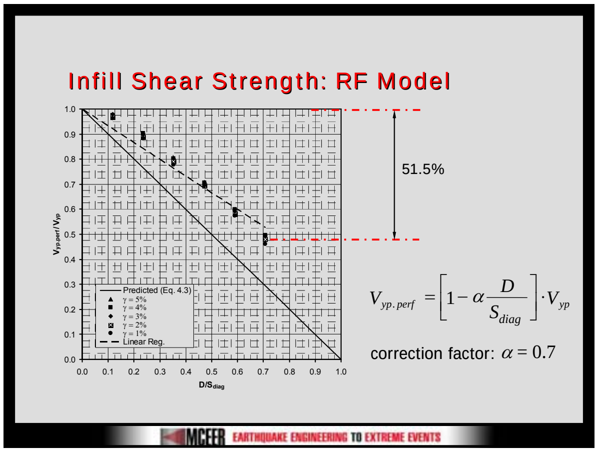


RF Model: Typical Panel Results
At monitored strain ε_{max} = 20%, D = 200 mm (D/S_{diag} = 0.471)

Maximum In-Plane Principal Strain Contours



Uniform Distributed Strip Axial Strain
ε_{un} versus Perforation Ratio D/S_{diag}

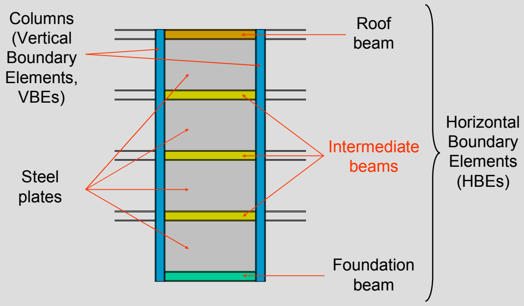


Infill Shear Strength: RF Model

$$V_{yp.perf} = \left[1 - \alpha \frac{D}{S_{diag}}\right] \cdot V_{yp}$$

correction factor: $\alpha = 0.7$

Steel Plate Shear Walls (SPSW)



Specimen before Tests

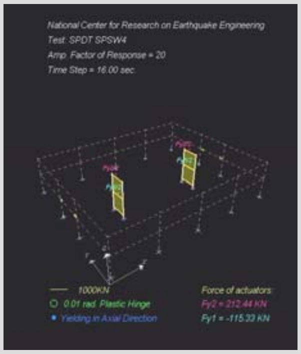
Experimental Program

- Phase I: Pseudo-dynamic load to an earthquake having a 2% in 50 years probability of occurrence. (Chi_Chi_CTU082EW--2／50 PGA=0.67g)
- Cut-out and replace webs at both levels
- Phase II: Repeat of pseudo-dynamic load to an earthquake having a 2% in 50 years probability of occurrence.
- Subsequently cyclic load to failure.

Web replacement

- Buckled web plate from first pseudo-dynamic test cut out and new web plate welded in place

Pseudo-dynamic Test (cont'd)



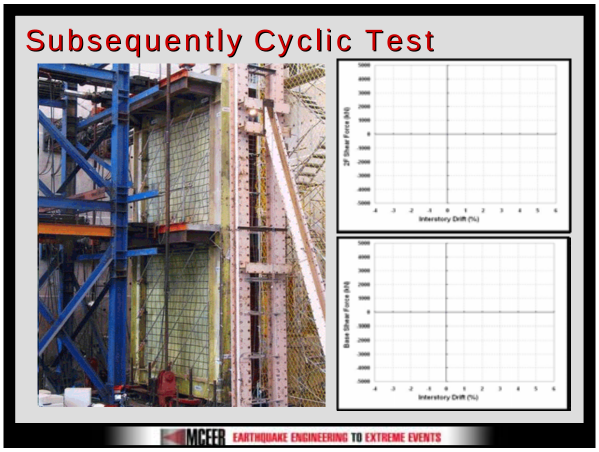
National Center for Research on Earthquake Engineering
Test: SPDT SPSW4
Amp. Factor of Response = 20
Time Step = 16.00 sec.

1000KN
○ 0.01 rad. Plastic Hinge
● Yielding in Axial Direction

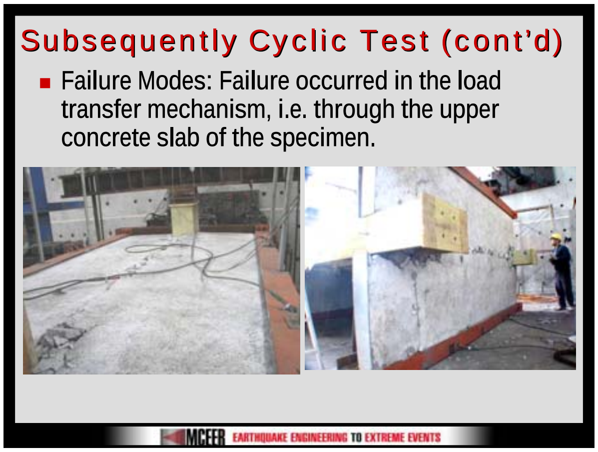
Force of actuators:
Fy2 = 212.44 KN
Fy1 = -115.33 KN

Pseudo-dynamic Test (cont'd)

Specimen after the maximum peak drifts of 2.6% at lower story and 2.3% at upper story in pseudo-dynamic test.

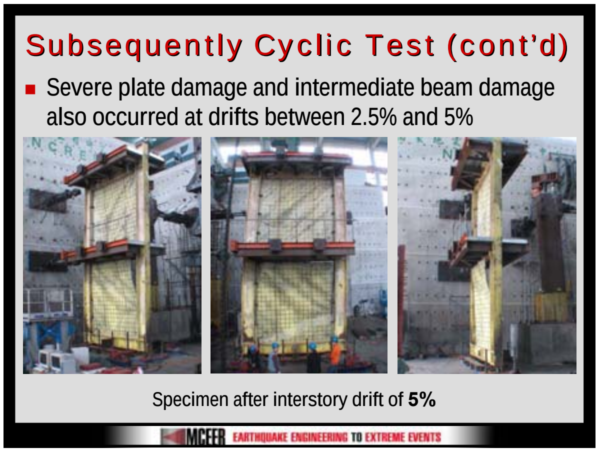


Subsequently Cyclic Test



Subsequently Cyclic Test (cont'd)

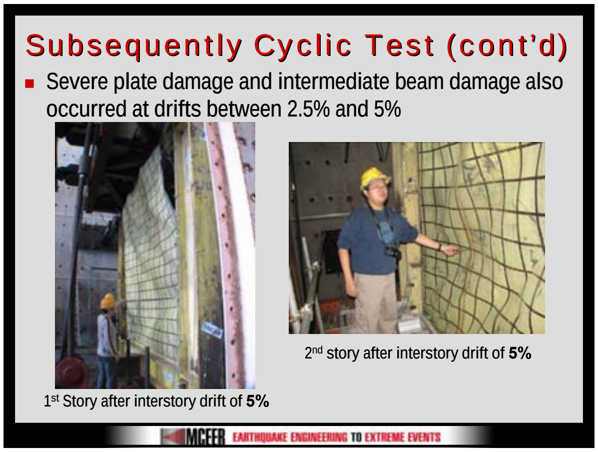
- Failure Modes: Failure occurred in the load transfer mechanism, i.e. through the upper concrete slab of the specimen.



Subsequently Cyclic Test (cont'd)

- Severe plate damage and intermediate beam damage also occurred at drifts between 2.5% and 5%

Specimen after interstory drift of **5%**

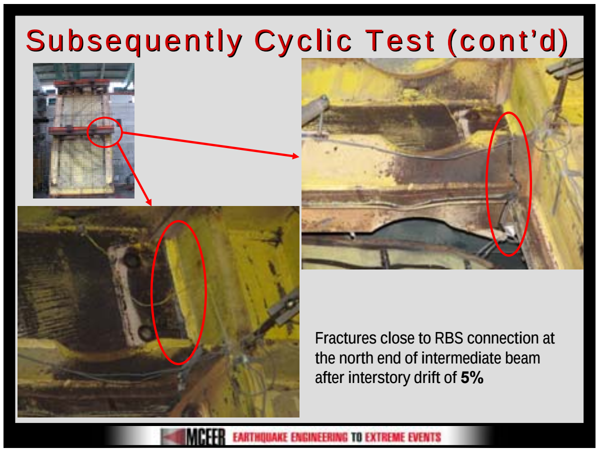


Subsequently Cyclic Test (cont'd)

- Severe plate damage and intermediate beam damage also occurred at drifts between 2.5% and 5%

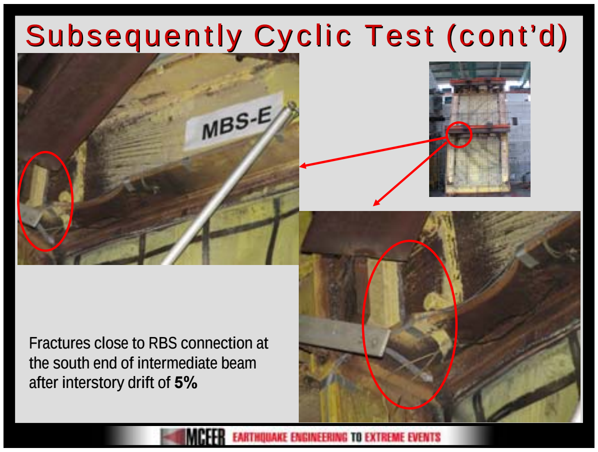
2nd story after interstory drift of **5%**

1st Story after interstory drift of **5%**



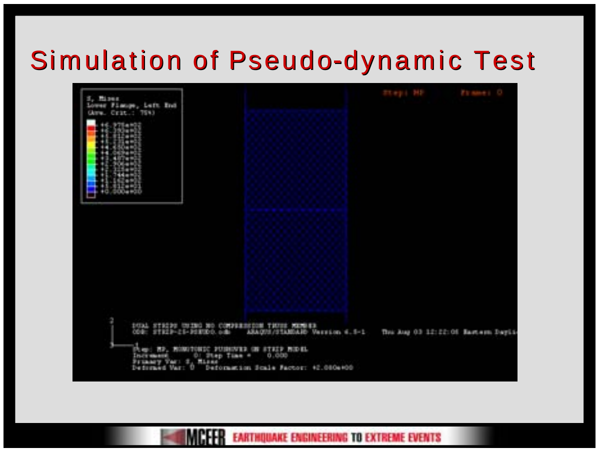
Subsequently Cyclic Test (cont'd)

Fractures close to RBS connection at the north end of intermediate beam after interstory drift of **5%**



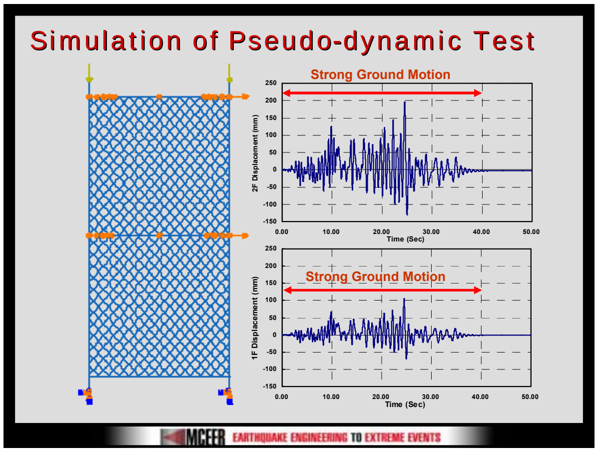
Subsequently Cyclic Test (cont'd)

Fractures close to RBS connection at the south end of intermediate beam after interstory drift of **5%**



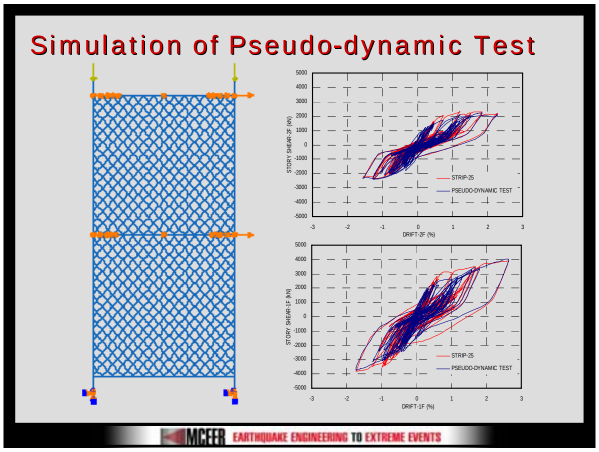
Simulation of Pseudo-dynamic Test

MCEER EARTHQUAKE ENGINEERING TO EXTREME EVENTS



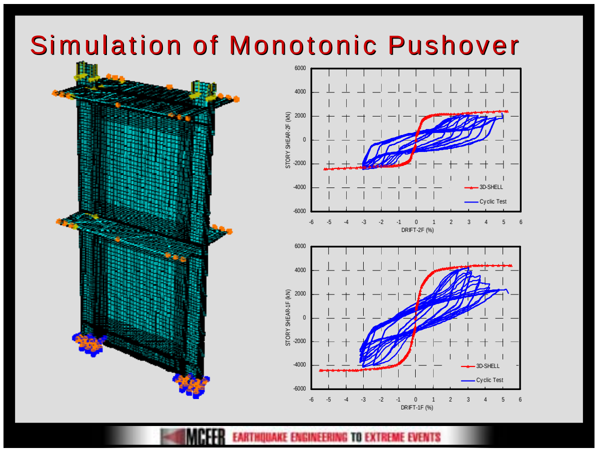
Simulation of Pseudo-dynamic Test

MCEER EARTHQUAKE ENGINEERING TO EXTREME EVENTS



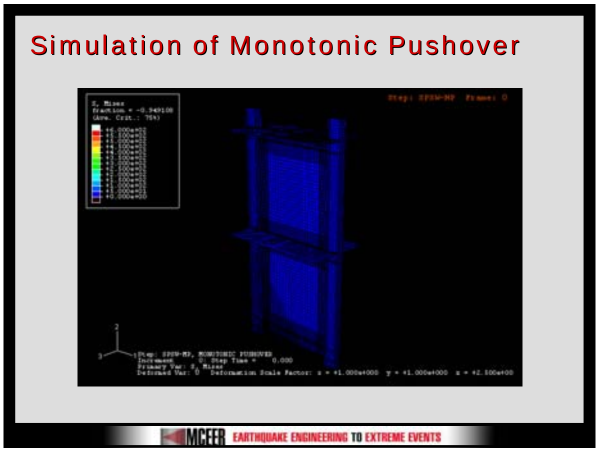
Simulation of Pseudo-dynamic Test

MCEER EARTHQUAKE ENGINEERING TO EXTREME EVENTS



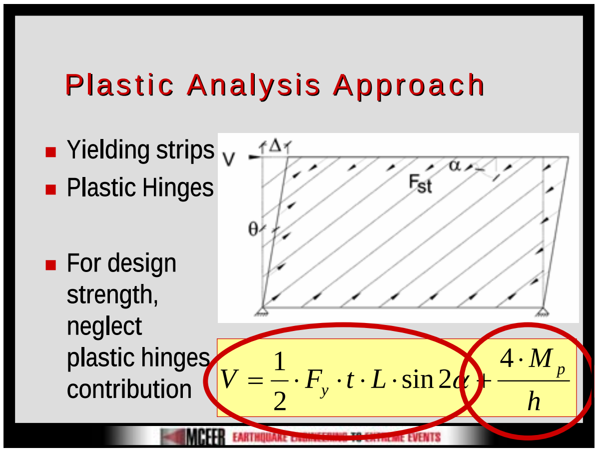
Simulation of Monotonic Pushover

MCEER EARTHQUAKE ENGINEERING TO EXTREME EVENTS



Simulation of Monotonic Pushover

MCEER EARTHQUAKE ENGINEERING TO EXTREME EVENTS

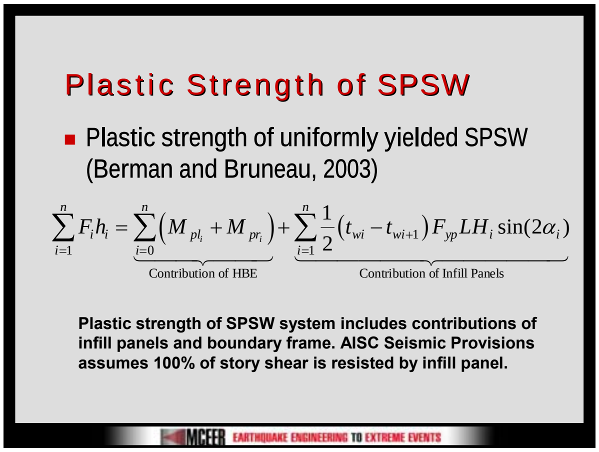


Plastic Analysis Approach

- Yielding strips
- Plastic Hinges
- For design strength, neglect plastic hinges contribution

$$V = \frac{1}{2}\cdot F_y \cdot t \cdot L \cdot \sin 2\alpha + \frac{4\cdot M_p}{h}$$

MCEER EARTHQUAKE ENGINEERING TO EXTREME EVENTS

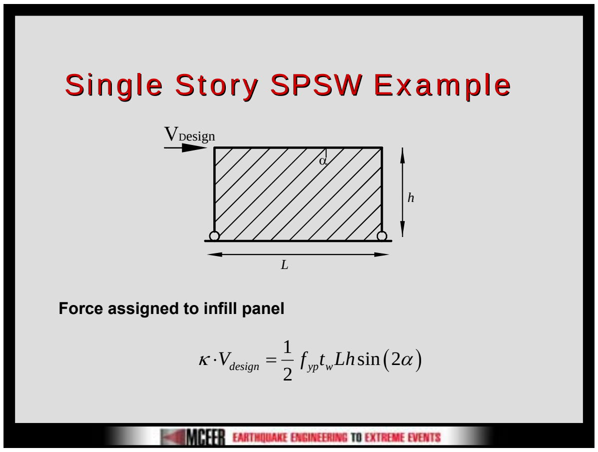


Plastic Strength of SPSW

- Plastic strength of uniformly yielded SPSW (Berman and Bruneau, 2003)

$$\sum_{i=1}^{n} F_i h_i = \underbrace{\sum_{i=0}^{n}\left(M_{pl_i} + M_{pr_i}\right)}_{\text{Contribution of HBE}} + \underbrace{\sum_{i=1}^{n}\frac{1}{2}\left(t_{wi} - t_{wi+1}\right) F_{yp} L H_i \sin(2\alpha_i)}_{\text{Contribution of Infill Panels}}$$

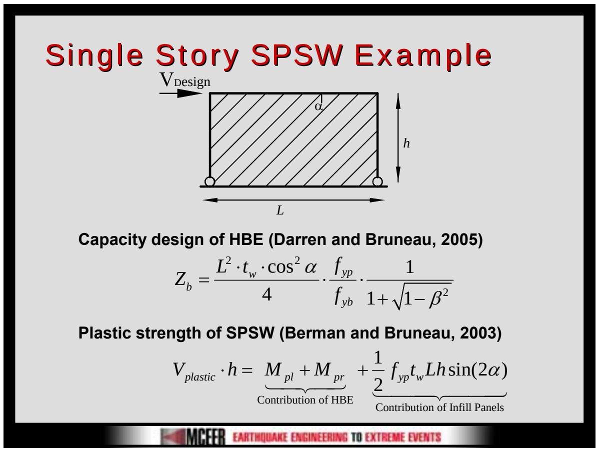
Plastic strength of SPSW system includes contributions of infill panels and boundary frame. AISC Seismic Provisions assumes 100% of story shear is resisted by infill panel.



Single Story SPSW Example

Force assigned to infill panel

$$\kappa \cdot V_{design} = \frac{1}{2} f_{yp} t_w L h \sin\left(2\alpha\right)$$



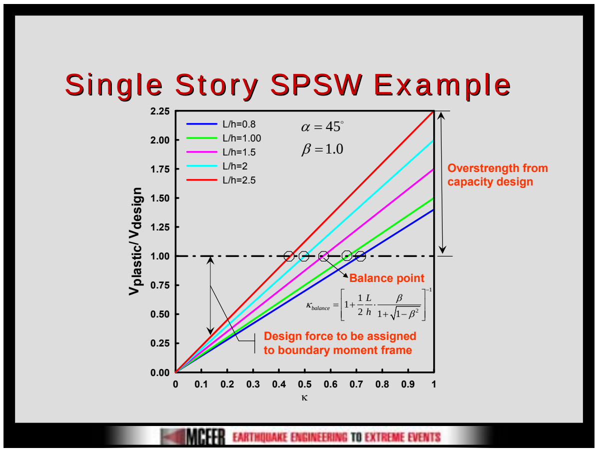
Single Story SPSW Example

Capacity design of HBE (Darren and Bruneau, 2005)

$$Z_b = \frac{L^2 \cdot t_w \cdot \cos^2\alpha}{4} \cdot \frac{f_{yp}}{f_{yb}} \cdot \frac{1}{1+\sqrt{1-\beta^2}}$$

Plastic strength of SPSW (Berman and Bruneau, 2003)

$$V_{plastic} \cdot h = \underbrace{M_{pl} + M_{pr}}_{\text{Contribution of HBE}} + \underbrace{\frac{1}{2} f_{yp} t_w L h \sin(2\alpha)}_{\text{Contribution of Infill Panels}}$$



Single Story SPSW Example

$\alpha = 45^\circ$

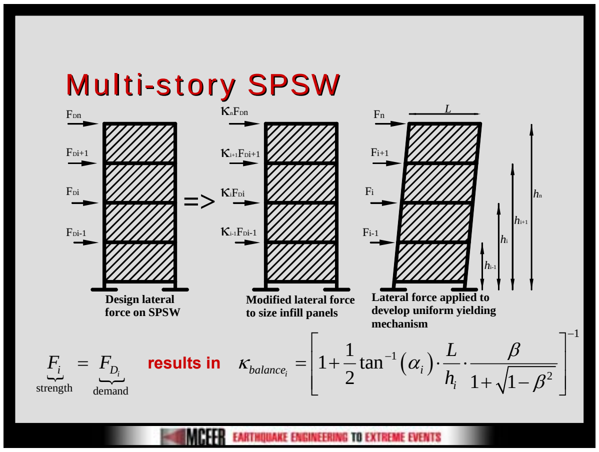
$\beta = 1.0$

Overstrength from capacity design

Balance point

$$\kappa_{balance} = \left[1 + \frac{1}{2}\frac{L}{h} \cdot \frac{\beta}{1+\sqrt{1-\beta^2}}\right]^{-1}$$

Design force to be assigned to boundary moment frame



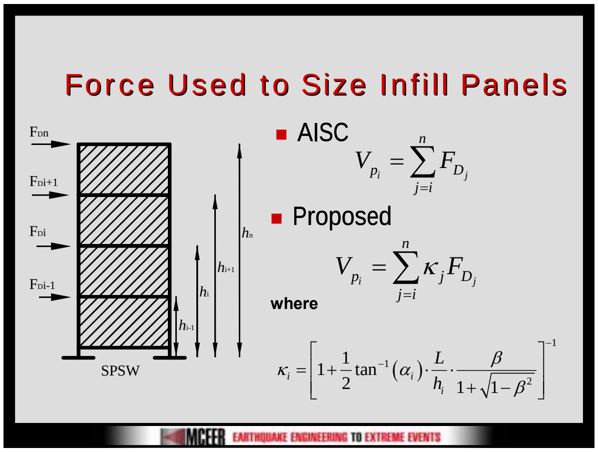
Multi-story SPSW

Design lateral force on SPSW

Modified lateral force to size infill panels

Lateral force applied to develop uniform yielding mechanism

$$\underbrace{F_i}_{\text{strength}} = \underbrace{F_{D_i}}_{\text{demand}}$$ **results in** $$\kappa_{balance_i} = \left[1 + \frac{1}{2}\tan^{-1}\left(\alpha_i\right) \cdot \frac{L}{h_i} \cdot \frac{\beta}{1+\sqrt{1-\beta^2}}\right]^{-1}$$



Force Used to Size Infill Panels

- AISC

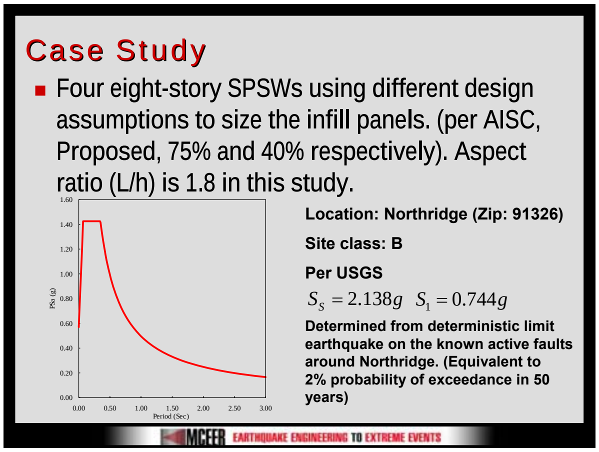
$$V_{p_i} = \sum_{j=i}^{n} F_{D_j}$$

- Proposed

$$V_{p_i} = \sum_{j=i}^{n} \kappa_j F_{D_j}$$

where

$$\kappa_i = \left[1 + \frac{1}{2}\tan^{-1}\left(\alpha_i\right) \cdot \frac{L}{h_i} \cdot \frac{\beta}{1+\sqrt{1-\beta^2}}\right]^{-1}$$



Case Study

- Four eight-story SPSWs using different design assumptions to size the infill panels. (per AISC, Proposed, 75% and 40% respectively). Aspect ratio (L/h) is 1.8 in this study.

Location: Northridge (Zip: 91326)

Site class: B

Per USGS

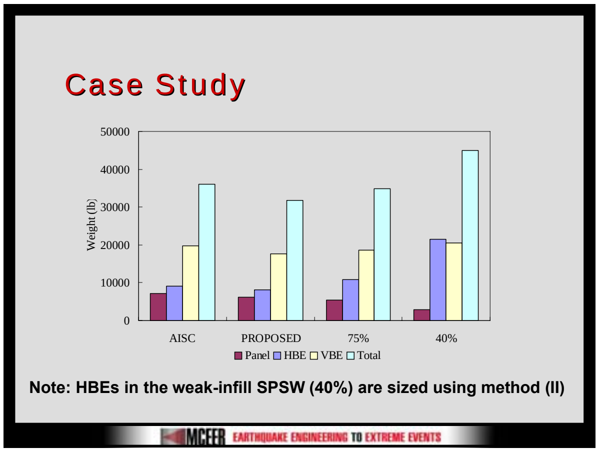
$S_S = 2.138g \quad S_1 = 0.744g$

Determined from deterministic limit earthquake on the known active faults around Northridge. (Equivalent to 2% probability of exceedance in 50 years)

Case Study

- Summary of designed infill panels

Story Level	Elevation (ft)	Lateral Force (kip)	Modified Story Shear (kip)				Infill panel thickness (in)			
			AISC	proposed	75%	40%	AISC	proposed	75%	40%
8	80	127.1	127.1	113.0	95.3	50.8	0.033	0.029	0.025	0.013
7	70	137.9	264.9	233.6	198.7	106.0	0.069	0.060	0.051	0.027
6	60	117.9	382.8	335.5	287.1	153.1	0.099	0.087	0.074	0.040
5	50	97.9	480.7	422.9	360.5	192.3	0.124	0.109	0.093	0.050
4	40	78.0	558.7	492.0	419.0	223.5	0.144	0.127	0.108	0.058
3	30	58.2	616.9	542.8	462.7	246.8	0.160	0.140	0.120	0.064
2	20	38.5	655.5	575.8	491.6	262.2	0.170	0.149	0.127	0.068
1	10	19.0	674.5	590.1	505.9	269.8	0.174	0.153	0.131	0.070

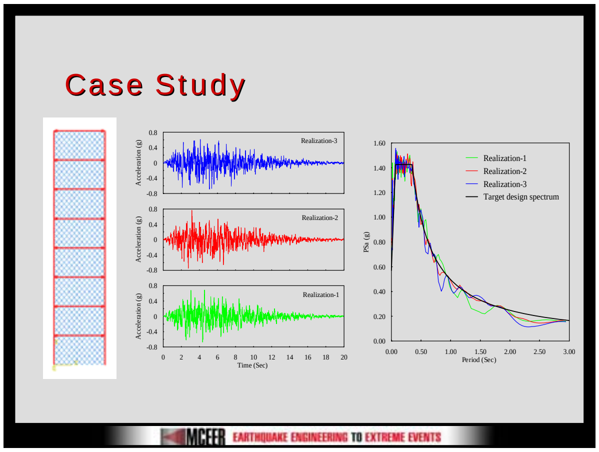


Case Study

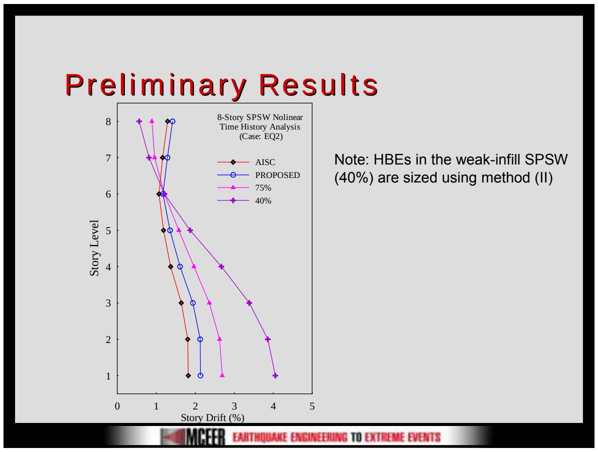
Note: HBEs in the weak-infill SPSW (40%) are sized using method (II)

Case Study

- Nonlinear time history analysis to assess performance of SPSWs using different design assumptions.
- Verified dual strip model (Qu and Bruneau, 2007)
- Target acceleration spectra compatible time histories (Papageorgiou, 2004).

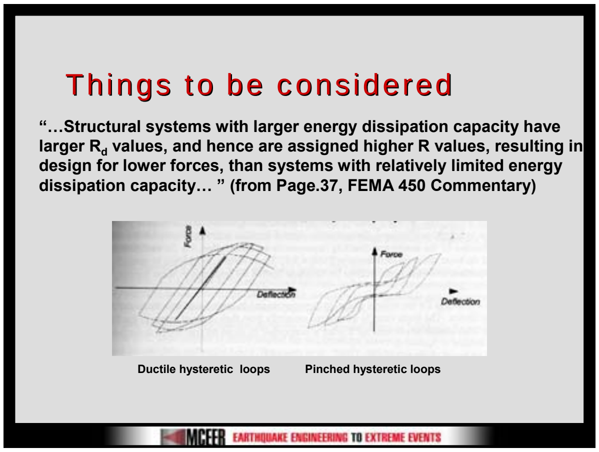


Case Study



Preliminary Results

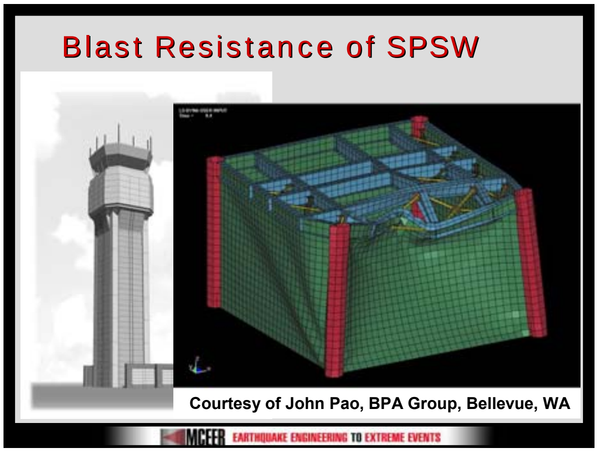
Note: HBEs in the weak-infill SPSW (40%) are sized using method (II)



Things to be considered

"…Structural systems with larger energy dissipation capacity have larger R_d values, and hence are assigned higher R values, resulting in design for lower forces, than systems with relatively limited energy dissipation capacity… " (from Page.37, FEMA 450 Commentary)

Ductile hysteretic loops Pinched hysteretic loops



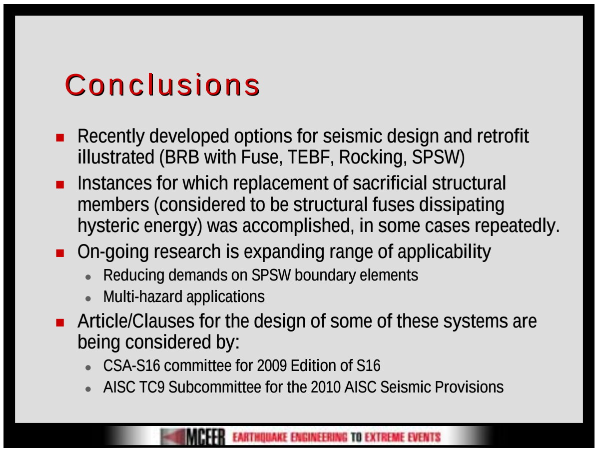
Blast Resistance of SPSW

Courtesy of John Pao, BPA Group, Bellevue, WA

Blast Resistance of SPSW

Blast Resistance of SPSW

Blast Resistance of SPSW



Conclusions

- Recently developed options for seismic design and retrofit illustrated (BRB with Fuse, TEBF, Rocking, SPSW)
- Instances for which replacement of sacrificial structural members (considered to be structural fuses dissipating hysteric energy) was accomplished, in some cases repeatedly.
- On-going research is expanding range of applicability
  - Reducing demands on SPSW boundary elements
  - Multi-hazard applications
- Article/Clauses for the design of some of these systems are being considered by:
  - CSA-S16 committee for 2009 Edition of S16
  - AISC TC9 Subcommittee for the 2010 AISC Seismic Provisions

Thank you!

Questions?