Conventional Braced Frames

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CBFs and Northridge Earthquake

- Conventional braced frames are concentrically braced frames (CBFs)
  - Behavior is dominated by brace buckling, tensile yielding and post buckling inelastic deformation
- Some earthquake damage to CBFs was noted during the Northridge Earthquake
  - Some brace buckling and brace fractures
  - Base plate and anchorage failures
  - Damage was much less wide spread and economically significant than for steel MRFs

CBFs and Northridge Earthquake

- Northridge was a change in direction for SMRFs but was a step forward for CBFs
  - CBF design started to transition to ductile detailing shortly before Northridge
  - CBF designs prior to about 1988 had no ductile detailing requirements
  - CBFs have had increasing usage since Northridge
  - Significant research and improvements in CBF design since Northridge

CBFs Prior to Northridge

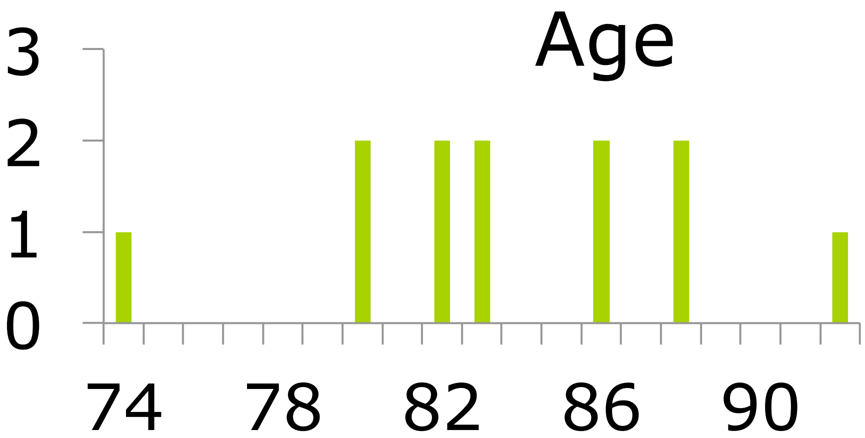
- CBFs historically not well understood
  - Buckling behavior disturbing to engineers
  - Engineers would often assume that the design was relatively simple – not understanding true CBF performance
  - Models for predicting and evaluating CBF behavior less well advanced than for SMRFs
- Today there is a larger inventory of pre-Northridge braced frames with uncertain seismic performance

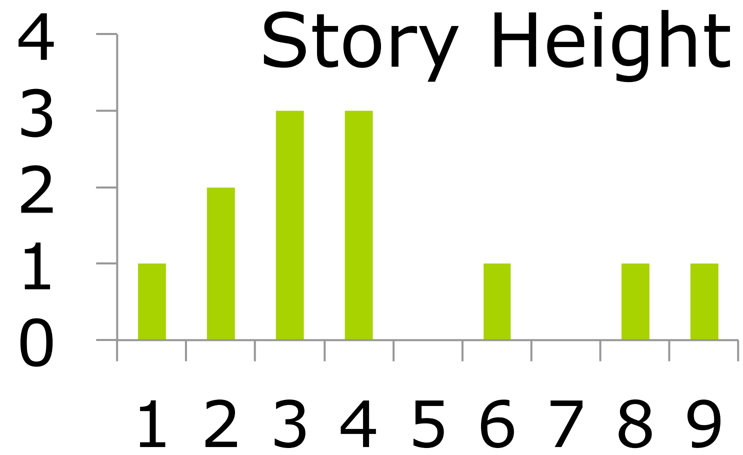
Review of Deficiencies in Older Braced Frames

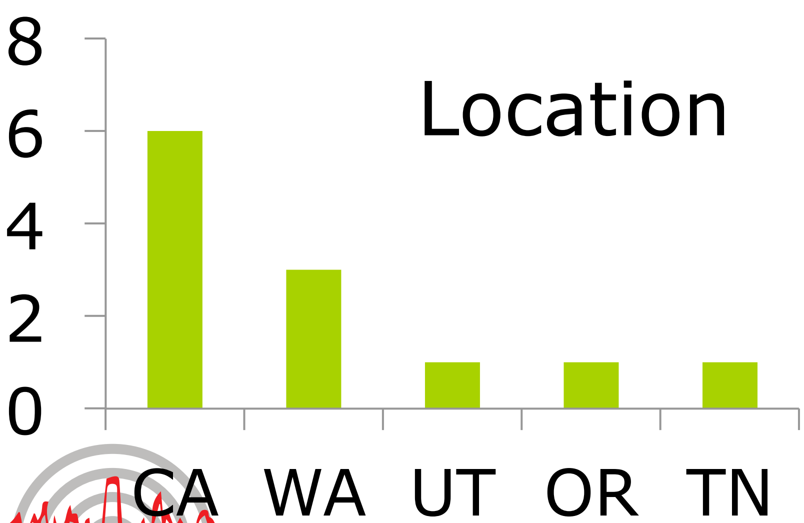
Background

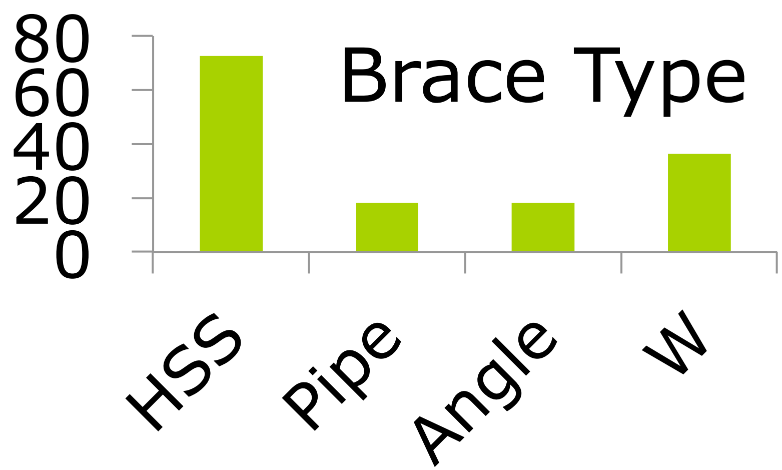
- Drawings obtained from approximately 20 buildings located in seismically active regions throughout the US and deficiencies of these frames noted

Approximate Distribution of Buildings

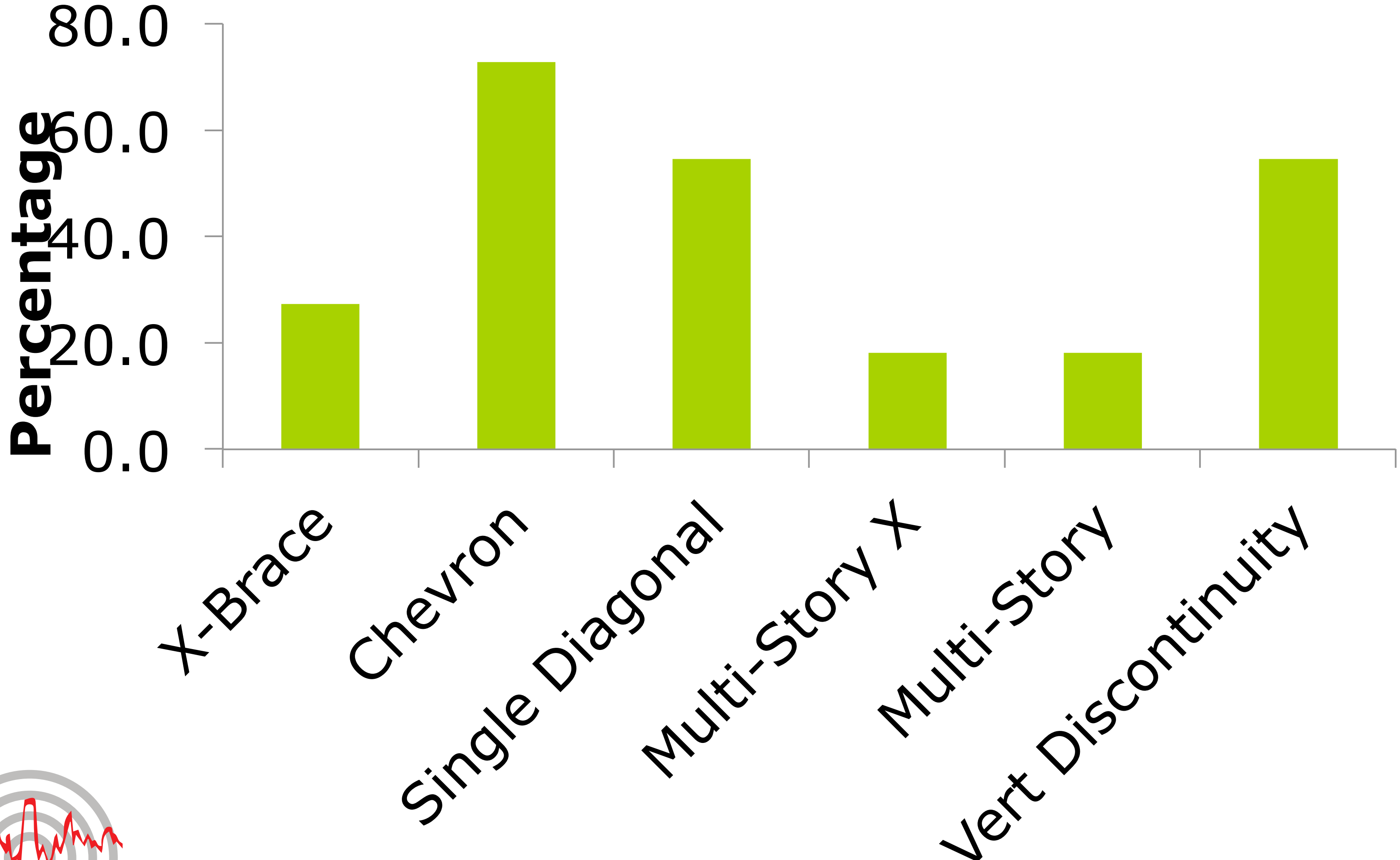








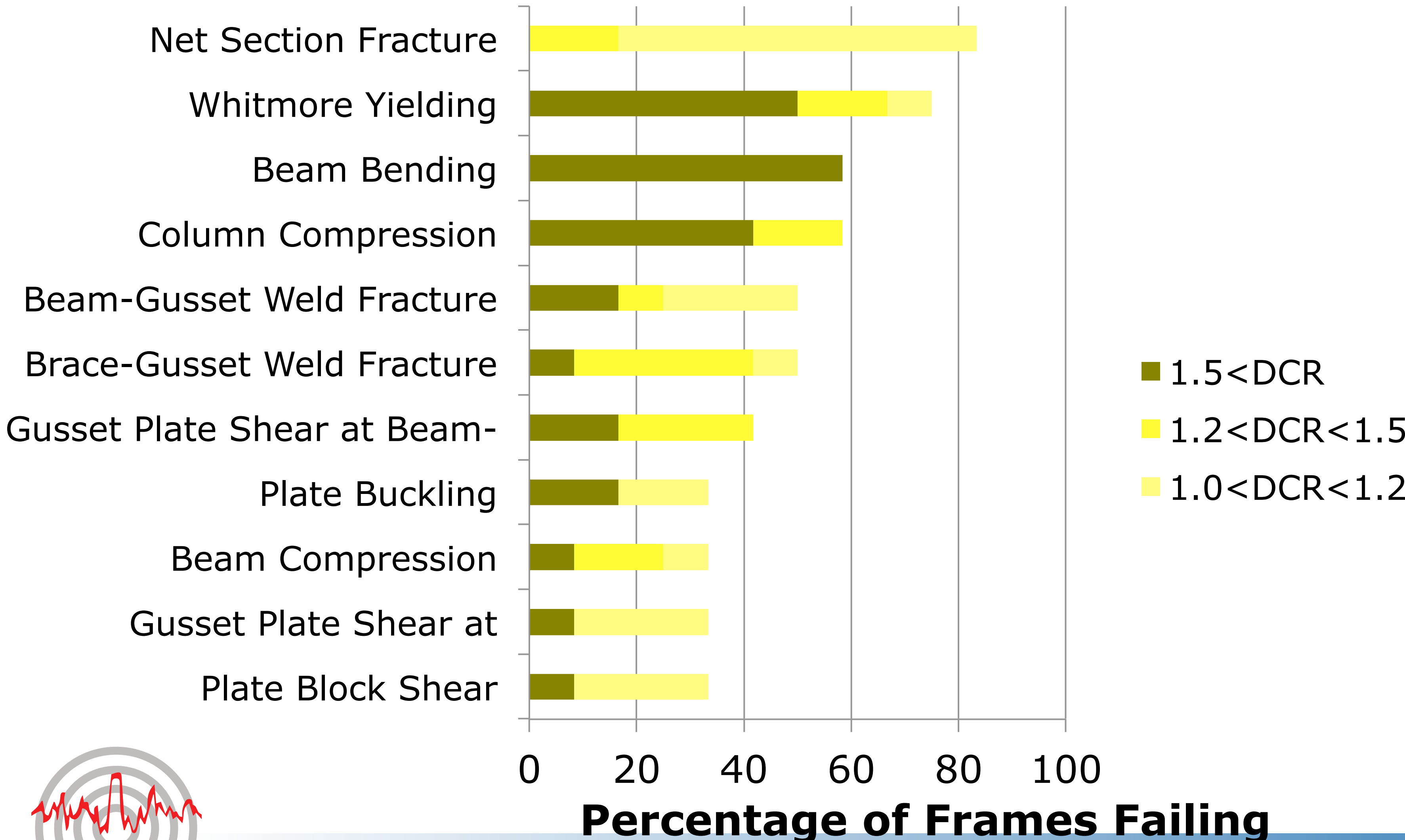
Distribution of Brace Configuration



Failure Modes

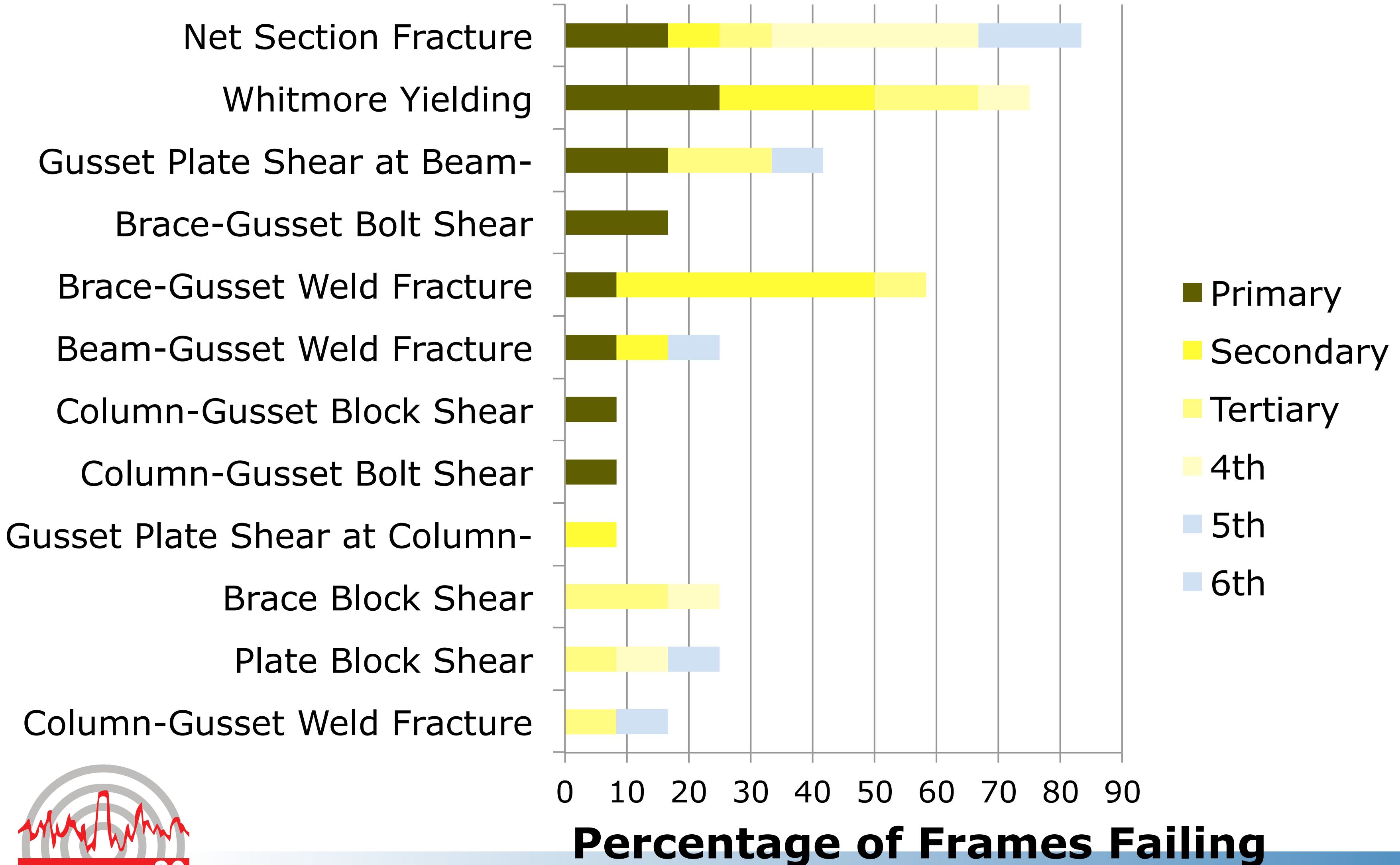
- Connection
  - Brace Net Section
  - Whitmore Yielding
  - Gusset Plate Buckling
  - Block Shear
  - Weld Fracture
  - Base Metal Shear
  - Bolt Shear/Tension
  - Bolt Bearing
- Brace
  - Tensile Yielding
  - Buckling
- Beams
  - Tension
  - Compression + Bending
- Columns
  - Tension
  - Buckling

Results



Net Section Fracture
Whitmore Yielding
Beam Bending
Column Compression
Beam-Gusset Weld Fracture
Brace-Gusset Weld Fracture
Gusset Plate Shear at Beam-
Plate Buckling
Beam Compression
Gusset Plate Shear at
Plate Block Shear
0
20
40
60
80
100
Percentage of Frames Failing
1.5<DCR
1.2<DCR<1.5
1.0<DCR<1.2

Results



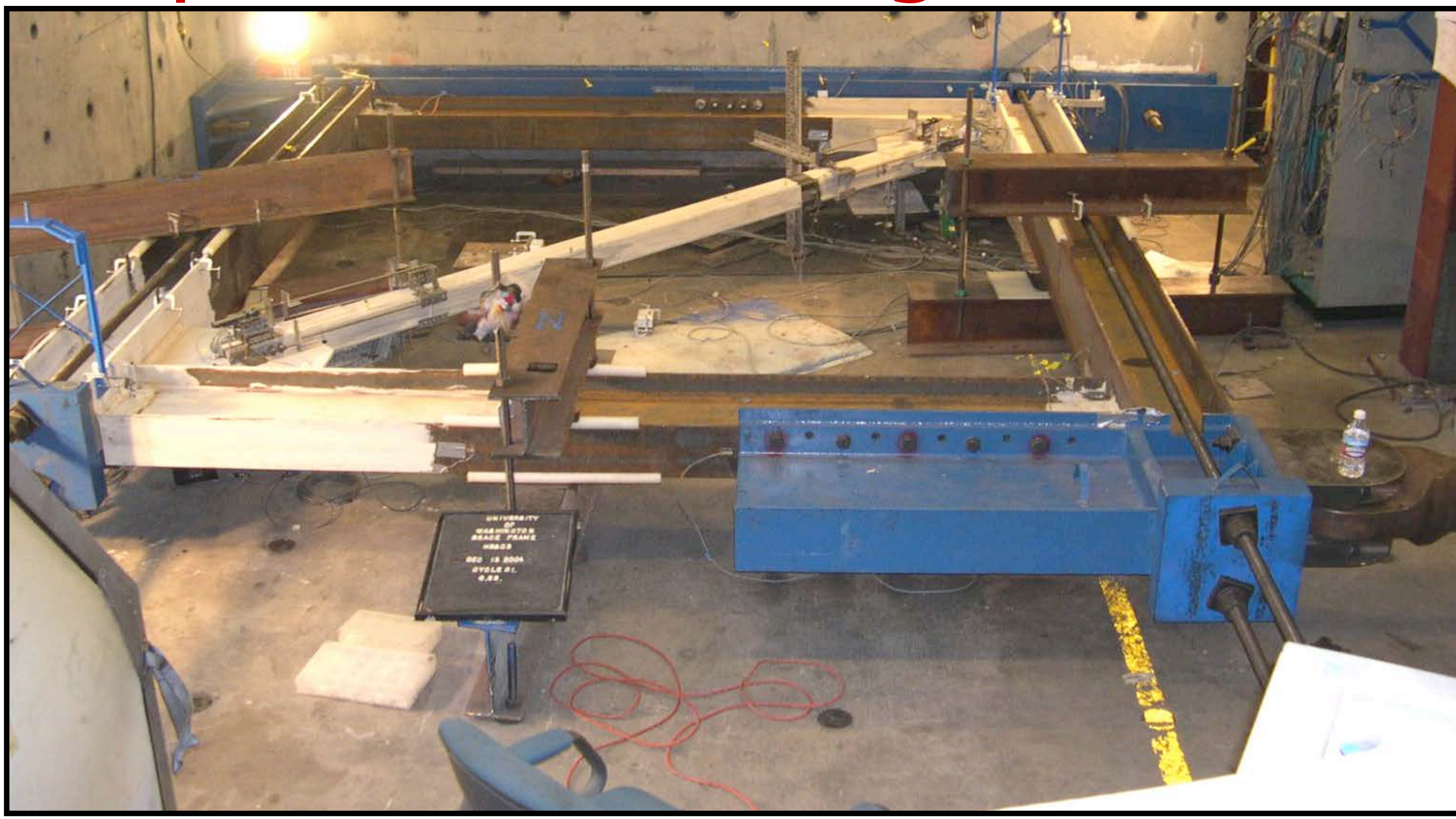
Net Section Fracture
Whitmore Yielding
Gusset Plate Shear at Beam-
Brace-Gusset Bolt Shear
Brace-Gusset Weld Fracture
Beam-Gusset Weld Fracture
Column-Gusset Block Shear
Column-Gusset Bolt Shear
Gusset Plate Shear at Column-
Brace Block Shear
Plate Block Shear
Column-Gusset Weld Fracture
0
10
20
30
40
50
60
70
80
90
Percentage of Frames Failing
Primary
Secondary
Tertiary
4th
5th
6th

Results

- No system met brace end rotation requirements
- All braces met KL/r requirements
- HSS, Chevrons very common
- None of the connections were sufficient to develop their brace capacity
- Few frames could develop the brace capacities
- Brittle failure modes were common

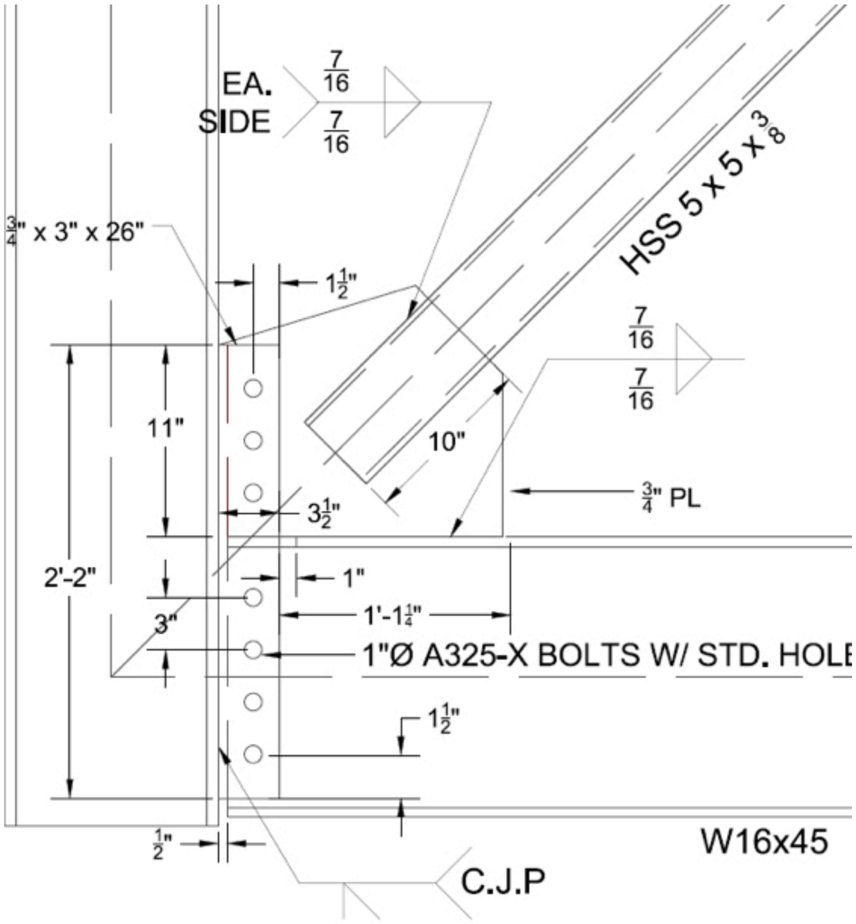
Experimental Research is in progress to evaluate the performance of the older CBFs

UW Experimental Program



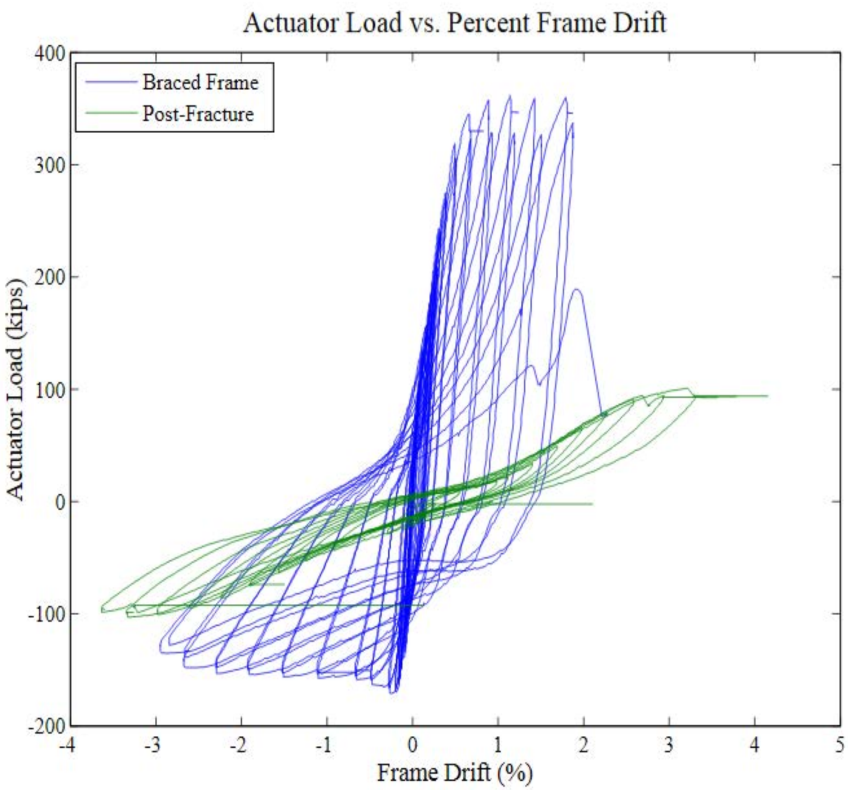
Some perform reasonably well

- Many connection flaws
  - Bolts understrength
  - Welds understrength and not demand critical
  - No end rotation clearance
  - No net section reinforcement



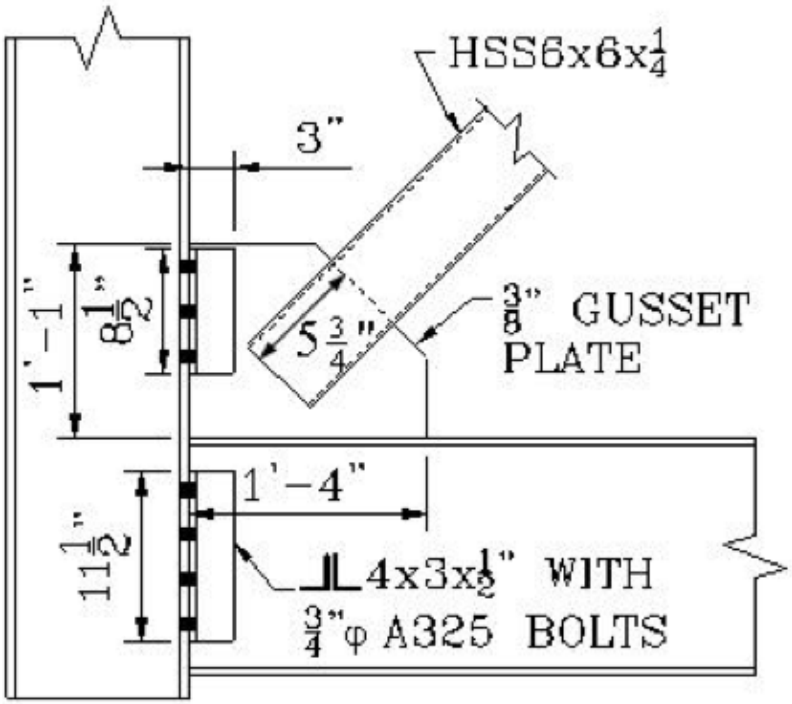
Some Perform reeasonably well

- 4.7% total drift range before brace fracture
- Ultimately had dramatic weld fracture after brace fracture
- Significant bolt hole elongation
- Significant yielding of beams and columns

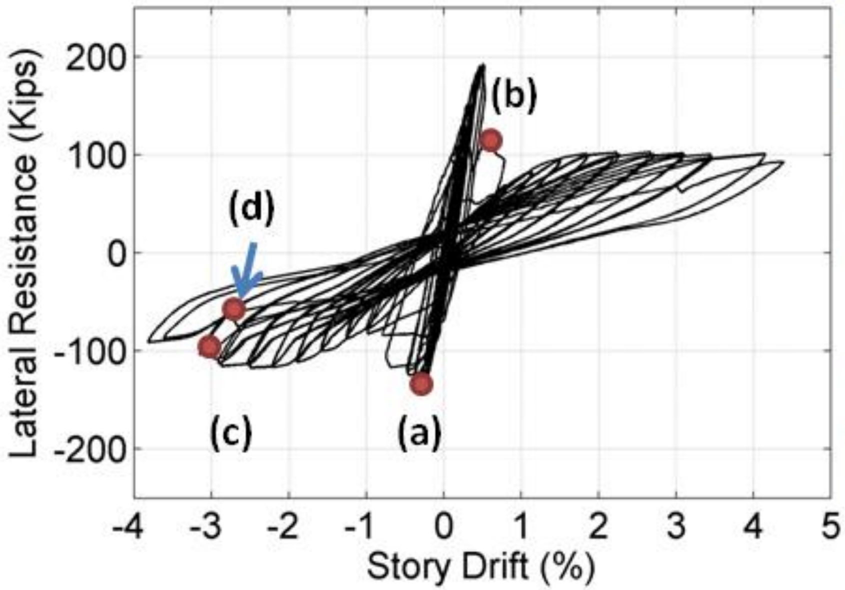


Some have performed poorly

- Bolted clip angle
- Relative to SCBF:
  - Shorter brace-to-gusset length
  - Gusset and associated connections weaker than brace

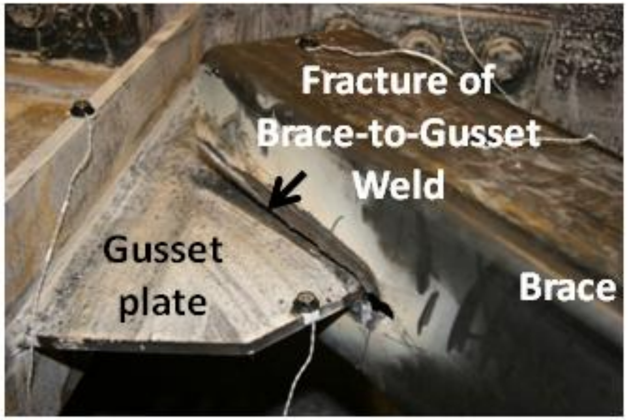


Some have performed poorly



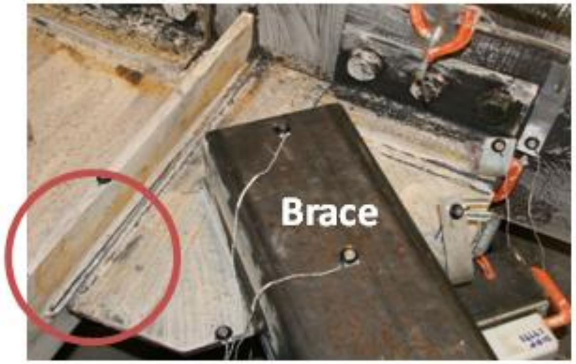
NCBF Braced Frame

(a) Local Buckling



(b) Connection Fracture

(c) Fracture of Gusset Connecting Angle



(d) Weld Failure in Opposing Gusset

This research is in progress and is expected to provide improved understanding of the seismic performance of older braced frames, improved models for evaluating their performance, and retrofit strategies for use in ASCE 41

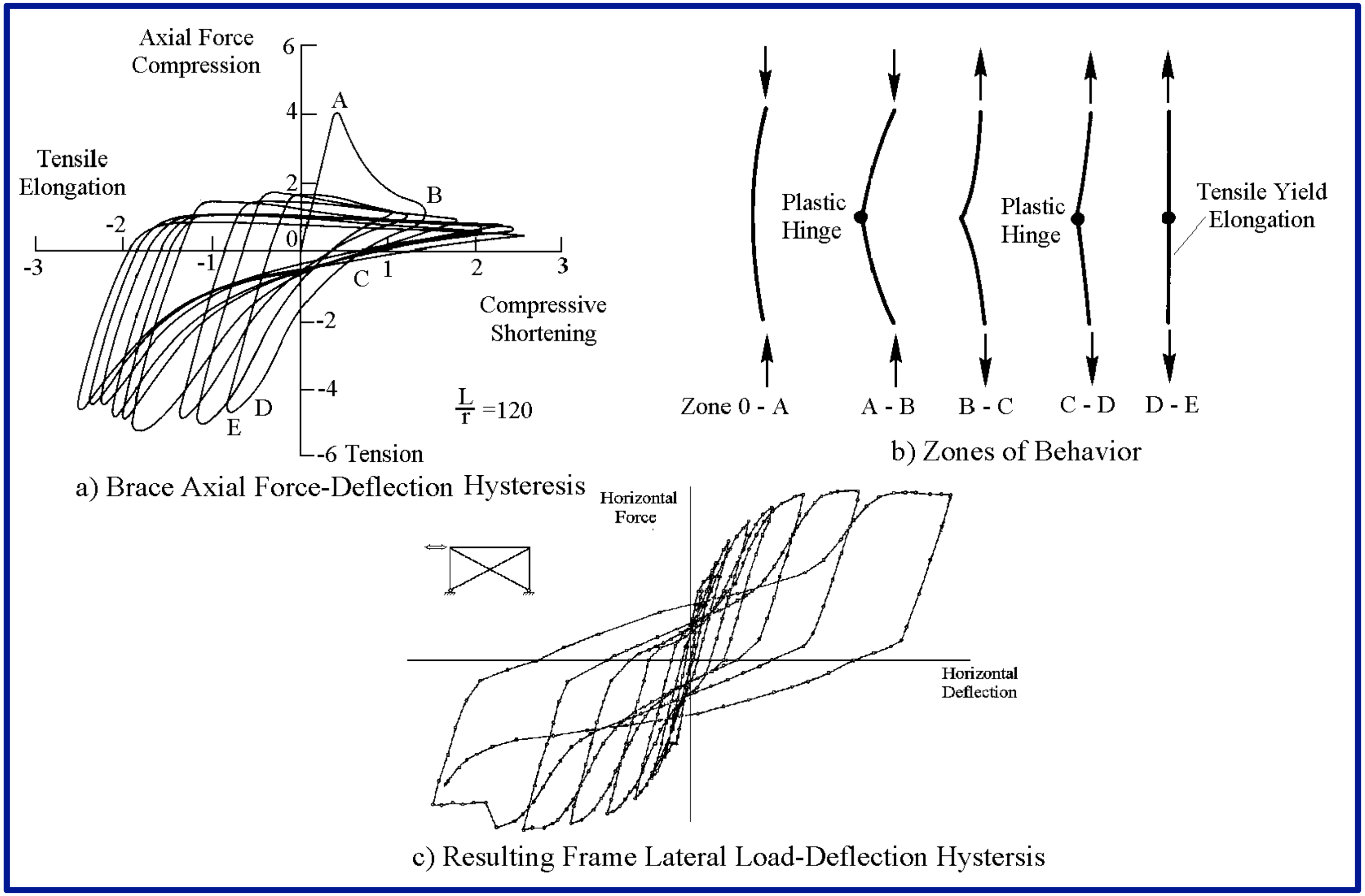
Brief Review of Recent Research and Recommendations on Modern SCBFs

Since Northridge extensive research on modern braced frames has been performed to better understand their performance and develop recommendations for improving their design

SCBF Detailing Requirements

- Limits on global and local slenderness
- Distribution of resistance between braces in tension and compression
- Connections must be designed for the expected capacity of the brace
- Limitations on various bracing configurations
- Requirements for accommodating brace buckling deformation
- Other Requirements

Brace Buckling Dominates Performance



Extensive experimental research has been performed on SCBF systems, and recommendations for improving performance have been developed. This short presentation cannot summarize all of the results, but a few examples are appropriate.

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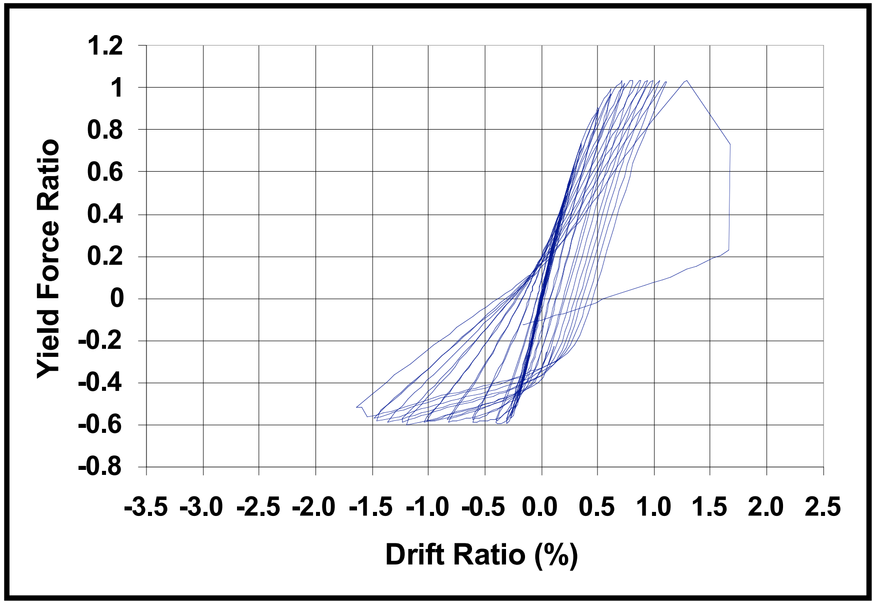
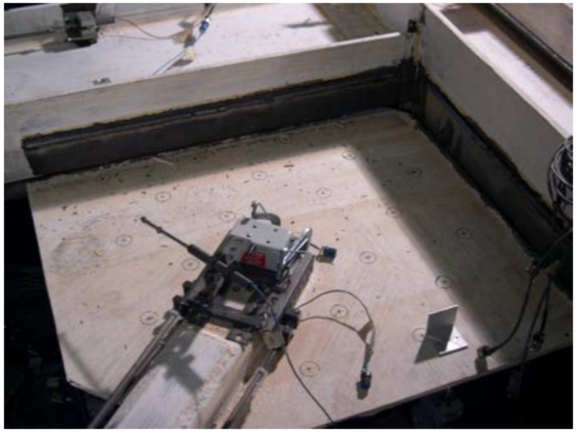
Current Criteria are not Perfect

- AISC SCBF design with 2t linear clearance and UFM
- Brace yielding and buckling occurred
- Gusset plate weld facture limited the performance
  - Welds design by UFM
- Inelastic deformation capacity limited





Bigger is better is wrong

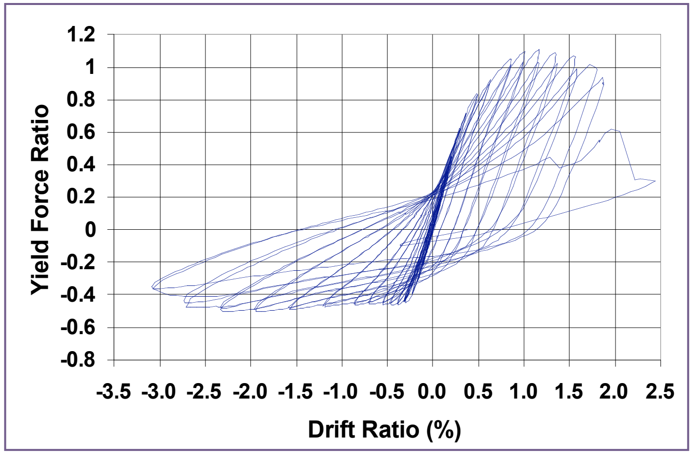
- Same brace and column
- One connection w/ very conservative design and other w/ balance design
- Both had brace Fracture
- Drift Capacities:

3/8" = 3.1% to 1.7% (4.8%)

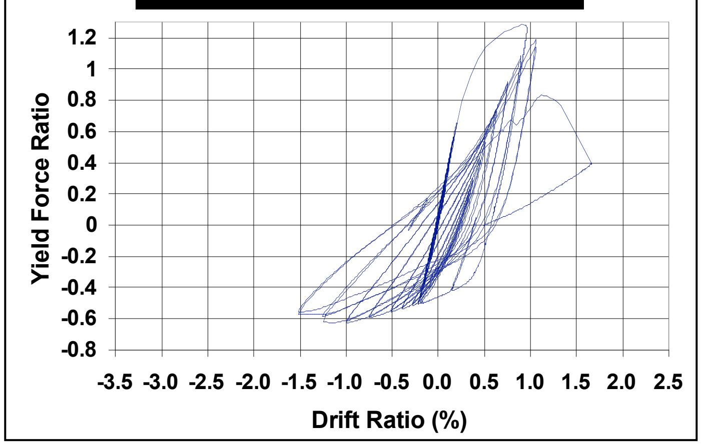
7/8" = -1.5% to 1.0% (2.5%)

- Overly conservative gusset plate connection results in significant reduction in defofmation capacity

3/8 Gusset



7/8 " Gusset



Brace Cross Section Influences Seismic Performance (WF Brace)

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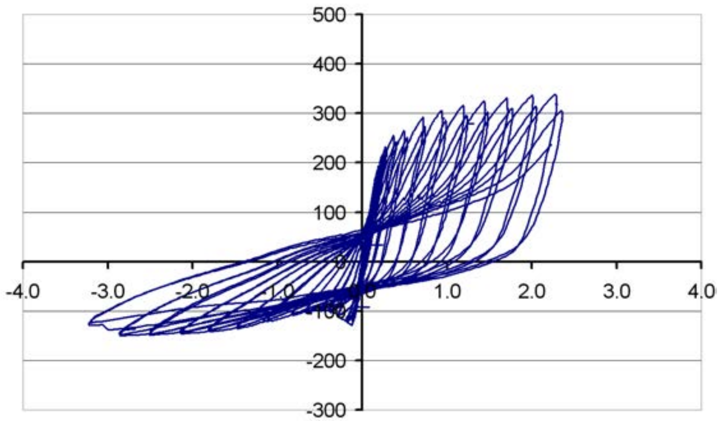
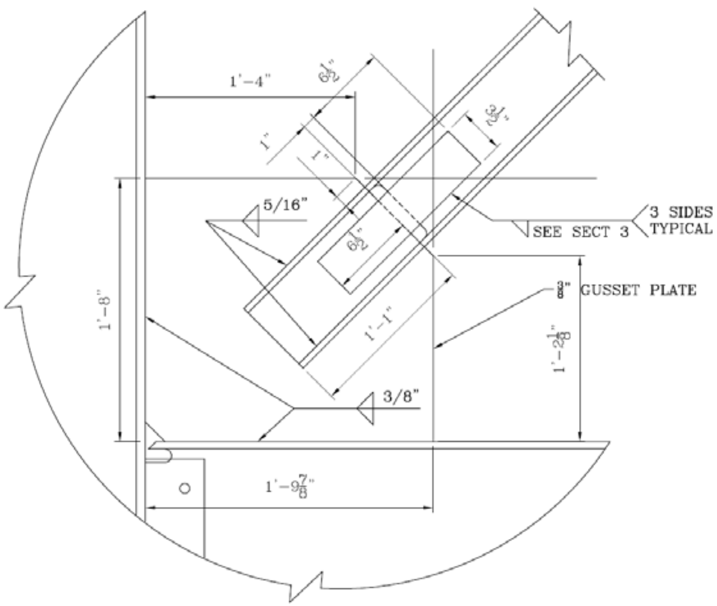
- SCBFs with wide flange braces provide greater inelastic deformation capacity than SCBFs with HSS braces





- Ultimate failure of system due to fracture of gusset adjacent to frame welds and shear failure of bolts shear tab connection
- Drift range between 2.36% and -3.23% (5.59% total)

Widely Distributed Yielding Expected

- Yielding in gusset plate
- Plastic hinging and local buckling in beam and column adjacent to gusset
- Ductile weld tearing
- But large deformation capacity from system if connection properly designed

Primary Research Results: Brace and Gusset Design

- Brace should be designed with a effective length coefficient of 1.0 and the true length of the brace.
- The gusset plate will yield and deform. The gusset plate must be stiff and strong enough to develop the capacity of the brace, but additional capacity and stiffness reduces seismic performance
- Yielding in the gusset plate should be encouraged since it significantly increases the inelastic deformation capacity. Whitmore width yield resistance should be only slightly larger than the yield capacity of the brace

Primary Research Results: Gusset Plate Geometry

- The $8t_p$ elliptical clearance model is recommended for corner gussets and horizontal clearance model for midspan gussets. They provide smaller, thinner gusset plate, increased inelastic deformation capacity of the brace, and reduced yield damage to the beam and column

- Tapered plates achieve similar performance to elliptical model but require thicker plates and place greater demands on bolts or welds. They may improve constructability.

- The edge buckling equations are not appropriate for SCBF gusset design.

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Major Research Results

- Cracking in the gusset near the welds must be expected, but is controlled with demand critical welds. Caused by brace end rotation and gusset deformation. Initiates at about 1.5% to 2% drift.

- The welds or bolts joining the gusset plate to the beam and column should be designed to achieve the yield capacity of the gusset plate <u>not the brace</u>!

- Significant yielding of the beam and column adjacent to the gusset must be expected. Yield damage is reduced with thinner gussets resulting from elliptical model.

- Welded-web welded-flange beam-column connections at the gusset are strongly encouraged.

Proposed
Balanced Design Procedure
(BDP)

Balanced Design Method

1. Design beams, columns and braces for factored design loads as current approach.

2. Establish expected plastic capacity of brace in tension ($R_yA_gF_y$) and compression ($1.1R_yA_gF_{cr}$) as currently. For compression, the effective length of the brace is true brace length.

3. For connection design, propose a balance procedure to assure good seismic performance rather than current forced-based method.

Balancing Yield Mechanisms and Failure Modes

Expected Brace Capacity $< \beta_{yield,1} \mathbf{R_y R_{yield,1}}$ $\ldots\ldots <$ $\beta_{yield,i} \mathbf{R_y R_{yield,i}}$

Expected Brace Capacity $< \beta_{fail,1} \mathbf{R_{fail,1}}$ $< \beta_{fail,2} \mathbf{R_{fail,2}}$ ...

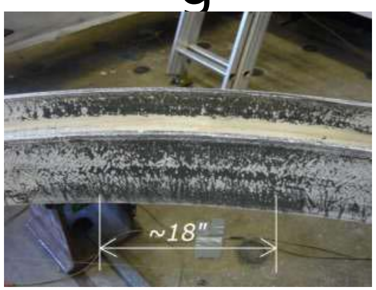
Notes: $\beta_{yield} < \beta_{fail}$

$R_{fail,1}$ is primary failure mode

NEES CBF Research Outcome

Balanced Design Procedure

Brace Buckling → Brace Yielding → Connection Yielding → Beam/Column Yielding → Brace Fracture



- Ensure that adequate ductility capacity is provided
  - Unwanted failure limit states are suppressed

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BDP: Connections

1. Size weld joining the tube for the expected tensile force as with current method
2. Compare the expected tensile yield force of the brace and the tensile fracture capacity of the brace net section using β of 0.95.
3. Using the weld length and brace section dimensions, check block shear of the gusset plate using β of 0.85

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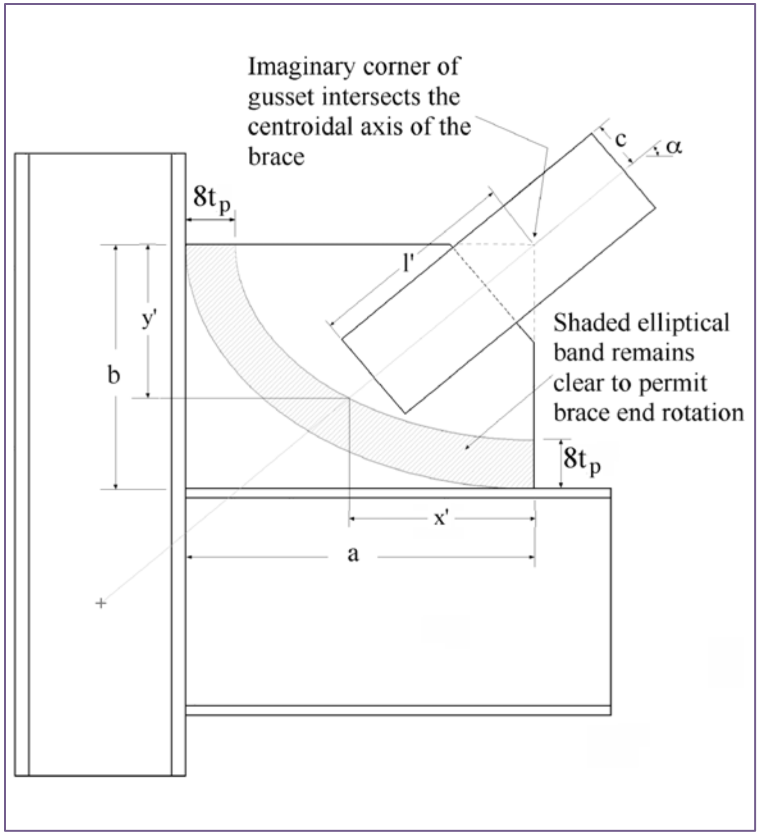
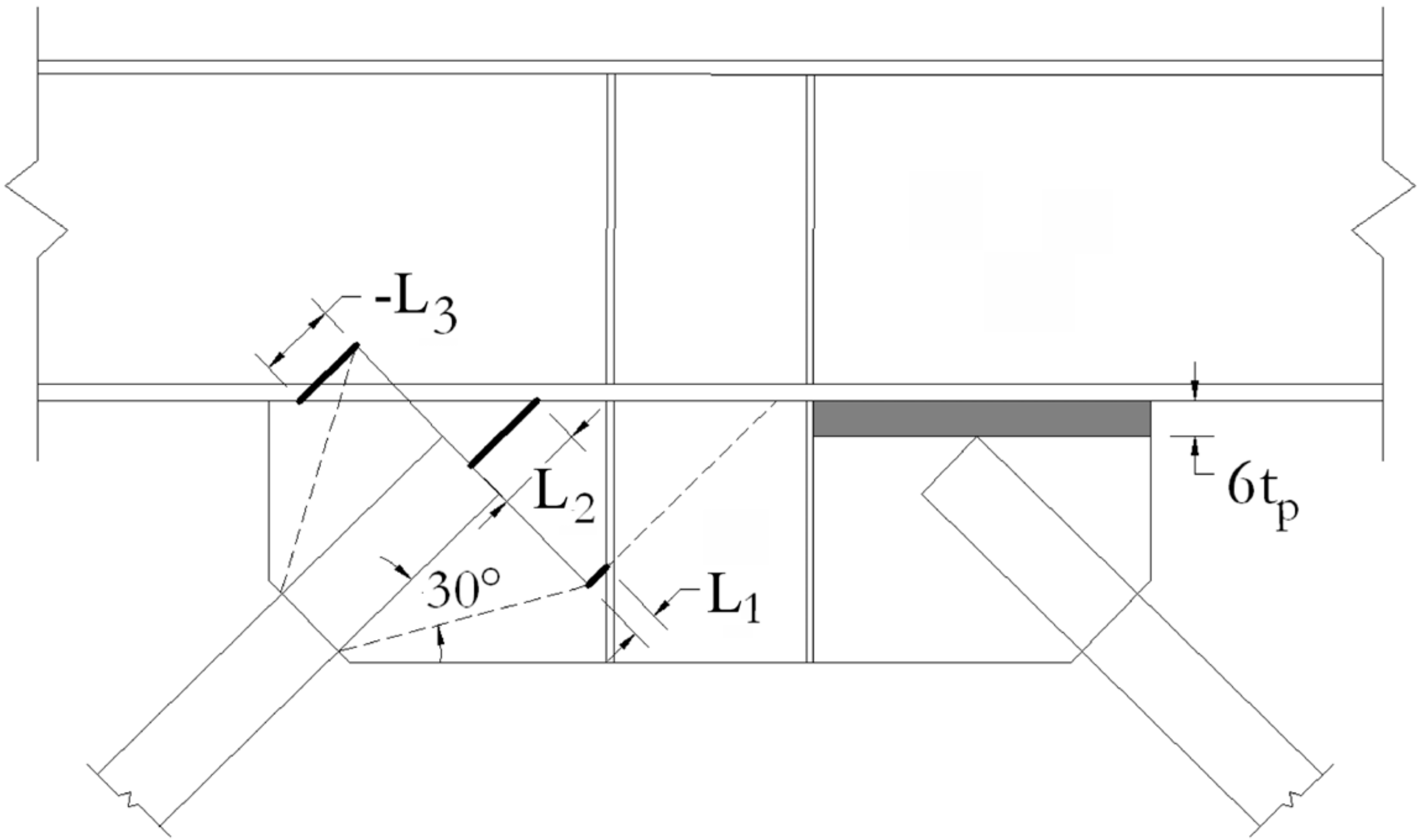
BDP Connections (continued)

4. Establish the Whitmore width by the 30^o projected angle method (as currently used).
5. Establish the dimensions of corner GPs with the $8t_p$ elliptical clearance model. This can be done graphically or by an approximate equation.
6. Establish the dimensions of midspan GPs with 6tp linear (horizontal) clearance.

$8t_p$ Elliptical Clearance for Corner GP



Recommended $6t_p$ Horizontal Clearance Band for Midspan GP



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BDP Connections (cont')

7. Use Whitmore width to check GP for buckling, tensile yield and tensile net section fracture.
   - Use average length with
K=0.65 for corner gussets
K of 1.4 for midspan gussets
   - For tensile yield compare <u>expected</u> tensile yield of GP to the expected tensile capacity of the brace with β=1.0
   - For tensile fracture compare the nominal ultimate tensile capacity of the plate to the expected yield capacity of the brace with a β of 0.85. (Bolts in GP?)
   - Ignore edge buckling

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BDP Connections (cont')

8. Size the welds joining GP to the beam and column to develop the full plastic capacity of GP. CJP welds (or fillet welds on both sides slightly larger than t_p) of matching metal
9. CJP welds to join the beam flanges to the column at beam-column connection

Resulting GP must be stiff and strong enough to support full loads but should have no extra stiffness or resistance

Results of research since Northridge

- Have a basis for improving seimic performance of modern braced frames
- Starting to understand performance of older NCBFs to know:
  - Which systems may be OK
  - How we can evaluate tese older systems
  - Retrofit strategies for economical retrofit of these systems
- There are still concerns but CBFs have improved significantly since Northridge

THANK YOU