COLD-FORMED STEEL RESEARCH CONSORTIUM

NBM TECHNOLOGIES, INC.

Advancing Seismic Provisions for Steel Diaphragms in Rigid Wall-Flexible Diaphragm Buildings

5 April 2018

Project Update to BSSC PUC

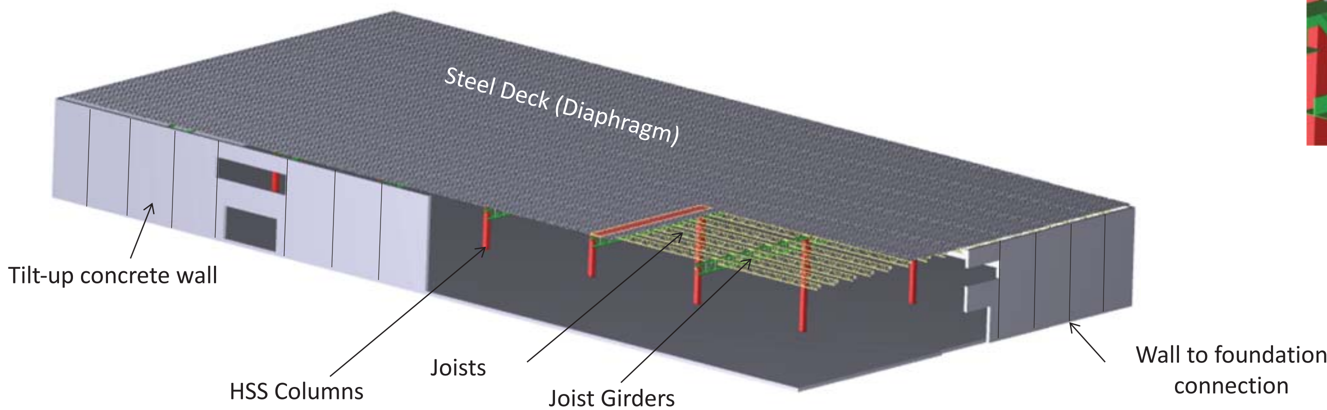
B.W. Schafer

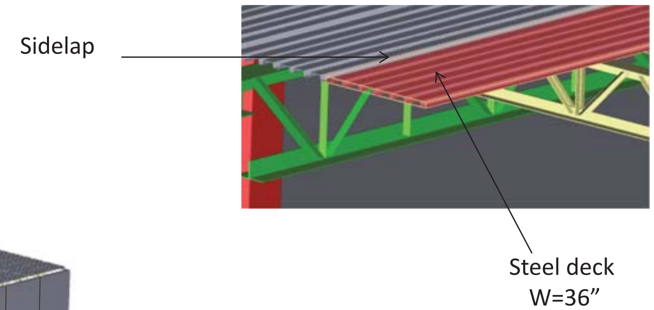
American Iron and Steel Institute

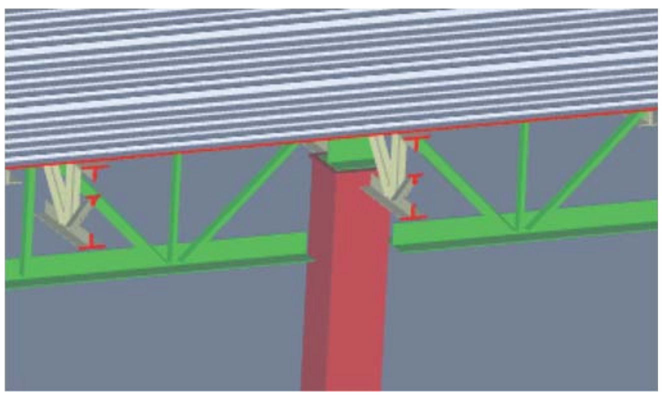
STEEL JOIST INSTITUTE MEMBER

STEEL DECK INSTITUTE sdi

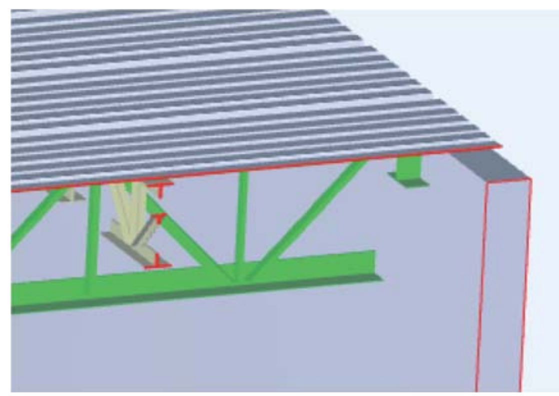
Steel Deck Rigid Wall-Flexible Diaphragm (RWFD) Building







Joist girder to column connection



Joist girder to concrete wall connection

Background: FEMA P-1026

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"At this time the alternate design procedure is not intended to apply to RWFD buildings with steel deck diaphragms. There are several reasons...
(1) tests results of a large scale diaphragm showed significantly less distribution of yielding than analyses ...,
(2) … design strengths are based on monotonic tests,
(3) data for reverse cyclically loaded connections is sparse …,
(4) the post-yield stiffness of connectors is positive for only a small deformation, …
(5) few reverse cyclically loaded diaphragm tests have been performed …, and
(6) many diaphragms in high seismic regions are designed using proprietary sidelaps for which no test data was available

… high priority for further research on steel deck diaphragms." pg. 6-7

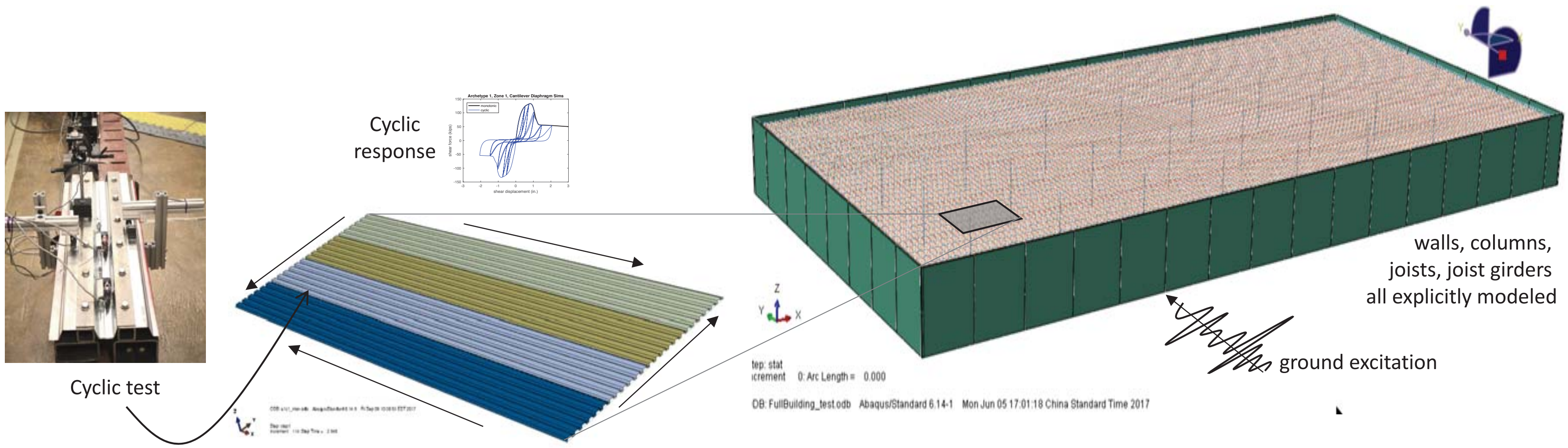
Seismic Design of Rigid Wall-Flexible Diaphragm Buildings: An Alternate Procedure

FEMA P-1026/March 2015

FEMA

nehrp

Overall framework for RWFD steel investigation



(a) Connector tests	(b) 3D Roof submodel	(c) 3D building model for dynamic analyses
• Cyclic sidelap and structural tests across gauges • Establish connector performance	• Shell FE model, material and geometric nonlin. • Similar to cantilever diaphragm testing • Nonlinear connectors • Establish cyclic perform-ance of roof segment	• Complete building archetype model • All primary and secondary systems modeled explicitly • Roof segments use nonlinear segments scaled to one joist span and one panel width • Opportunity to explore realistic expected response with damage progression • Vibration, pushover, IDA to reveal behavior

Solution to overcome previous challenges

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Challenge

“At this time the alternate design procedure is not intended to apply to RWFD buildings with steel deck diaphragms. There are several reasons...

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… high priority for further research on steel deck diaphragms.” pg. 6-7

Proposed Solution

(1) Change model from 2D to 3D and include all deck nonlinearities
(2) Dig into data, provide cyclic model
(3) Perform more testing across typical connections and gauges
(4) Employ nonlinear models that can capture negative post-peak behavior
(5) Dig into data, and create a simulation path for cyclic diaphragm performance
(6) Partner with industry in separate effort to investigate proprietary systems in a compatible manner to generic work

Acknowledgments

- The work reported in these slides was conducted by NBM Technologies for the American Iron and Steel Institute (AISI), Steel Deck Institute (SDI), and Steel Joist Institute (SJI).
- The NBM project team consisted of Ben Schafer, Matt Eatherton, Vahid Meimand, Shahab Torabian, and Brooks Smith.
- Today the work is being managed by the Cold-Formed Steel Research Consortium (CFSRC) through an MOU with AISI, SDI, and SJI and conducted by Ben Schafer.
- The Industry Steering Group (ISG) consists of Ken Charles, Bob Paul, Bonnie Manley, Dave Samuelson, Jay Larson, Jim Fisher, Pat Bodwell, and Tom Sputo.
- The work is being peer-reviewed by ATC-135, regular participants in this team include Veronica Cedillos (staff), John Lawson, Dominic Kelly (Chair), Robert Tremblay, Kelly Cobeen, Bob Hanson, Bonnie Manley, Mike Tong

Overview of Work

- Cyclic Connector Testing
- Primary Building Archetype
- Roof Submodels
- Building Archetype Modeling
- Next Steps and Discussion

ICTWS (2018) Summary paper of work to date

Eighth International Conference on
THIN-WALLED STRUCTURES – ICTWS 2018
Lisbon, Portugal, July 24-27, 2018

MODELING AND PERFORMANCE OF THIN-WALLED STEEL DECK IN ROOF DIAPHRAGMS UNDER SEISMIC DEMANDS

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Keywords: Thin-walled deck, bare steel deck, rigid-wall flexible-diaphragm, inelastic diaphragms.

Abstract. *The objective of this paper is to discuss new models used to predict the seismic performance of large warehouse buildings that employ bare thin-walled profiled steel deck panels as the primary diaphragm element. Warehouse and similar buildings that use tilt-up concrete walls and steel deck supported by open web steel joists, typically resting on HSS columns, are a class of Rigid Wall Flexible Diaphragm (RWFD) buildings. In North America the seismic design of such RWFD buildings has come under question and new methods have been proposed. The nonlinear behavior of the thin-walled steel deck in shear combined with additional nonlinearity between deck-to-deck connections and deck-to-structural connections that form the complete roof diaphragm creates a unique system with unusual energy dissipating mechanisms. A multi-scale model of RWFD buildings has recently been created and exercised under nonlinear time history analyses. Beyond revealing fundamental behavior, the intent of the RWFD modeling work is to provide an evaluation of existing design and newly proposed alternatives for design in North America - an effort that is currently ongoing.*

1 INTRODUCTION

Rigid Wall Flexible Diaphragm (RWFD) buildings are a unique class of structure which combines stiff, often heavy and compact, vertical elements (walls) with light, often thin-walled, horizontal elements (roof diaphragms), as shown in Figure 1. RWFD buildings potentially have unique seismic response since the mass, stiffness, and ductility are all distributed differently than in common building construction. Many warehouses may be classified as RWFD buildings, thus large economic exposure potentially exists when RWFD buildings experience seismic events.

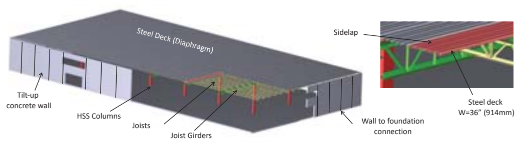

Figure 1: Typical RWFD steel deck diaphragm building

Cyclic Connector Testing

Cyclic Connector Tests

- Cover both sidelap (deck-to-deck) and structural (deck-to-joist/frame) connections in shear
- Cover generic connection types consistent with East and West Coast practice
- Modify AISI S905 test standard for cyclic

Report with all testing details available

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Cyclic performance and characterization of steel deck connections

NBM Technologies Inc.

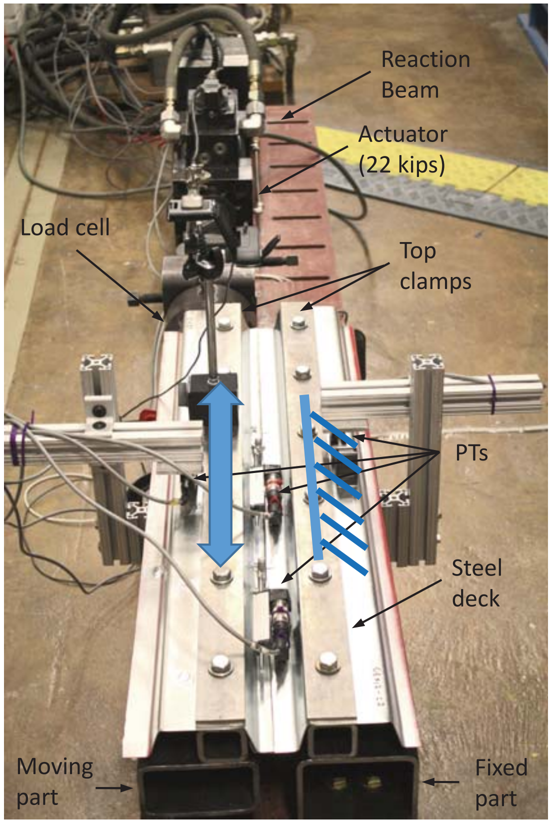
Shahab Torabian, PhD
Benjamin W. Schafer, PhD, PE

Prepared for:
American Iron and Steel Institute,
Steel Deck Institute,
Steel Joist Institute

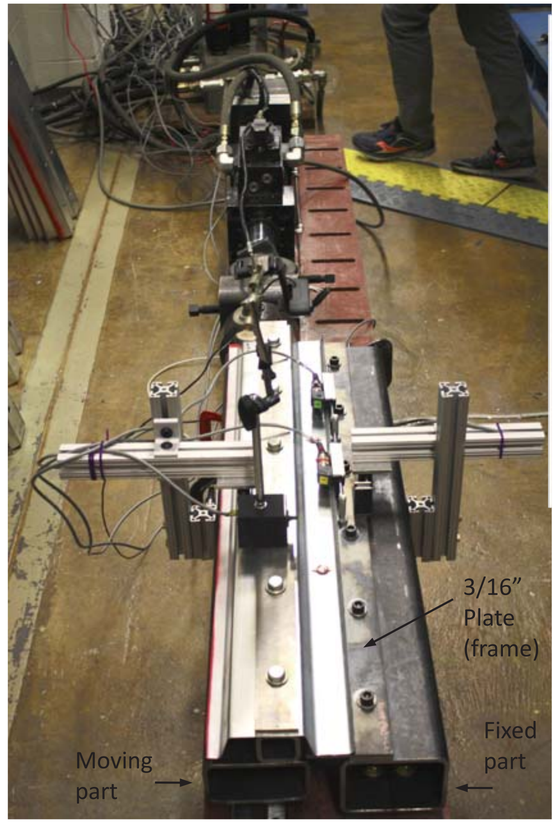
As part of the project: Advancing Seismic Provisions for Steel Diaphragms in Rigid Wall-Flexible Diaphragm Buildings, Phase II

14 December 2017

Testing in the TWS lab at JHU



Nestable sidelap



PAF frame connection

FEMA 461-Interim Protocol 1
Quasi-Static Cyclic Testing

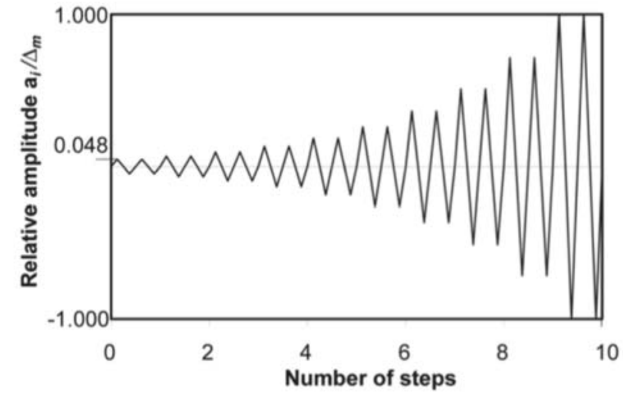


Figure 2-2 Loading history for $a_1 = 0.048\Delta_m$

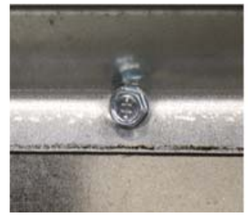
The loading history consists of repeated cycles of step-wise increasing deformation amplitudes. Two cycles at each amplitude shall be completed.

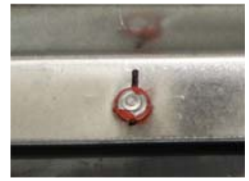
$a_{i+1}=1.4a_i$

Tested Connector Configurations

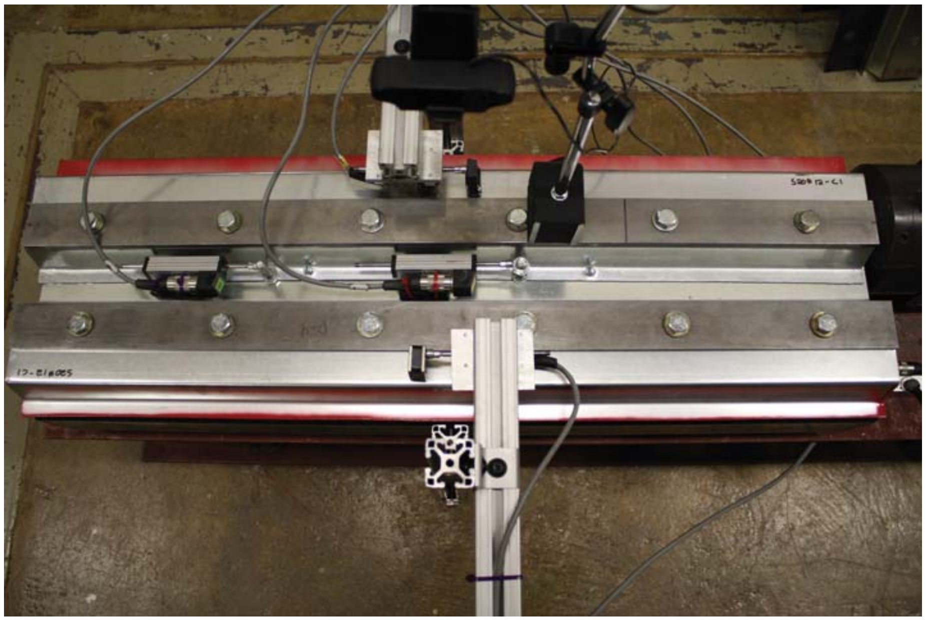
Deck (1.5 in. WR)	Ply 1 (gauge)	Ply 2 (gauge)	Connector	# tests[6] n
nestable	18	18	#12 screw	4
nestable	20	20	#12 screw	4
nestable	22	22	#10 screw	4
interlock	18	18	Top Arc Seam Weld[2]	4
interlock	20	20	Top Arc Seam Weld[2]	4
interlock	22	22	Top Arc Seam Weld[2]	4
nestable	18	plate[1]	PAF-Hilti[3]	4
nestable	20	plate[1]	PAF-Hilti[3]	4
nestable	22	plate[1]	PAF-Hilti[3]	4
nestable	18	plate[1]	Arc spot[4]	4
nestable	20	plate[1]	Arc spot[4]	4
nestable	22	plate[1]	Arc spot[4]	4
interlock	18	plate[1]	Arc seam[5]	4
interlock	20	plate[1]	Arc seam[5]	4
interlock	22	plate[1]	Arc seam[5]	4

1. 4.76 mm (3/16 in. plate)
2. 38.1 mm (1.5 in.) long weld
3. HILTI X-HSN 24 PAF
4. visible weld diameter 19 mm (3/4 in.)
5. Visible length 38 mm (1.5 in.), width 9.5 mm (3/8 in.)
6. 1 monotonic and 3 cyclic for each unique condition.





Test Results : Sidelap - Screw

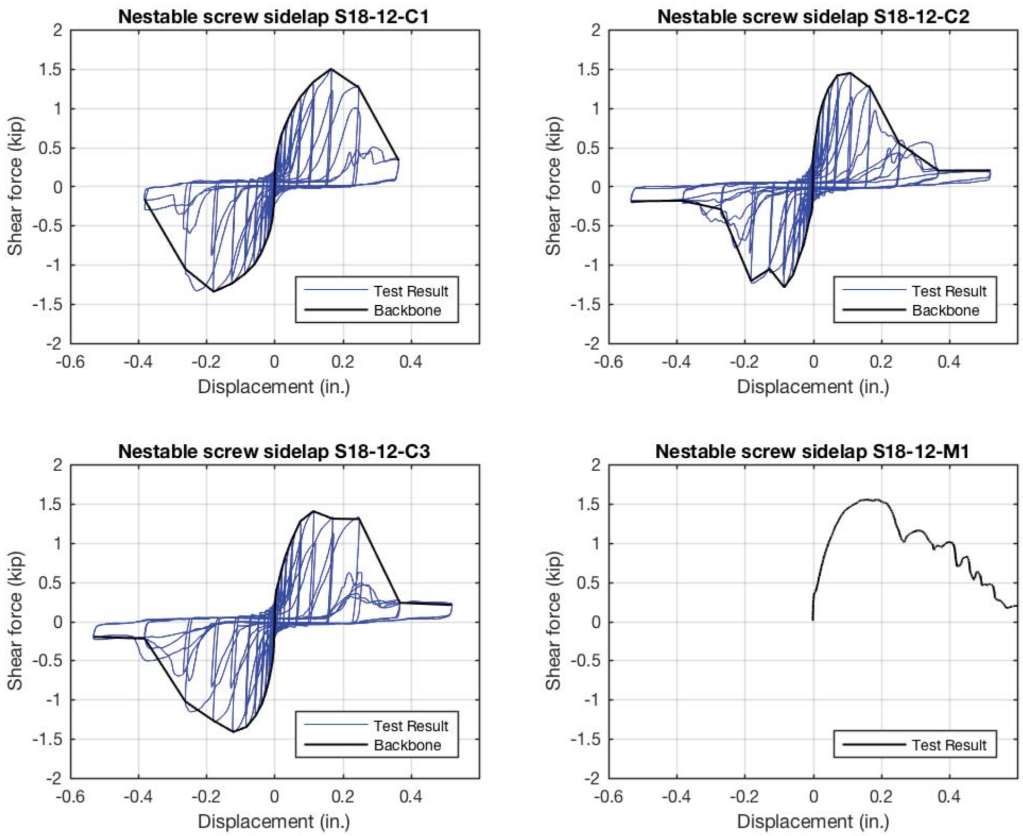


Specimen designation:

S18-12-C1

- SideLap
- gage 18, 20, and 22
- #12, #10
- C: Cyclic
  M: Monotonic

Test Results : Sidelap – Screw (1 fastener)

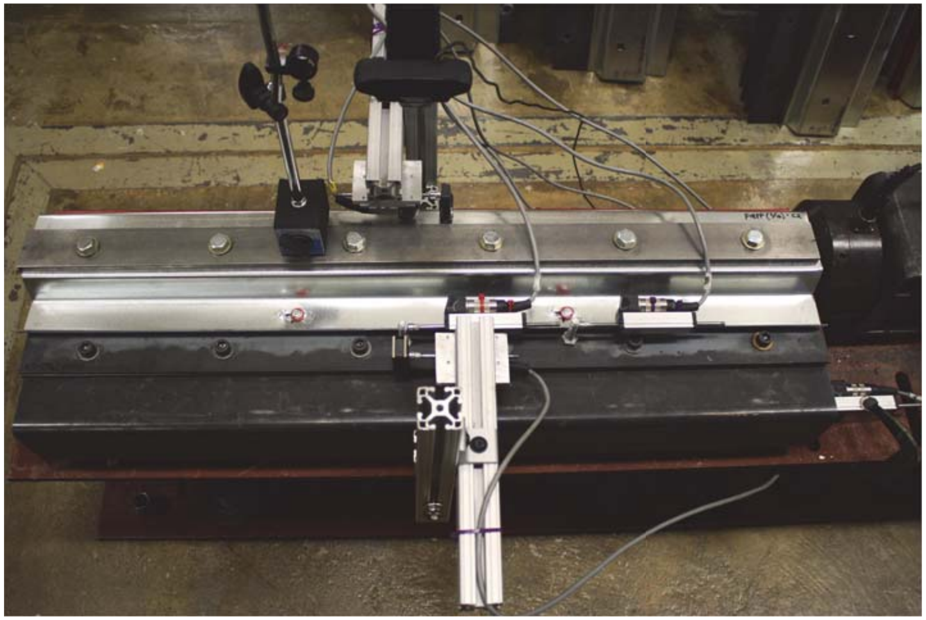


1.5 in WR nestable sidelap

Deck thickness: 1.5 in WR- 18 ga
Fastener: Hilti #12

Note:
Test results (shear force) are divided by two to provide results for one fastener.

Test Results : Framing- Powder Actuated Fasteners



Specimen designation:

F18PF-3-16-C1

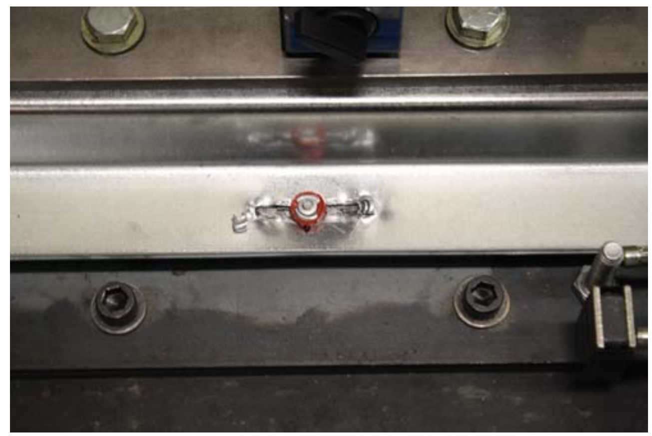
Framing

gage
18, 20, and 22

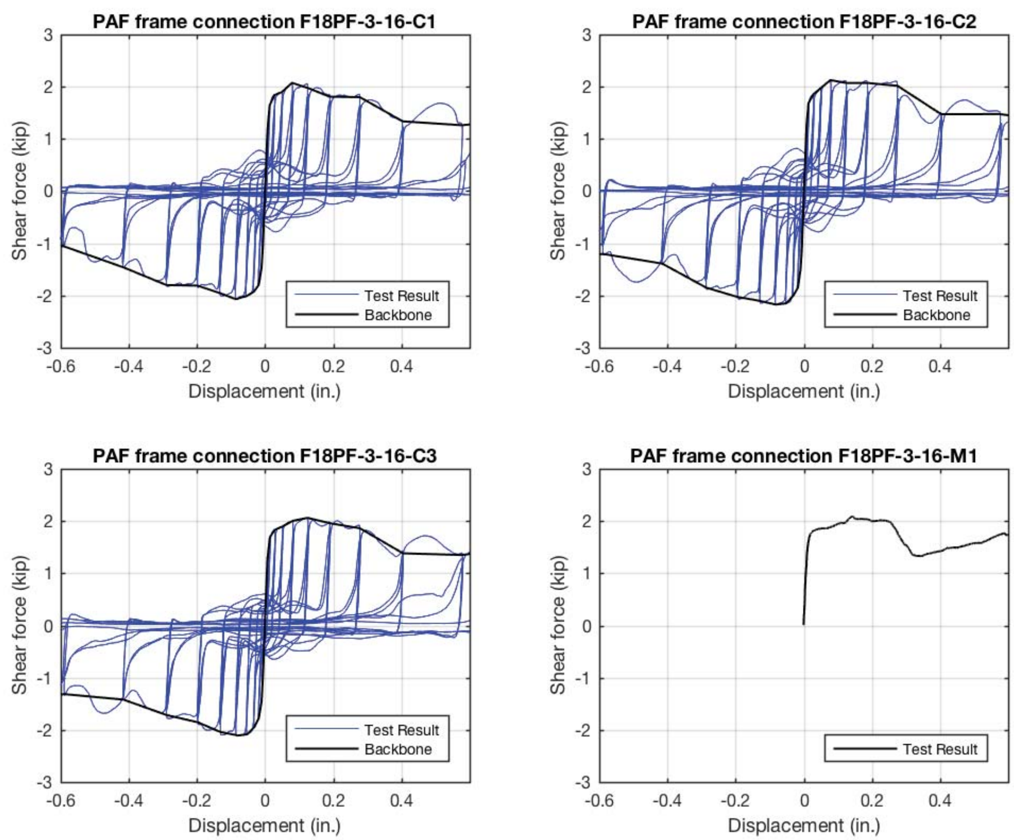
Powder
Actuated
Fasteners
(PAF)

Thickness
of frame=
3/16”

C: Cyclic
M: Monotonic



Test Results : Framing- PAF (1 fastener)



1.5 in WR connection to frame

Deck thickness: 1.5 in WR- 18 ga
Frame thickness: 3/16”
Fastener: PAF HILTI X-HSN 24

Note:
Test results (shear force) are divided by two to provide results for one fastener.

Frame element is a flat plate.
Width=4 in. and length =36 in.

Test Results : Framing- Arc Spot Weld

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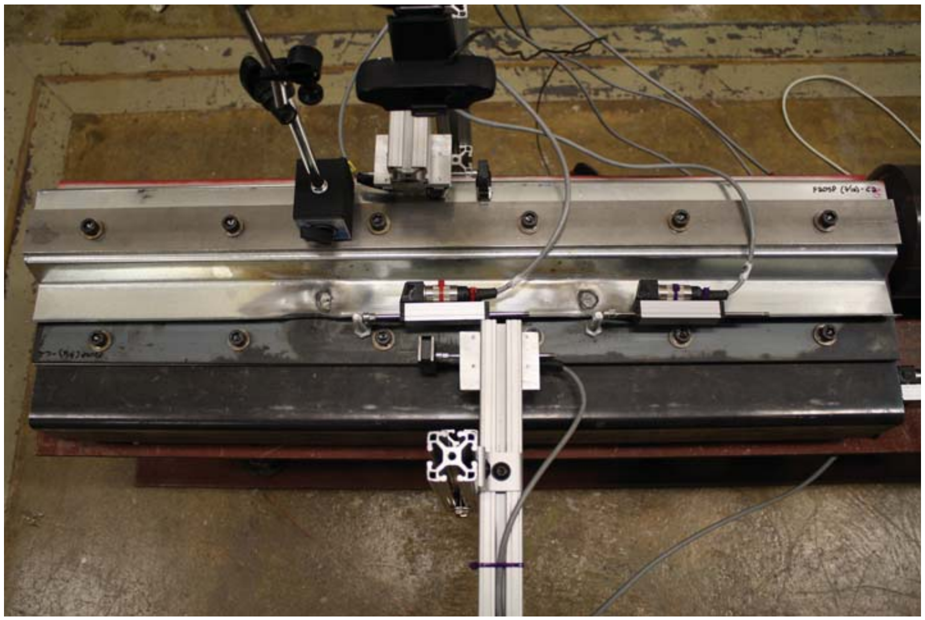
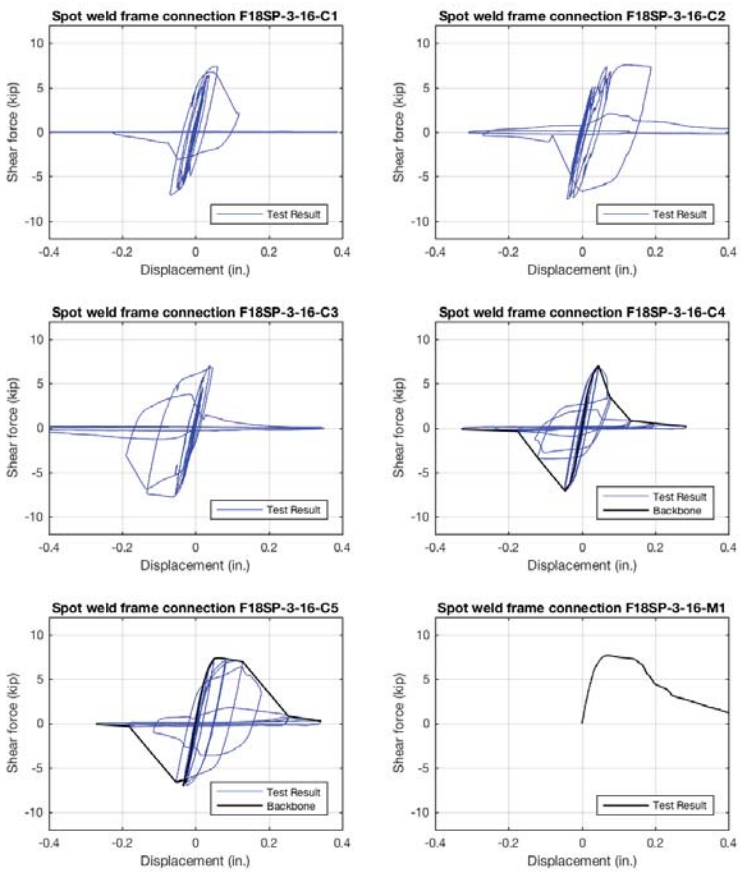


Specimen designation:

F18SP-3-16-C1

- Framing
- gage 18, 20, and 22
- Arc Spot Weld
- Thickness of frame= 3/16”
- C: Cyclic
  M: Monotonic

Test Results : Framing- Arc Spot Weld (per weld)



1.5 in WR connection to a frame member via Arc Spot Welds

Deck thickness: 1.5 in WR- 22 ga
Frame thickness: 3/16”
Fastener: Arc Spot Weld, Nominal diameter of 5/8”. Welds were measured as follows:

Specimen	Weld Diameter (in)				Average (in)
	1_long	1_trans	2_long	2_trans	
F18SP (3/16) - M1	0.788	0.760	0.771	0.757	0.769
F18SP (3/16) - C1	0.797	0.744	0.814	0.754	0.777
F18SP (3/16) - C2	0.783	0.824	0.750	0.747	0.776
F18SP (3/16) - C3	0.765	0.788	0.763	0.726	0.761
F18SP (3/16) - C4	0.773	0.774	0.795	0.797	0.785
F18SP (3/16) - C5	0.707	0.783	0.783	0.746	0.755

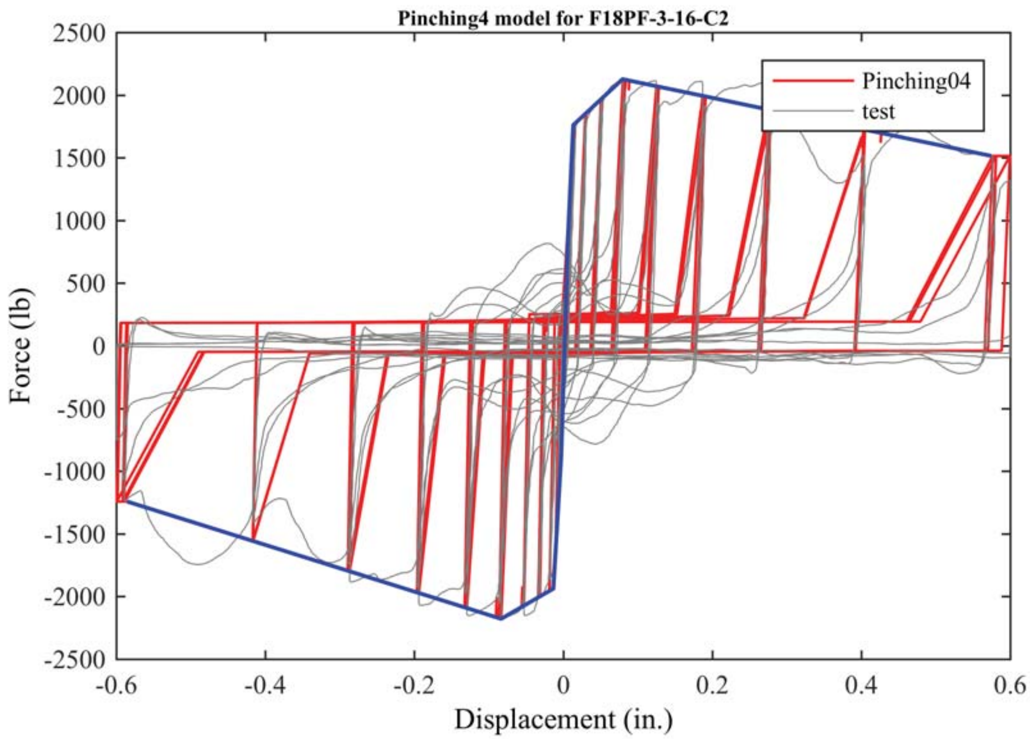
Note: Nominal weld diameter=0.625"
long = longitudinal
trans = transverse

Note:
Test results (shear force) are divided by two to provide results for one fastener.

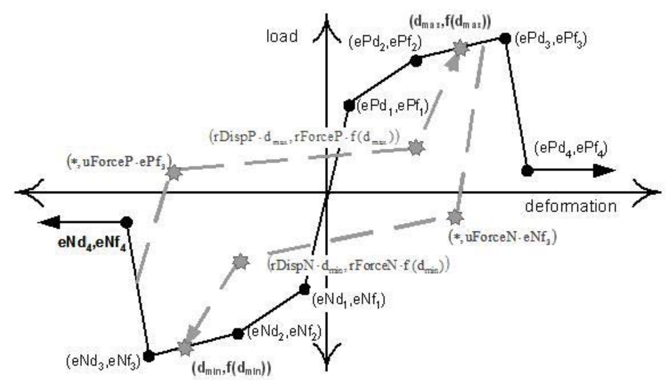
F18SP(3/16)-C1, C2 , and C3 are just used for initial stiffness and maximum load and F18SP(3/16)-C4 are C5 applicable for cyclic backbone.

Frame element is a flat plate. Width=4 in. and length =36 in.

Fit hysteretic spring to connector tests



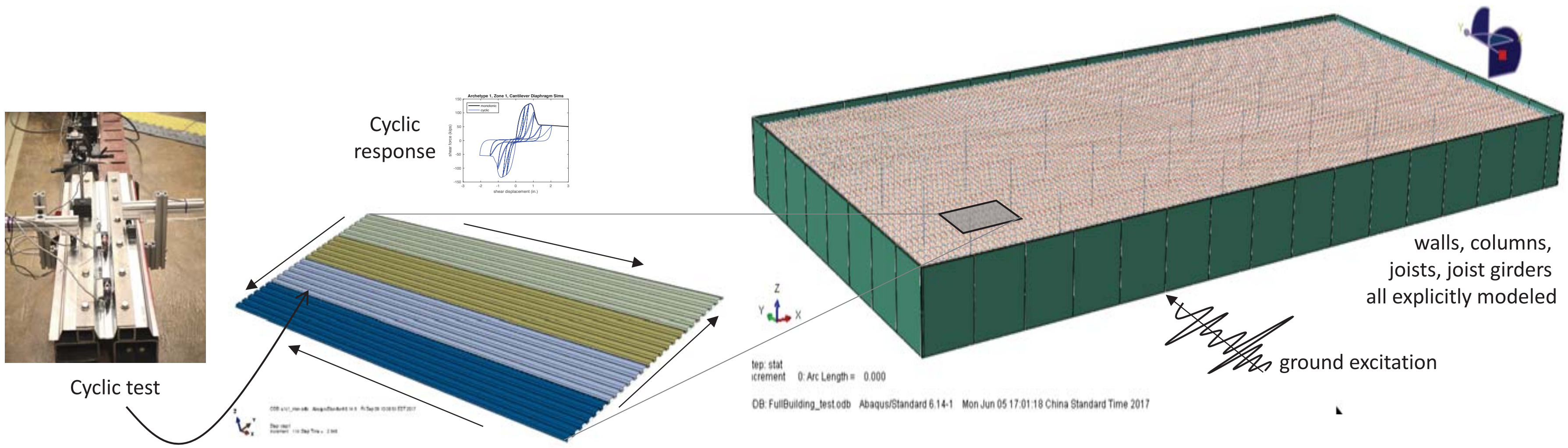
Pinching04 model is fit to the individual test results by implementing an unconstrained optimization method to find re-loading and un-loading parameters:



http://opensees.berkeley.edu/wiki

Ding (2015) implemented thid model in ABAQUS

Overall framework for RWFD steel investigation



(a) Connector tests

- Cyclic sidelap and structural tests across gauges
- Establish connector performance

(b) 3D Roof submodel

- Shell FE model, material and geometric nonlin.
- Similar to cantilever diaphragm testing
- Nonlinear connectors
- Establish cyclic perform-ance of roof segment

(c) 3D building model for dynamic analyses

- Complete building archetype model
- All primary and secondary systems modeled explicitly
- Roof segments use nonlinear segments scaled to one joist span and one panel width
- Opportunity to explore realistic expected response with damage progression
- Vibration, pushover, IDA to reveal behavior

Primary Building Archetype

RWFD Building Archetypes

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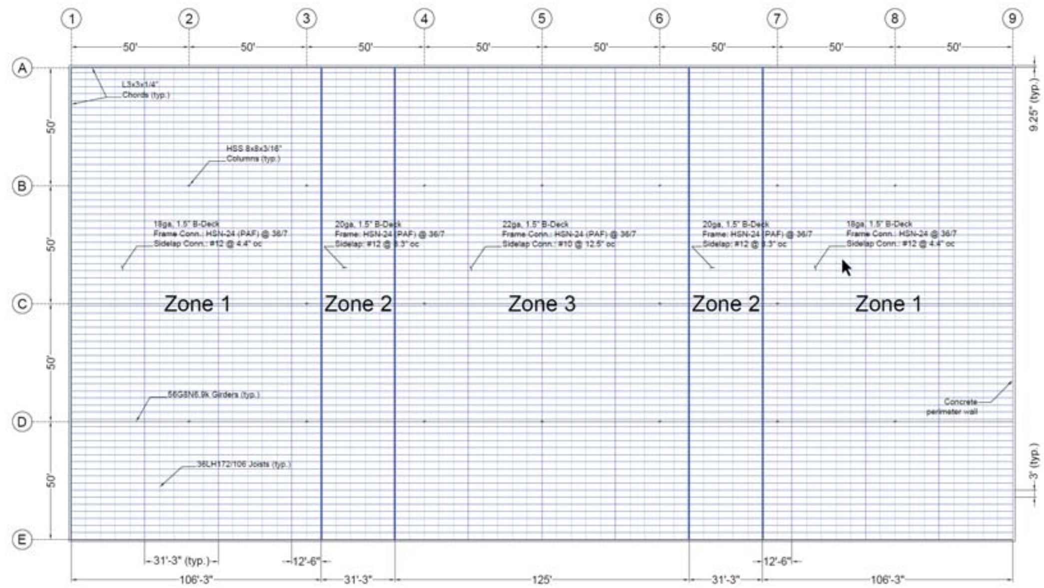
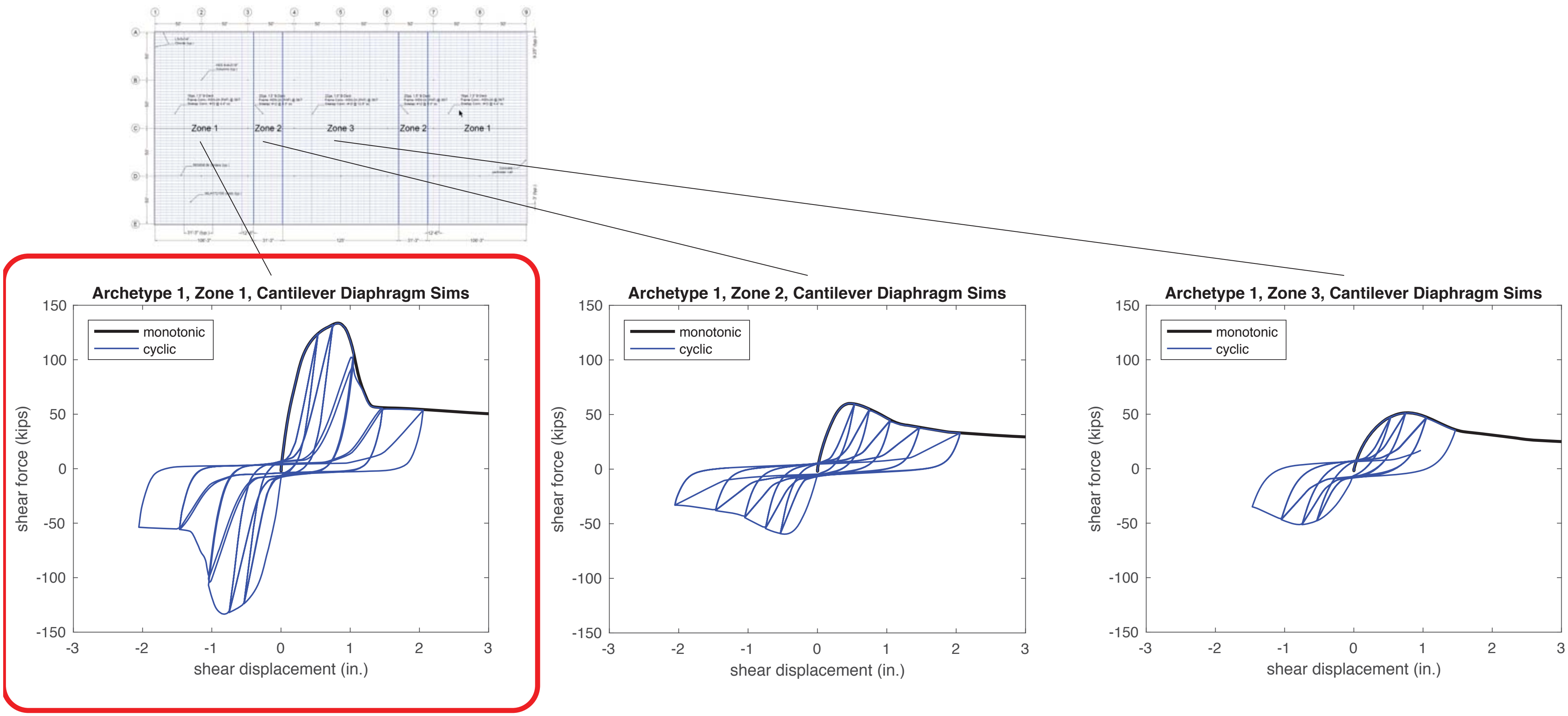
- RWFD Building Archetype suite established in FEMA P-1026
- Large, Medium, and Small RWFD buildings in SDC C and D_{max}
- For steel FEMA P-1026 model predicted poorest performance in Large RWFD buildings, High Seismic (SDC D_{max})
- Large building selected for primary archetype study here
- Roof re-designed per AISI S310-13 and ASCE7-10, limited to configurations tested (using code nominals not test)



- 200x400 ft in plan
- 1.5in. WR deck
- PAF for structural, screw for sidelaps
- Detailed across three zones 18→22 gauge
- 31' panels, 5' joist spacing

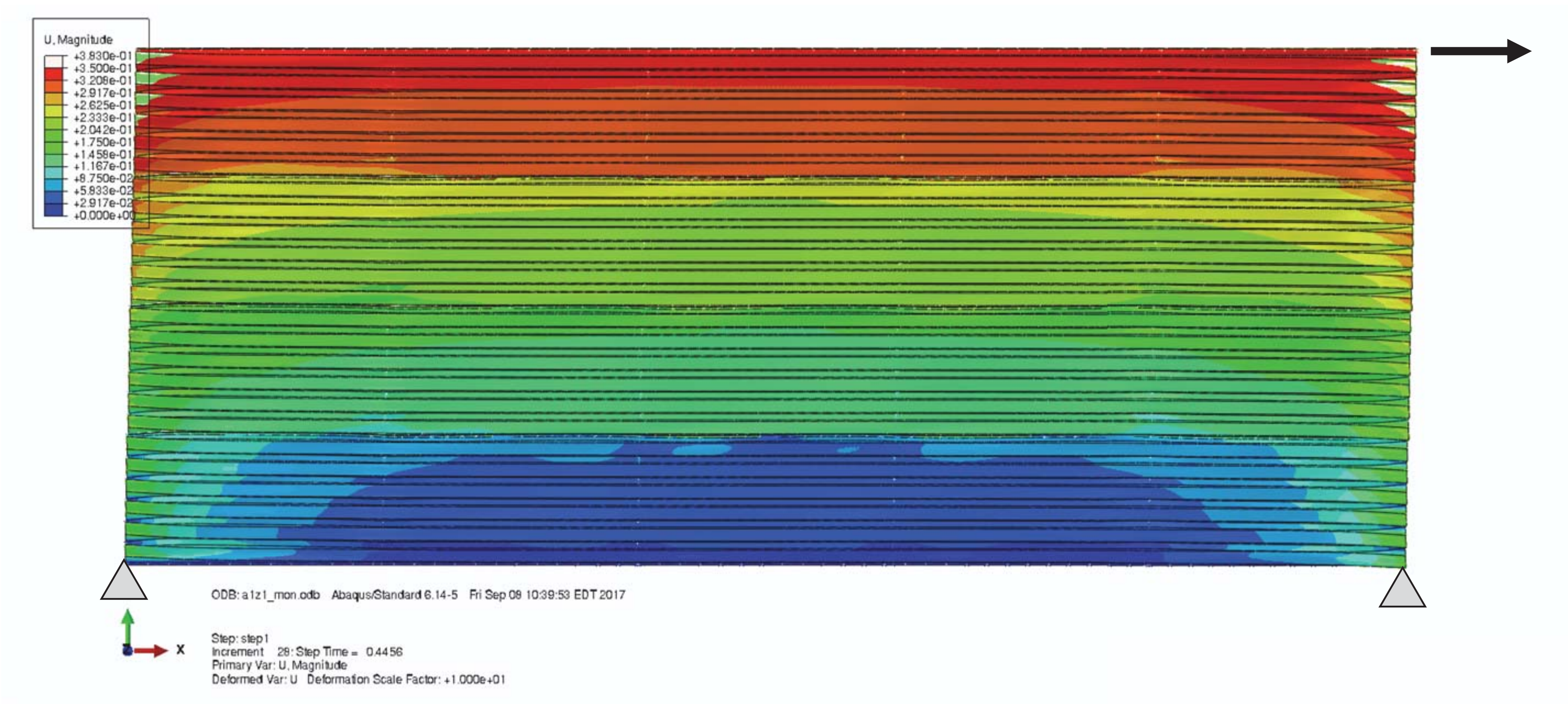
Roof Submodel Performance

A1: Roof submodel performance



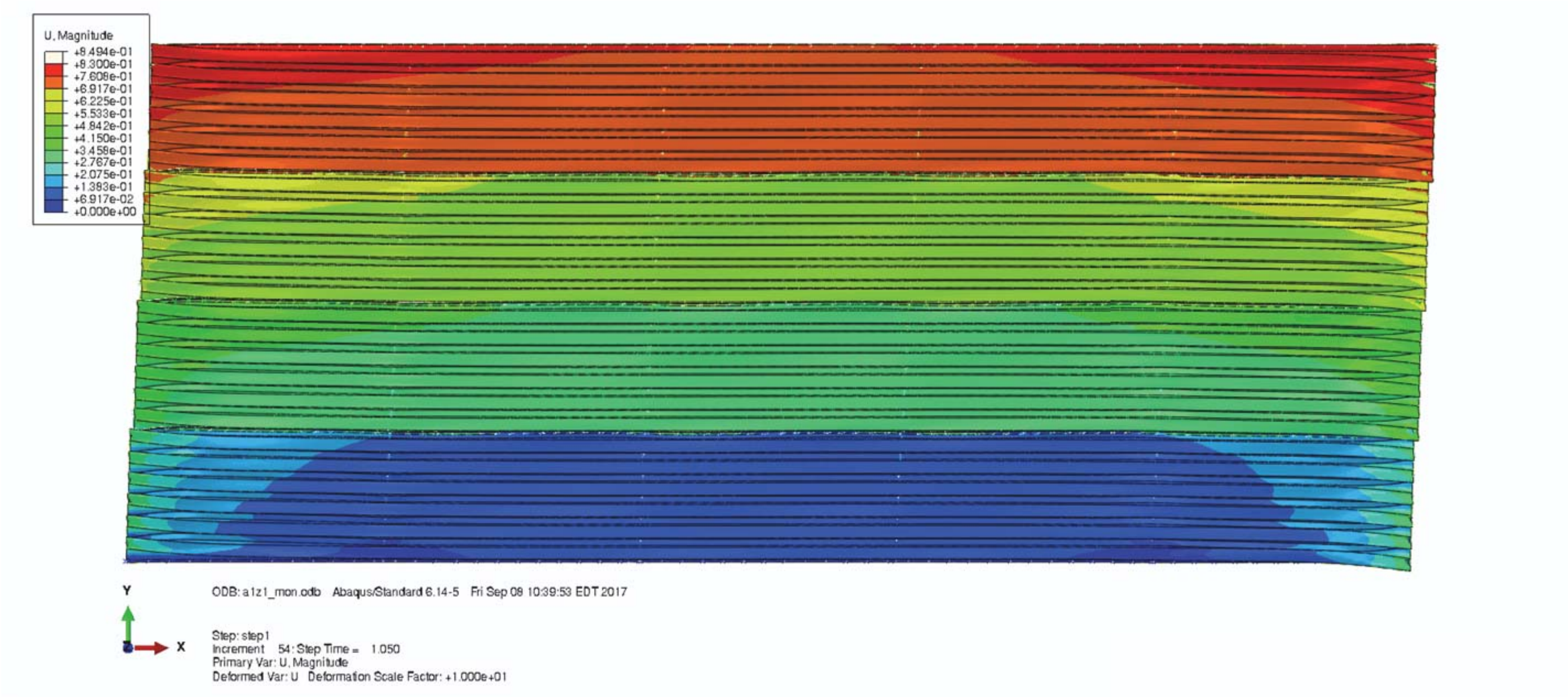
Result of roof submodel simulations that drive the full building model in-plane shear

A1Z1 – 80% pre-peak shear - displacement



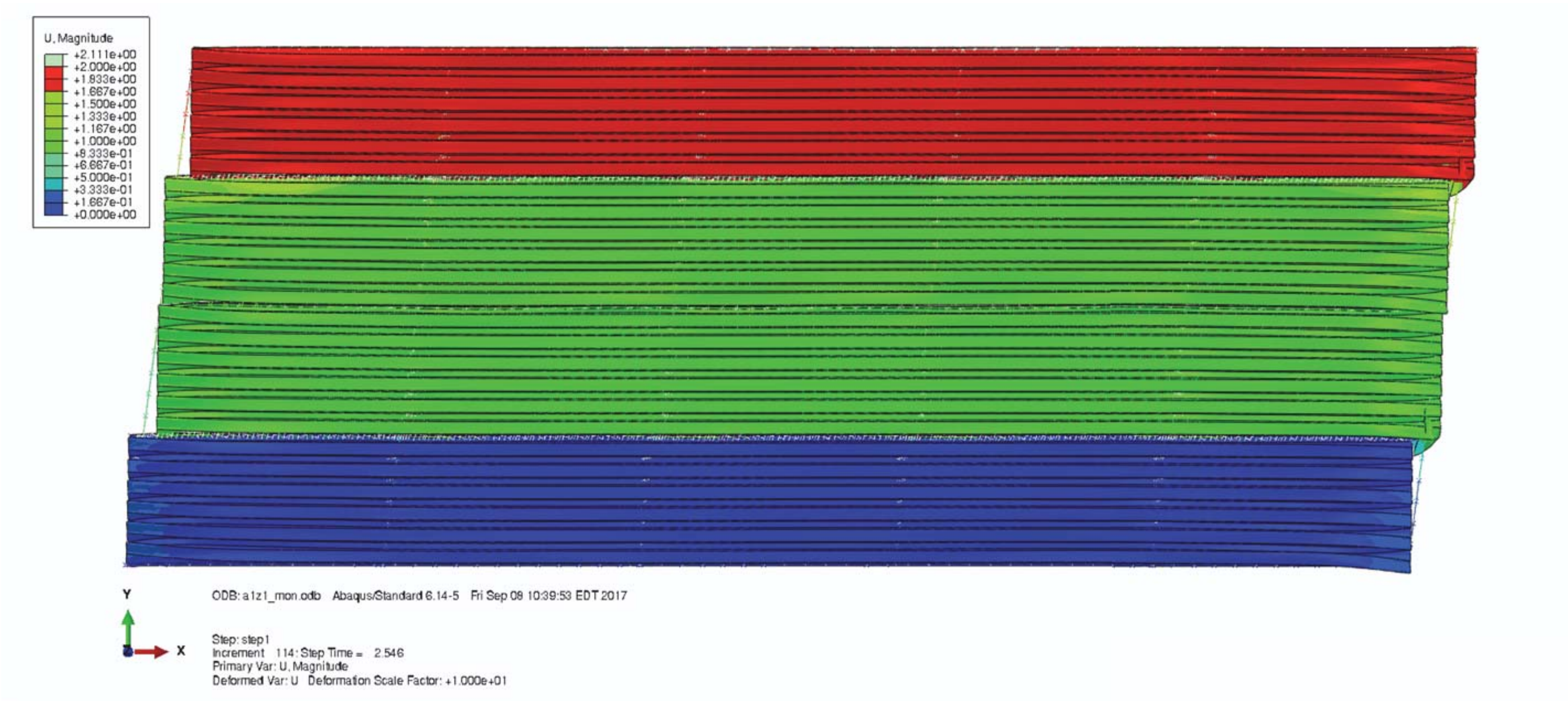
Note deck warping, otherwise relatively continuous “stressed” skin response

A1Z1 – peak shear - displacement



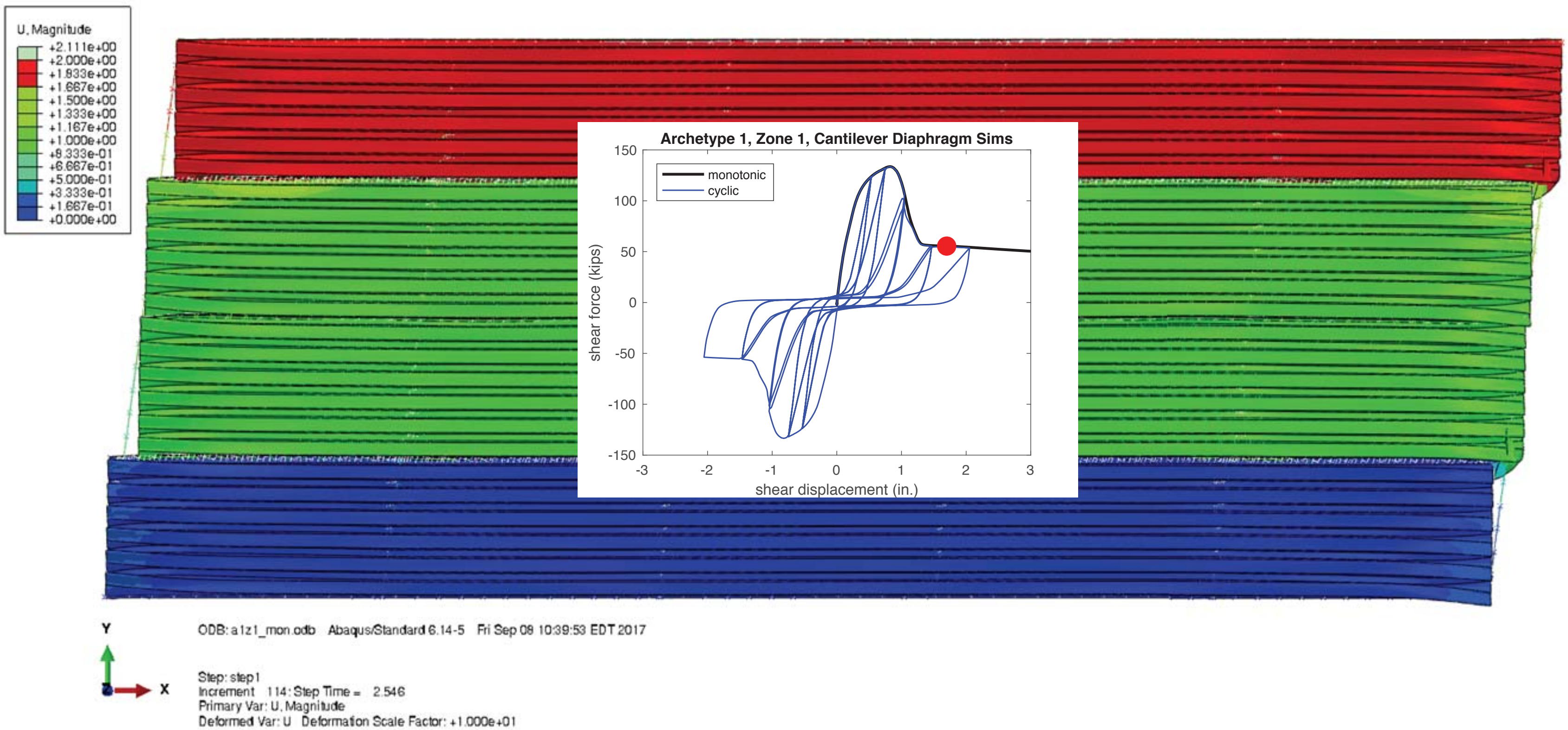
Note beginning of sidelap slip (separately, deck stress nonuniform but below yield)

A1Z1 – 40% post-peak shear - displacement



Note sidelaps lost, but individual panels deform with joists until structural connectors lost

A1Z1 – 40% post-peak shear - displacement

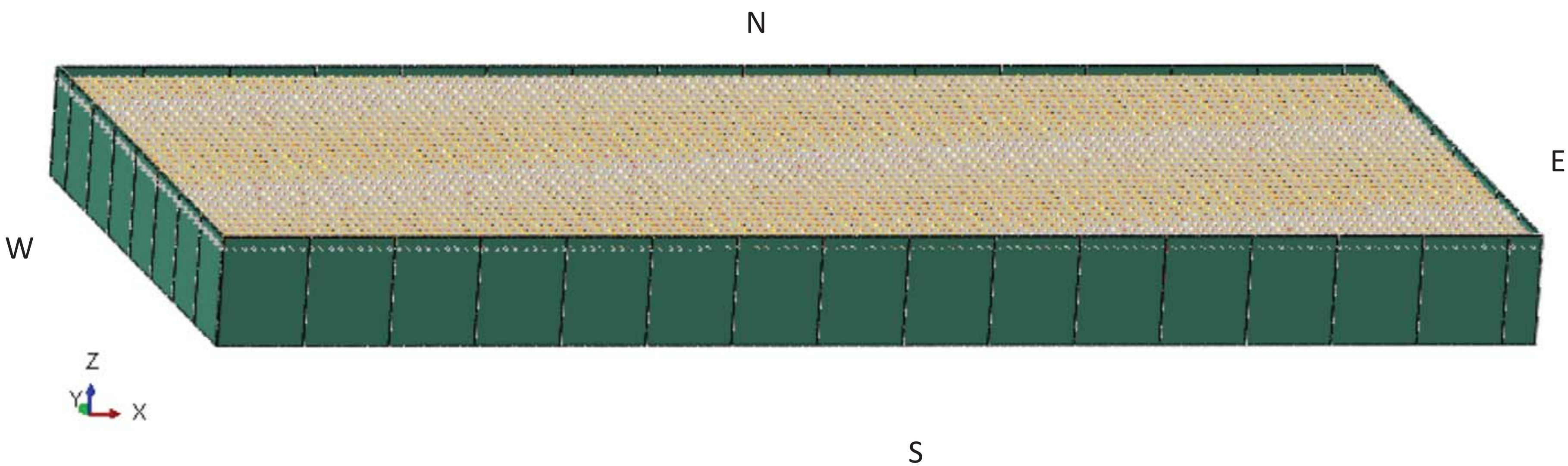


Note sidelaps lost, but individual panels deform with joists until structural connectors lost

Building Archetype Performance

Archetype 1 Building Model – Isometric

3 roof zones.. See archetype report for further details.



ABAQUS building model, roof developed by submodel, see previous work for full details.

A1: Modal Analysis and Discussion

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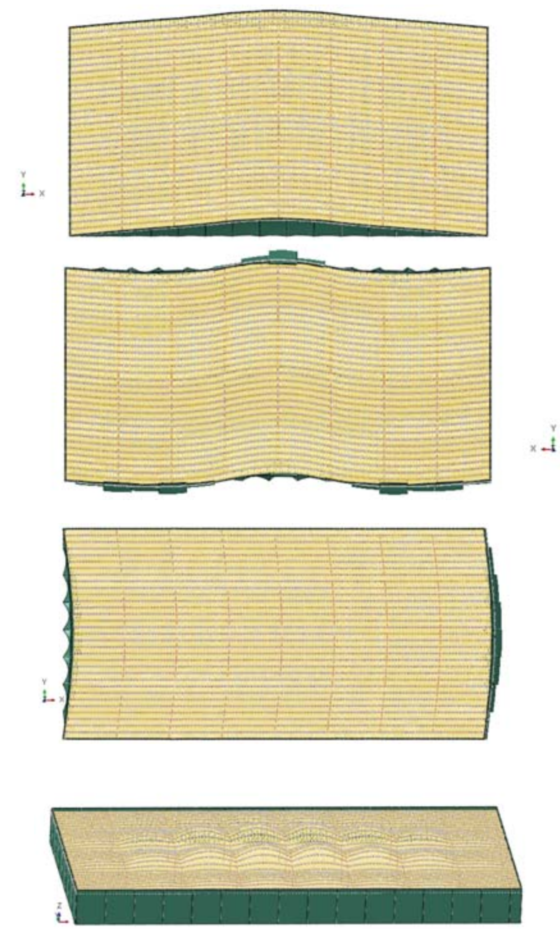
Mode	f	T	M_{eff}	W_{trib}
N-S Lateral (1st)	1.69 Hz	0.59 s	$0.49M_{tot}$	$0.47W_{tot}$
N-S Lateral (2nd)	3.43 Hz	0.29 s	$0.09M_{tot}$	
E-W Lateral (1st)	3.56 Hz	0.28 s	$0.21M_{tot}$	$0.32W_{tot}$
Vertical (1st)	3.31 Hz	0.30 s	$0.07M_{tot}$	$0.17W_{tot}$



Discussion

- Mode shapes and period are generally per expectation
- ASCE 7, T prediction is 0.26 s, actual T predicted longer (NS vs EW..)
- The effective mass can tell us something about the mass which is engaged and thus what ELF should be designed for,
    - ASCE 7-16, language
      "The effective seismic weight, W, of a structure shall include the dead load…above the base"
      "w_{px} = the weight tributary to the diaphragm at level x."
- In RWFD building can/should W and w_{px} be different?
- Simple tributary assumptions for w_{px}, common in design examples, don't seem grossly in error, extend to W (likely not)?

A1: N-S Pushover and Discussion

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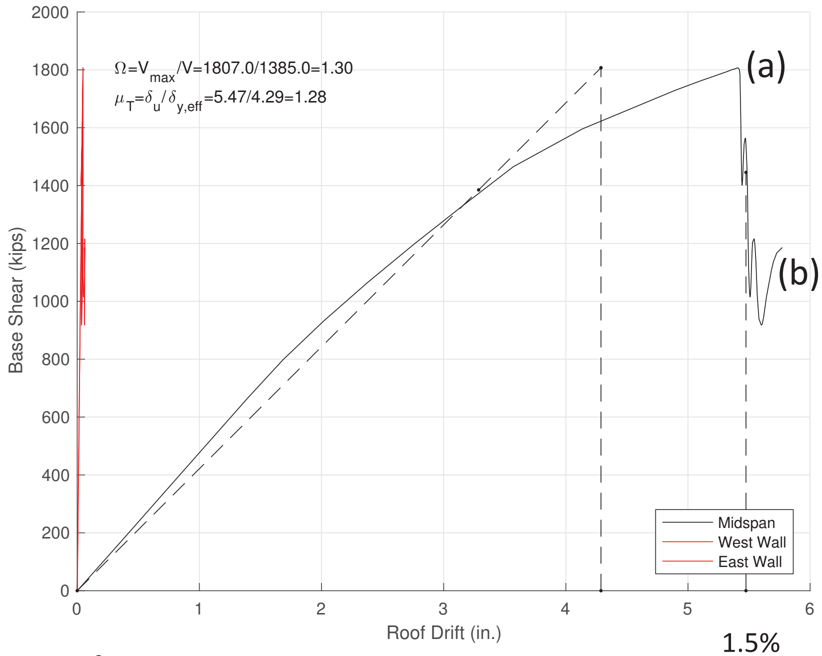
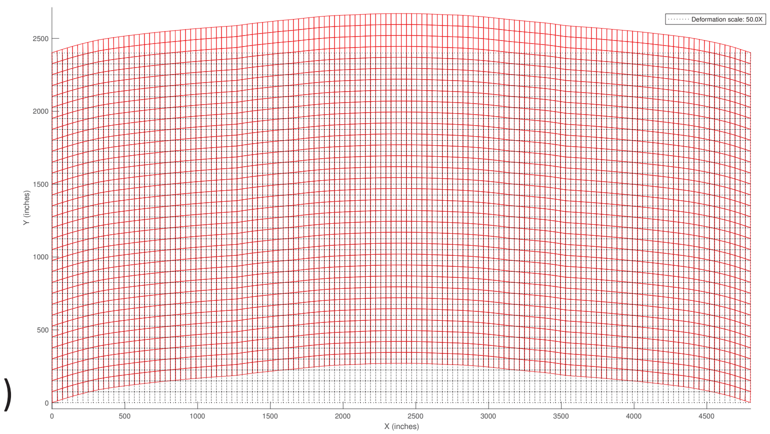


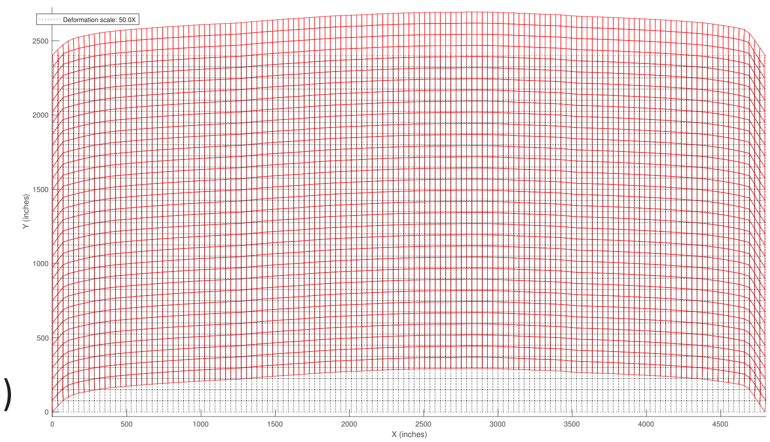
Discussion

- Loading is line load at the edge, could modify
- Capacity OK, note 1.5% drift & large force drop
- Previous runs show response continues past (b) this may be needed/helpful for interpreting later results
- Note, base shear at first drop is ~1800 kips.

(a)



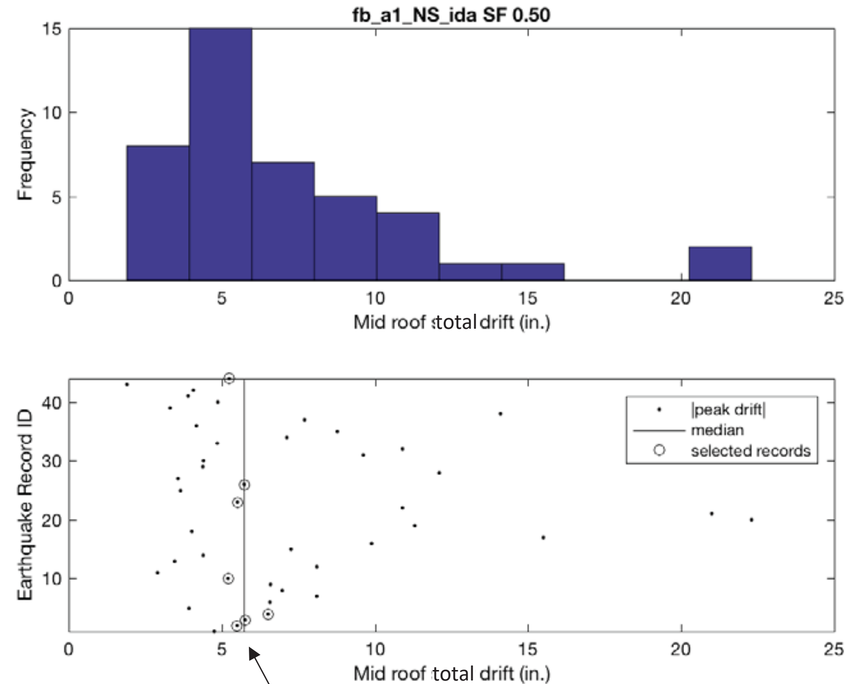
(b)



A1: N-S IDA procedure

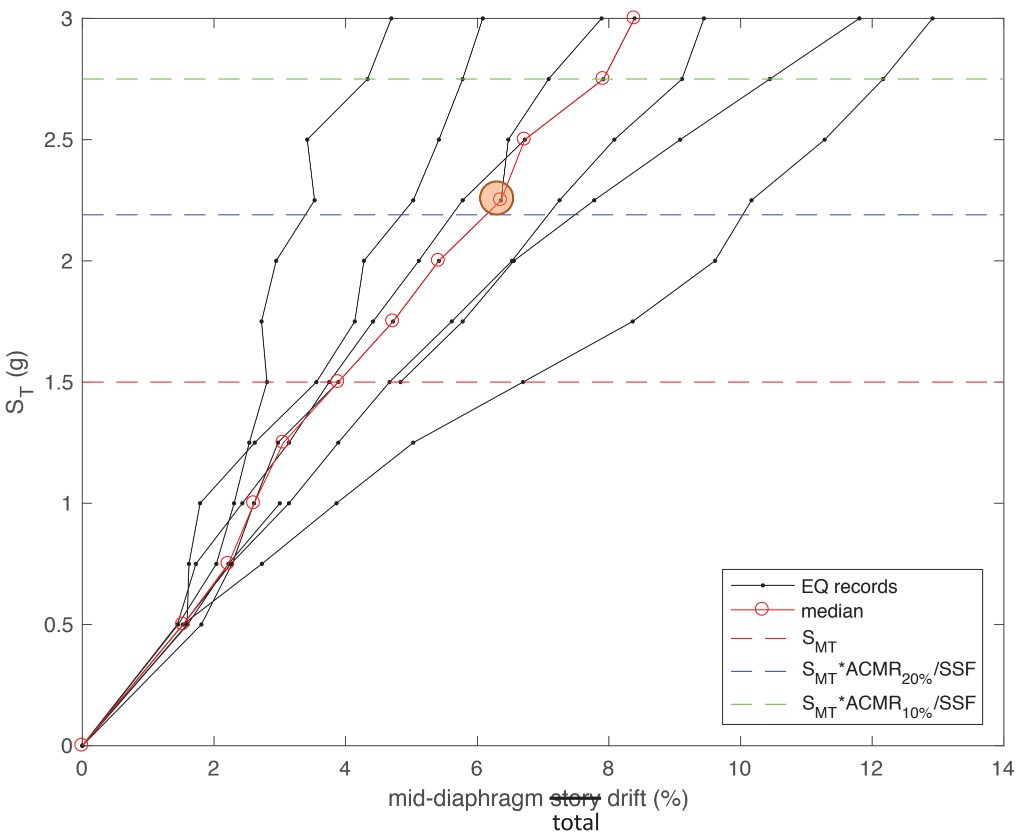
- FEMA P695 notion is to perform nonlinear time history analyses of the building across broad suite of EQs and EQ levels
- From these analyses we are intended to find the median response and compare the EQ level that collapses the building to the adjusted (MCE) design level
- For Archetype 1, S_{DS}=1.0 (DBE) and therefore S_{MT}=1.50 (MCE).
- We ran the 44 P695 EQ motions at $0.5S_{MT}$ in an effort to find the median 7 EQs that we would explore further.
- Results to the right used to find median 7 EQs to continue

Peak total mid roof drift across P695 EQ suite



7 closest to median selected

A1: N-S IDA Results

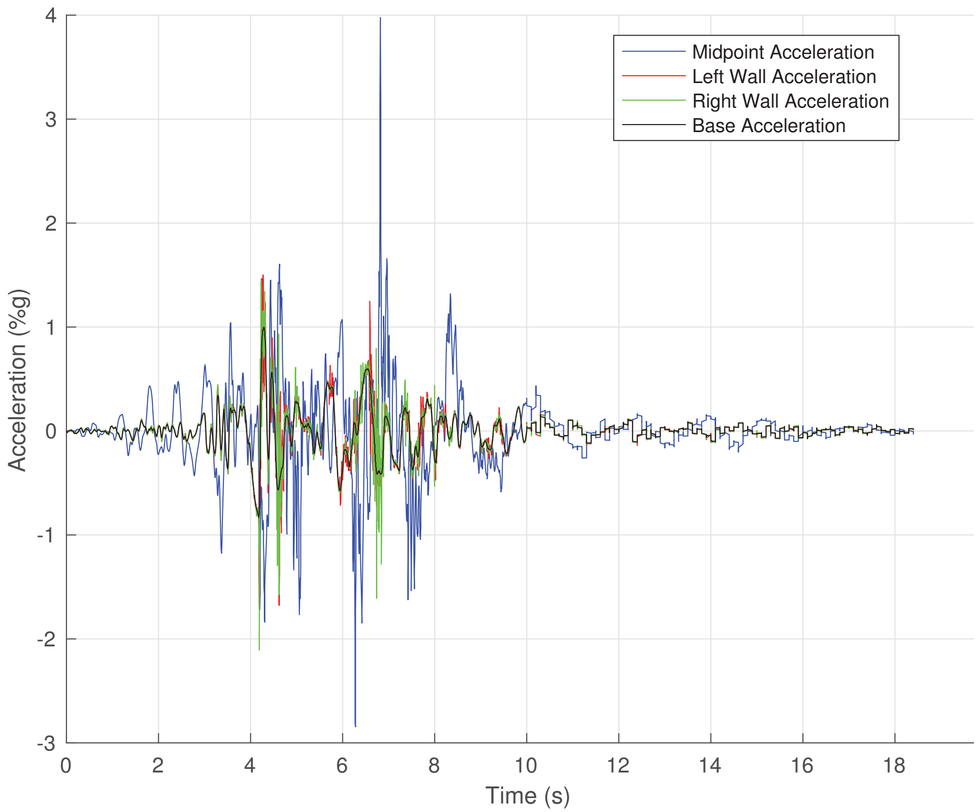


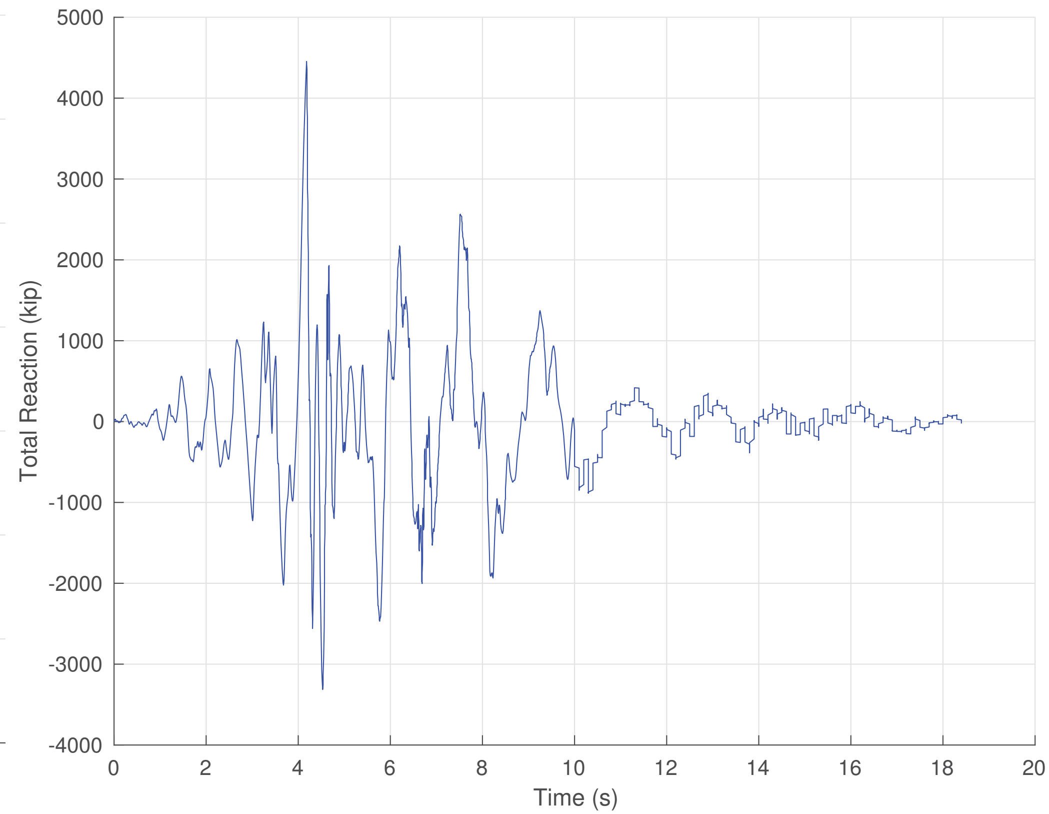
(This drift is actually total drift. Needs small correction as right now base drift is included here. TBD, does not change overall picture..)

Discussion

- Every point is the peak total roof drift from a nonlinear time history analysis
- Drift demands increase as the EQ level is increased
- Drifts have not "blown up" though they are large
- These predicted drifts need small correction, see note below figure
- To understand what this is really telling us we pick median EQ to dive down into the results further.

A1: N-S SF2.25 EQ4 Time History Response





This is a strongly scaled motion, large amplification of the Accelerations, nearly 4g peak experienced mid-wall.

Base shear experienced in this record is high > 4000 kips, and significantly in excess of pushover ($m\ddot{x} + c\dot{x} + kx$)!

A1: N-S SF2.25 EQ4 Time History Response

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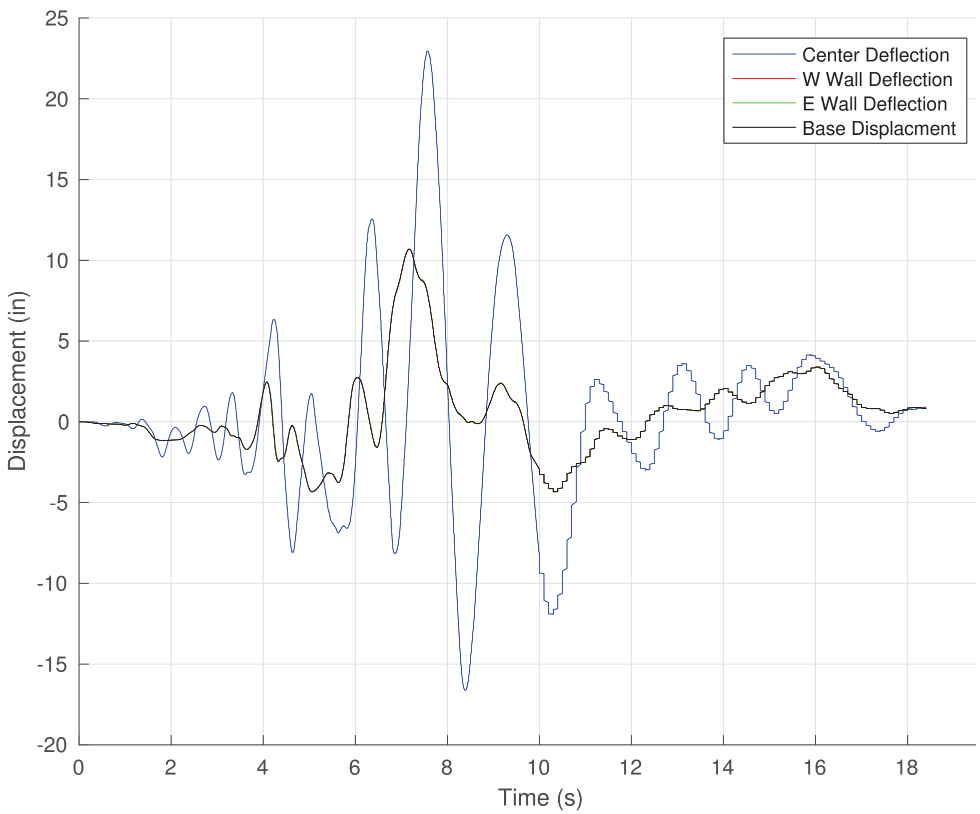
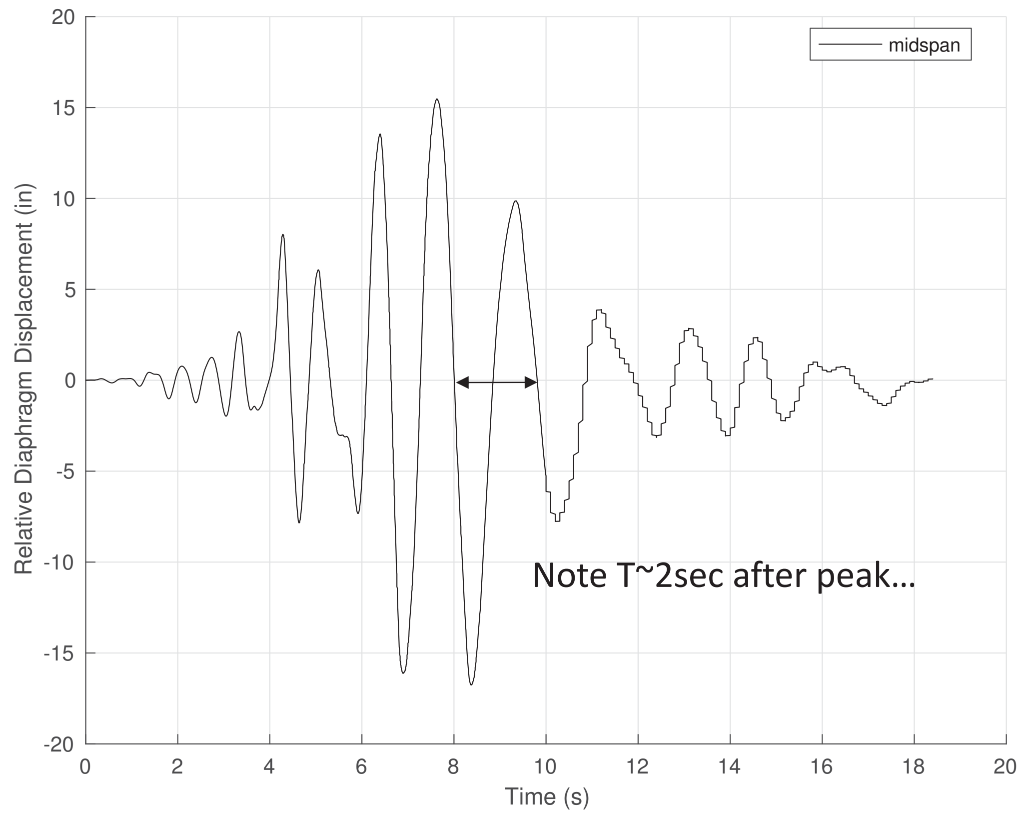
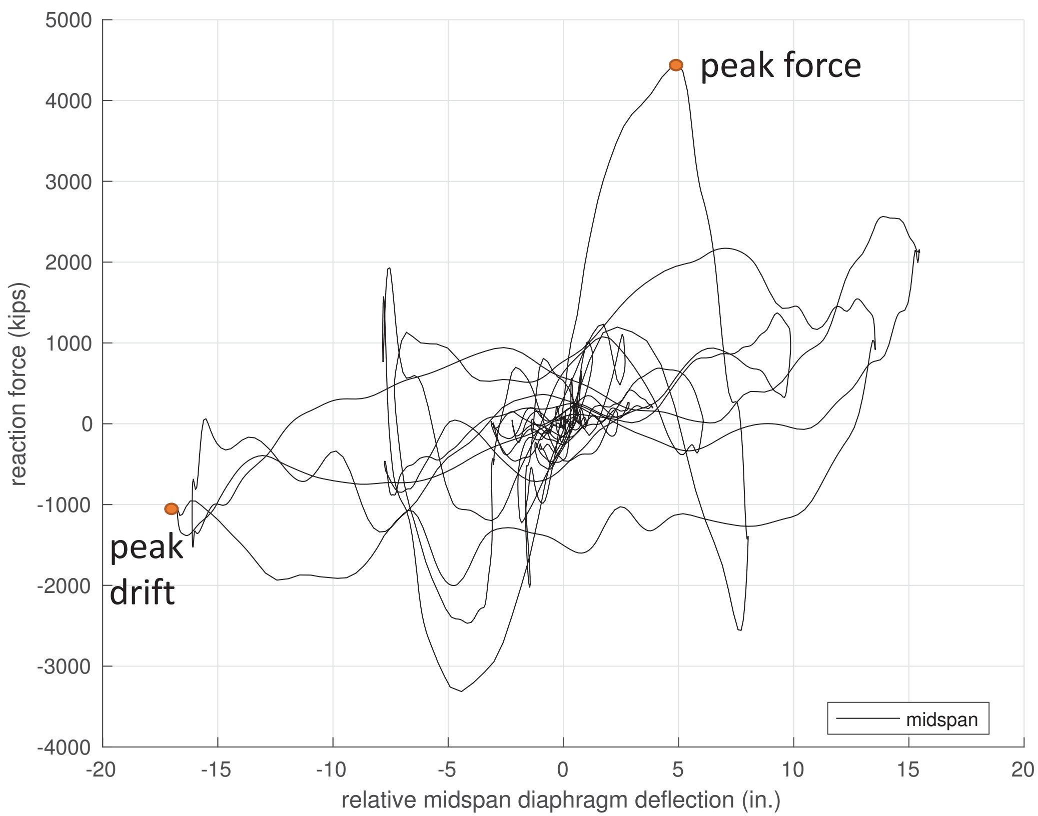




Base accelerations imply base drift, so what we need is relative drift at top of diaphragm.

Relative drift, or just story drift for the diaphragm is much less, though still peaks are 16 in. = 4.4%.

A1: N-S SF2.25 EQ4 Force-Displacement

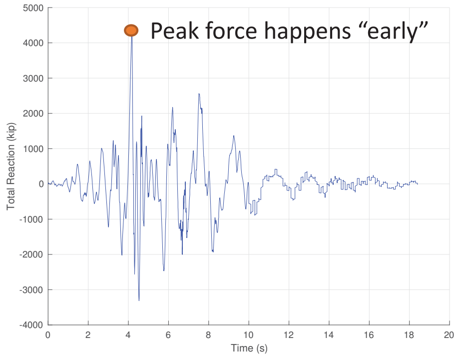


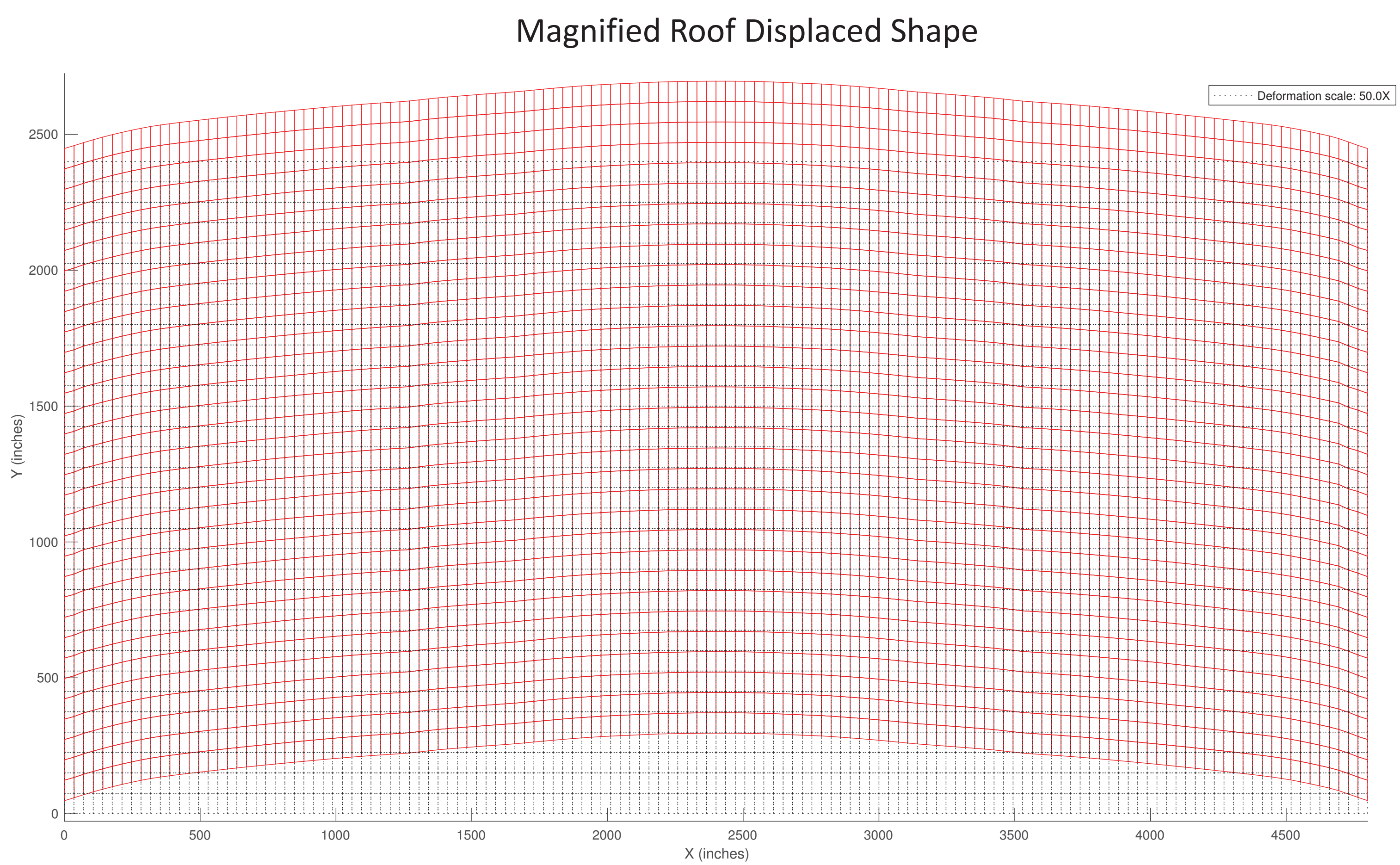
Base shear vs. diaphragm displacement EQ response.
Stiffness degrades (a lot) response continues

Discussion

- Interesting to interpret!
- What we see is a large cycle that led to damage and heavily degraded stiffness
- Response still dissipating energy, still zero centered (not drifting away)
- Base shear high, how to reconcile with pushover?
- Drift levels fairly high ~5%, how to reconcile with pushover?
- Dig deeper at peak force and peak drift to learn more

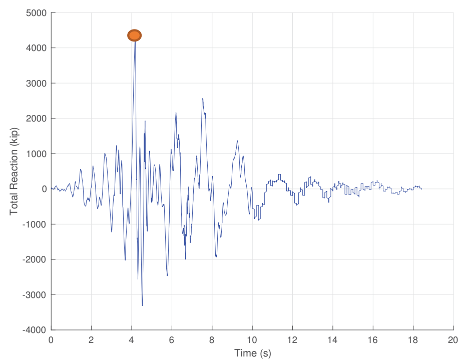
A1: N-S SF2.25 EQ4 at Peak Force





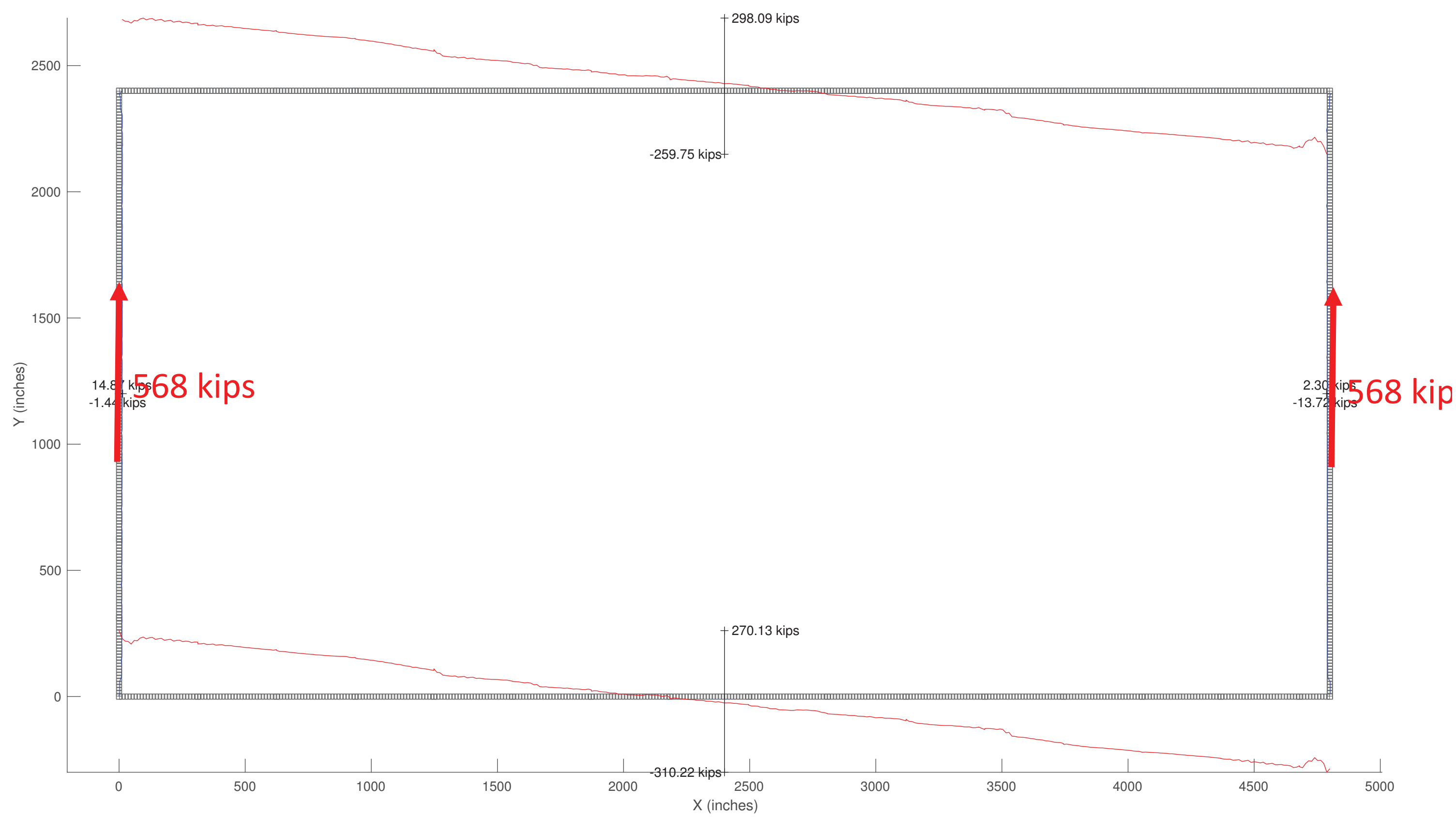
Notes:
Displaced shape modestly indicates stiffness changes at zones.

A1: N-S SF2.25 EQ4 at Peak Force

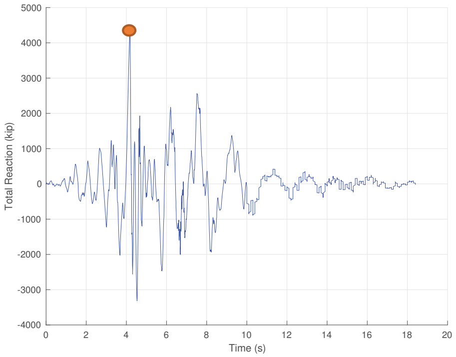


Notes:
Lateral forces at diaphragm level considerably less than at the base.
(568 / 2000=28%)
Lateral forces at diaphragm level still below pushover capacity.
(1136 / 1800=63%)

Sum of wall anchorage forces at diaphragm level (30’ up)

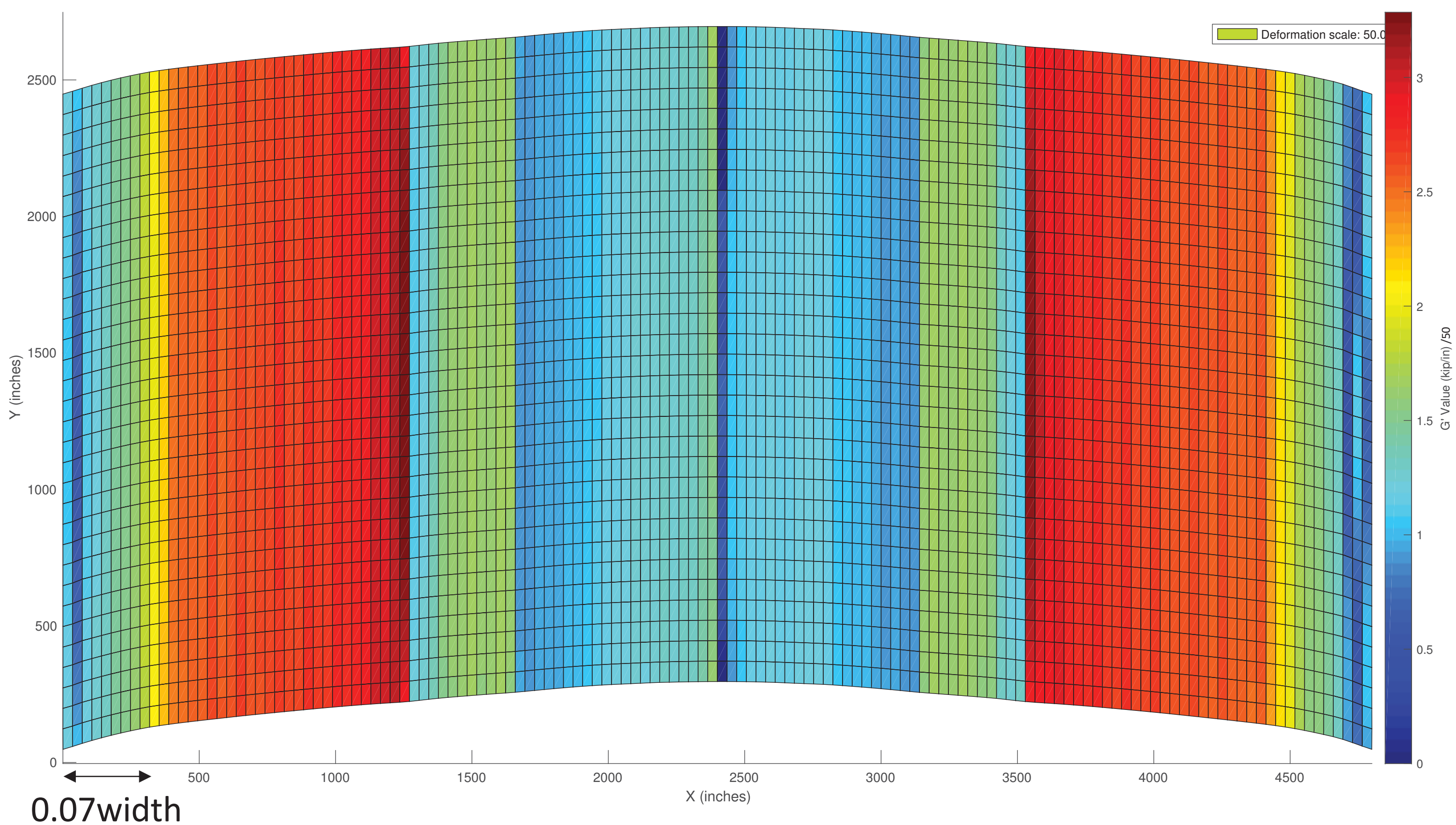


A1: N-S SF2.25 EQ4 at Peak Force

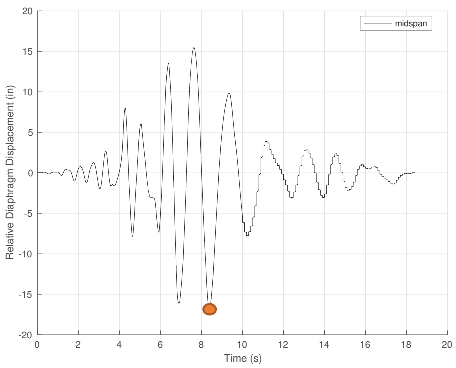


Notes:
Diaphragm at edge has experienced significant stiffness reduction. About 0.07 x width of the diaphragm is experiencing reductions consistent with peak capacities being reached. Plot can be improved to show state of roof panel.

G' contour

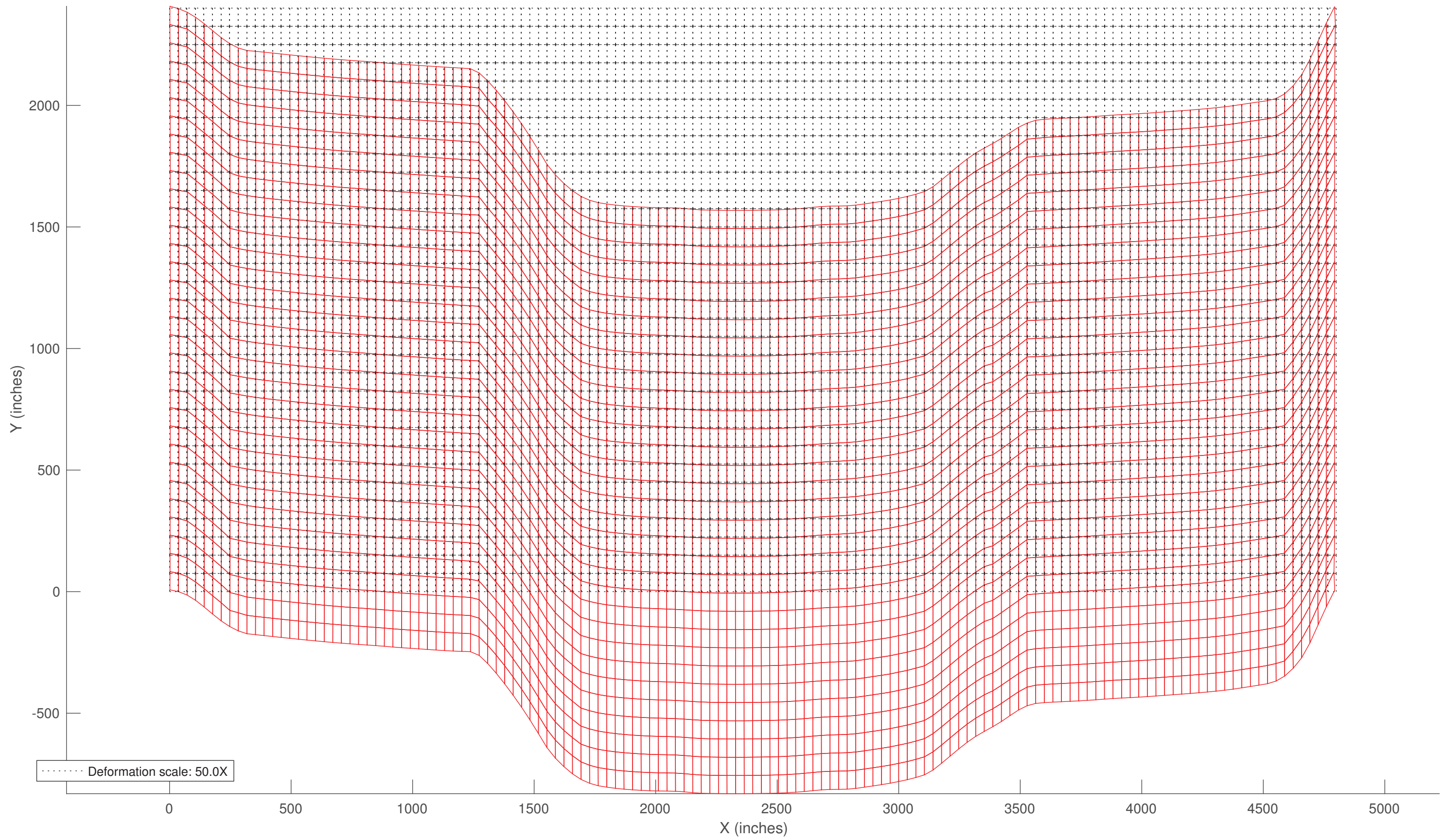


A1: N-S SF2.25 EQ4 at Peak Drift

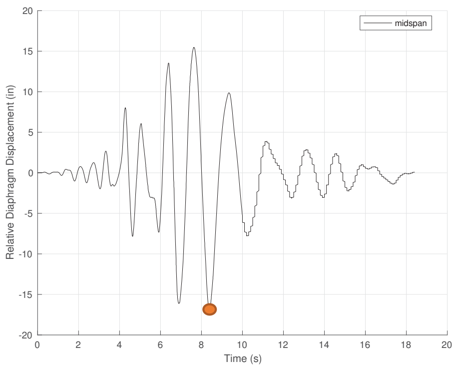


Notes:
Displaced shape is a series of smaller cantilevers from zone to zone..

Magnified Roof Displaced Shape

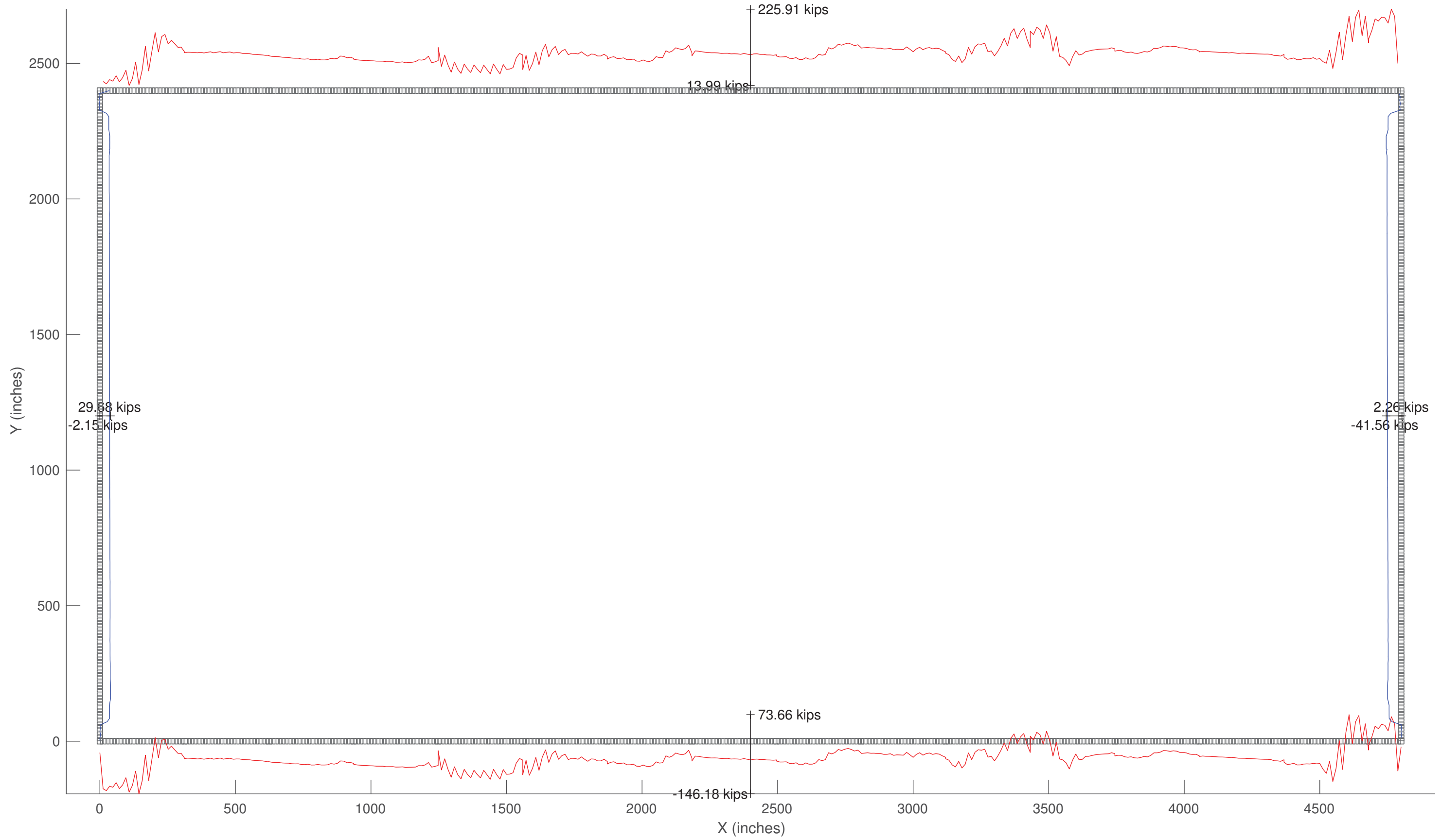


A1: N-S SF2.25 EQ4 at Peak Drift



Notes:
Beam analogy no longer holds for understanding relation between shear in diaphragm and its anchorage.

Sum of wall anchorage forces at diaphragm level (30' up)



A1: N-S SF2.25 EQ4 at Peak Drift

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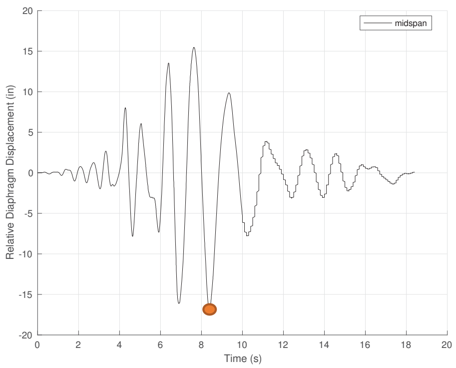
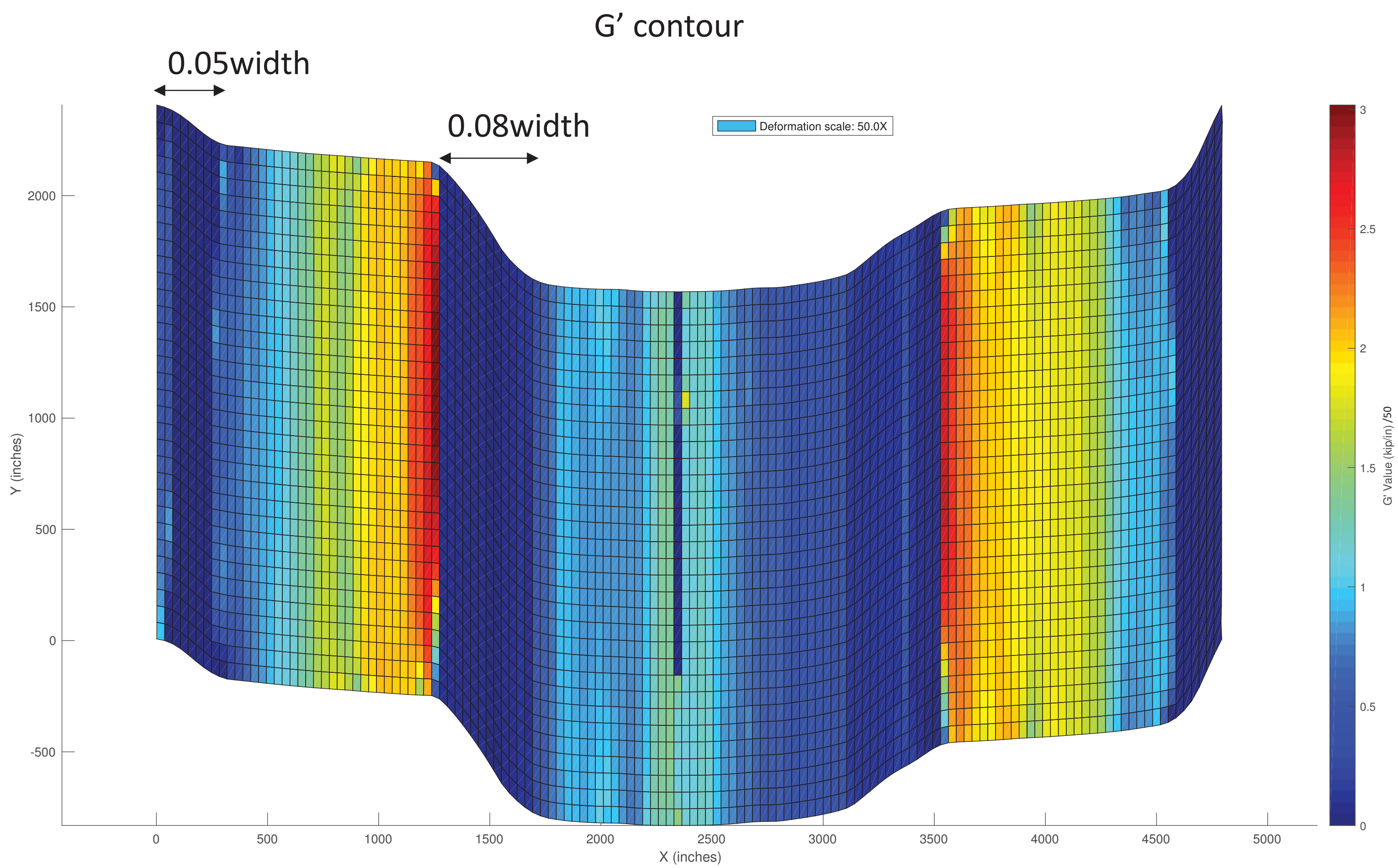
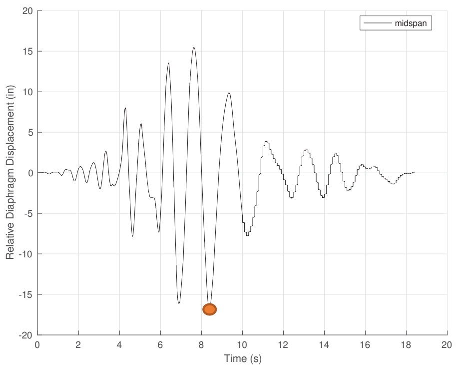


Notes:
Diaphragm edge and zone boundaries experience high shear strains. Length of "plastic" zone reduced for edge, but 2nd zone created at zone transition.
(Width ~ joist girder spans… in this case)

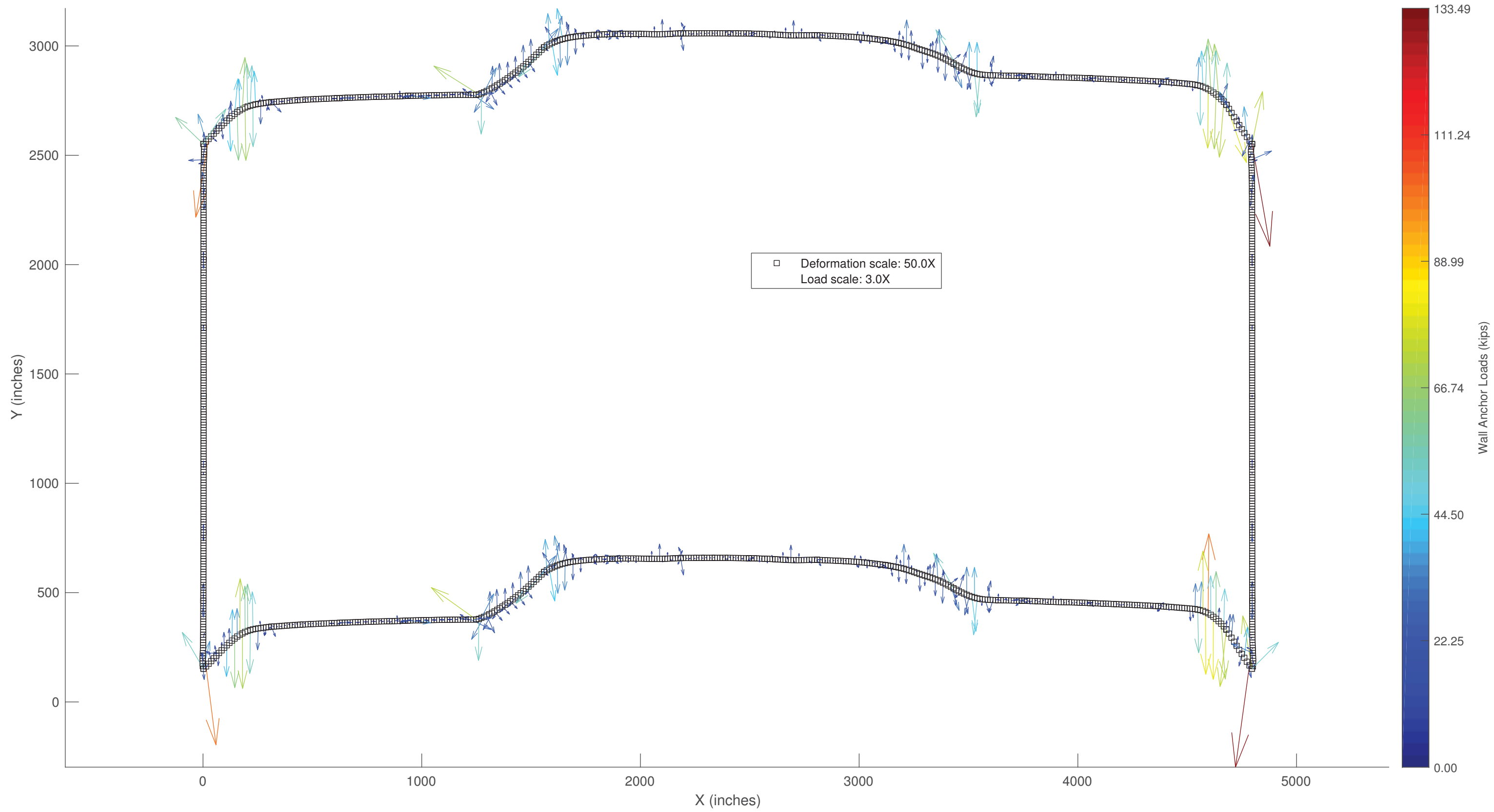


A1: N-S SF2.25 EQ4 at Peak Drift

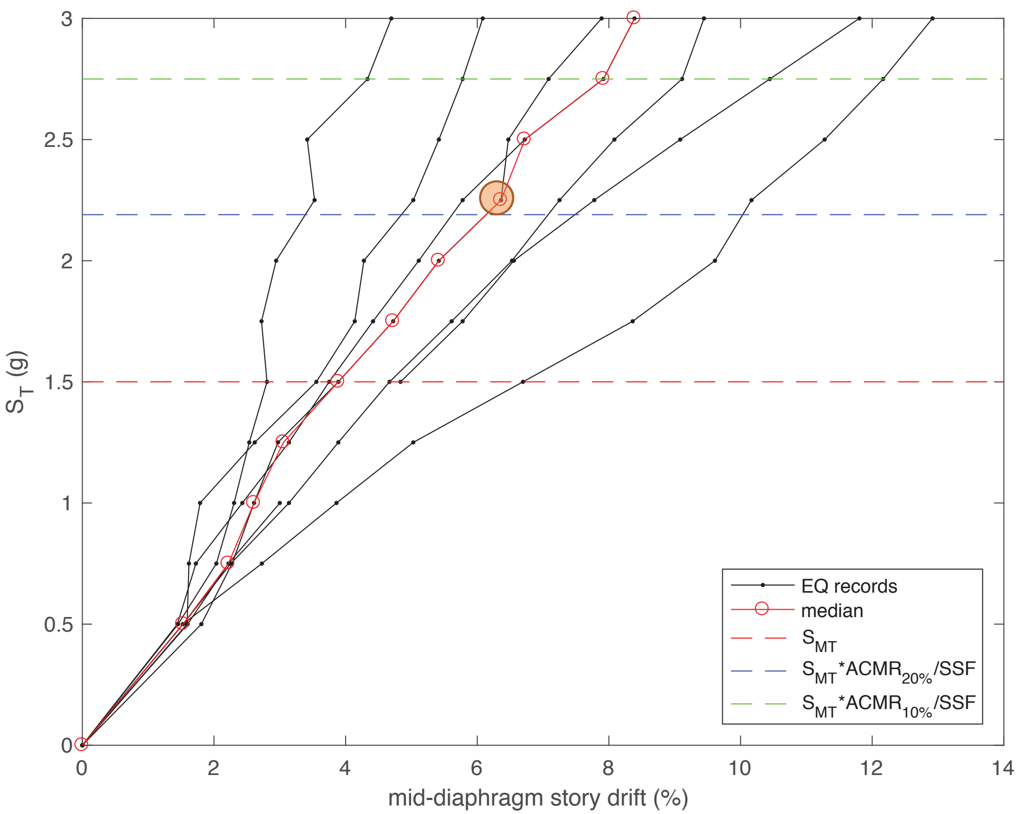


Notes:
Wall anchorage forces are greatest in "plastic" zones. Can dig deeper into magnitudes, but note the total developed diaphragm force is relatively low.

Wall anchorage forces at diaphragm level (30' up)



What about P695 of Archetype 1 in NS?



(This drift is actually total drift. Needs small correction as right now base drift is included here. TBD, does not change overall picture..)

- The analysis indicates the building would pass P695 at $ACMR_{20\%}$ and $ACMR_{10\%}$ under a traditional ASCE 7 based design, with some caveats
- The post-peak roof response is relied on significantly, and needs to be understood a bit more fully and role of secondary systems teased out for full understanding
- Anchorage forces can be large in regions of high shear in the roof and may need to be handled. Certainly need further quantification
- We would need to agree on some of the input factors for the P695 uncertainties and the use of the reduced 7 EQ suite as done here

What do we conclude N-S IDA

- Based on this model the building has not collapsed
- At these high levels of excitation significant portions of the diaphragm are permanently damaged
- Inelastic diaphragm response is beneficial for diaphragm forces, but does not guarantee that the vertical system sees small forces - in this case because so much of the mass is in the walls not the roof
- Wall anchorage forces increase in portions of roof that experience damage/large strain demands
- Zone transitions can be as influential as the edge of the diaphragm and do provide potential for some ”spread of inelasticity”
- Secondary (catenary) action in the chord and roof framing are playing a role in the response, this may need further characterization to understand this influence more completely

Next steps and discussion

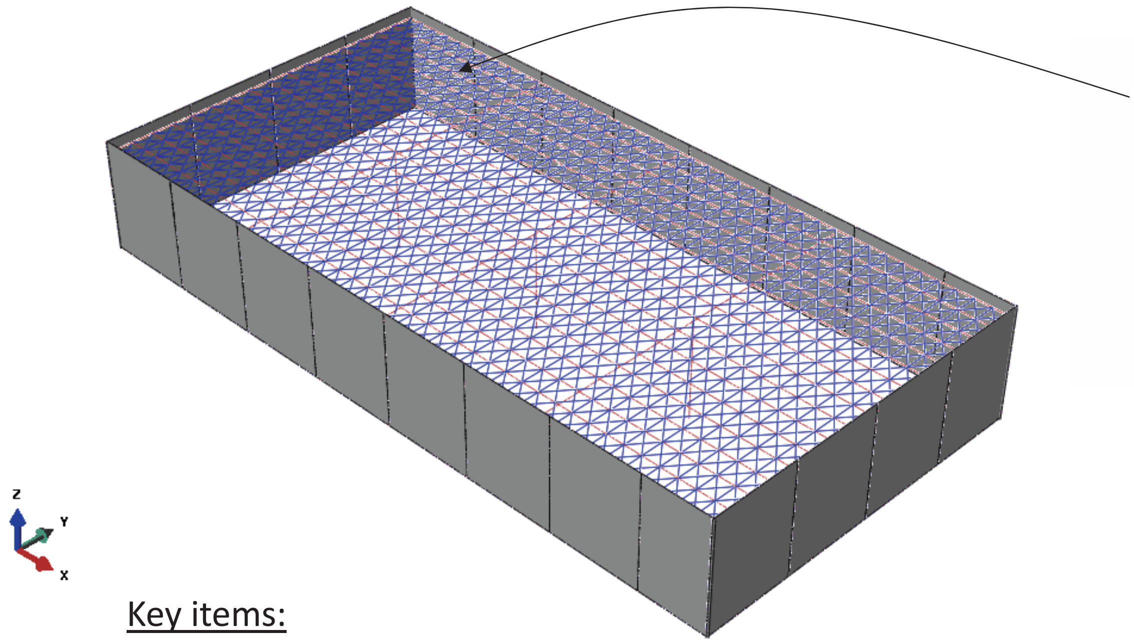
RWFD Steel Next Steps

- Based on ATC-135 and ISG input, we will consider the same archetype but with a re-designed roof using welds for the structural connectors and nestable deck connected by button-punches.
  - This provides a deck with more historical performance record
  - This roof, at least as the connector level, has significantly less ductility than the PAF/screw so provides something akin to a lowerbound
  - This requires more cyclic testing, one of the industry partners has funded this testing, it will initiate immediately
- We are exploring anchorage forces and other details from the models for comparison with existing and proposed provisions
- Industry is interested in pursuing a solution with more controlled/known yielding locations in the roof - this is an anticipated future path
- Work is active, but with limited resources in this year, working with researchers from FEMA P-1026 team and industry input we intend to provide guidance in this code cycle w.r.t. RWFD for steel and are actively participating in BSSC IT9 activities.

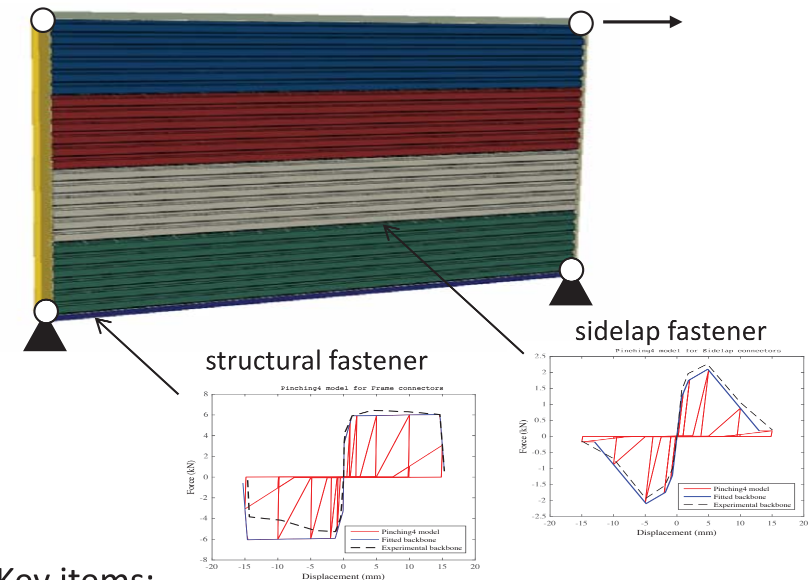
Supplemental

Test Data Feeds Building Models

Building Model



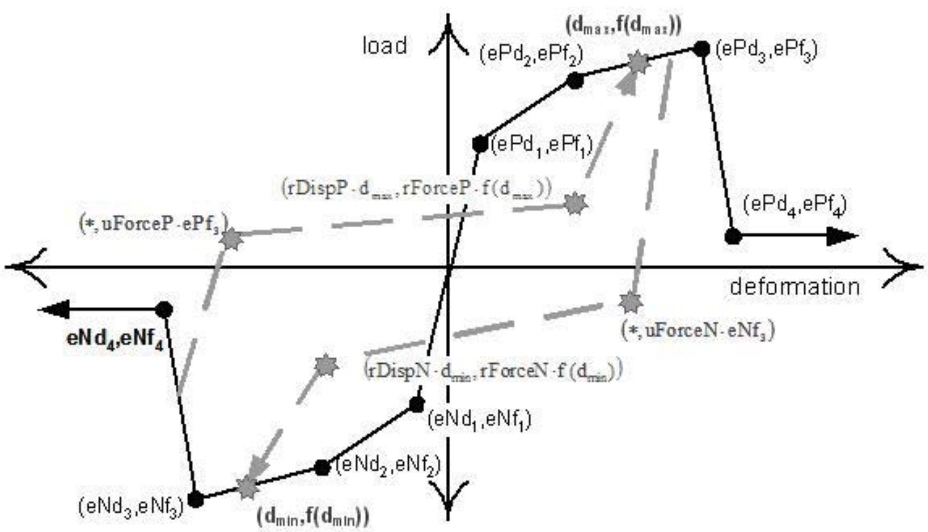
Roof Submodel



Key items:
Nonlinear “trusses” for roof
Secondary framing modeled
Elastic shells for walls
Follows building geometry

Key items:
Deck deformation and yielding in shell elements
Fastener nonlinearity from small-scale testing
Submodel follows fastener spacing in roof
V-Δ behavior converted into nonlinear “truss”

Note on hysteretic characterization

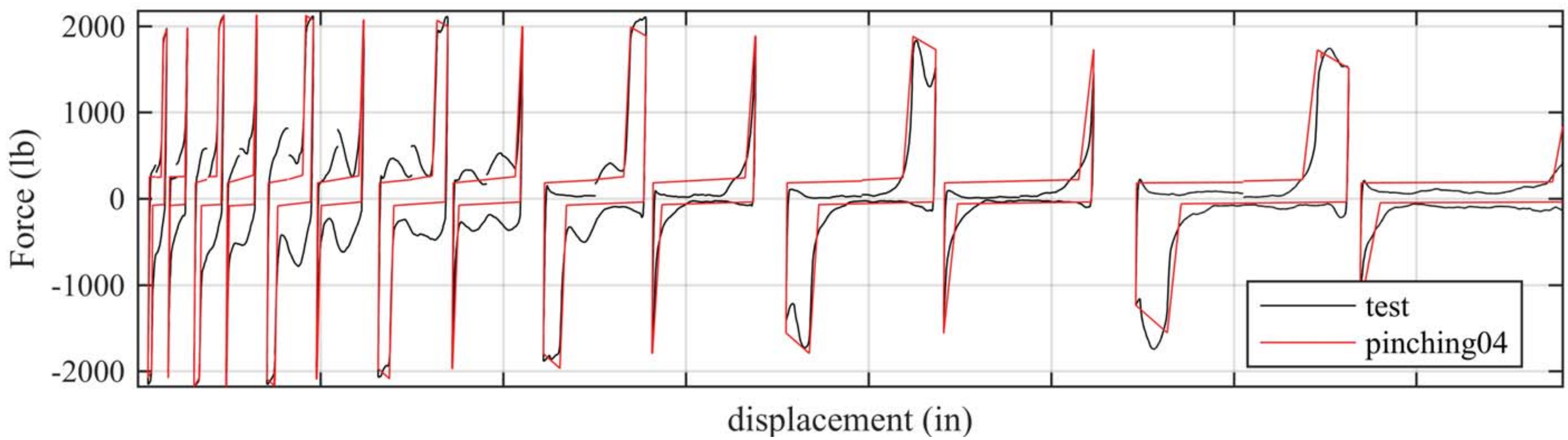


4 point backbone
capable of capturing pinching
And cyclic degradation

- Pinching04 is a nonlinear 1D model developed by Altoontash (2004) and Lowes et al. (2004) for OpenSees and ported to ABAQUS as a UEL spring by Ding (2015)
- Fastener/Deck testing in cyclic shear are fit to the Pinching04 model and then employed in the shell finite roof submodel simulations as fasteners
- Roof submodel simulations are also fit to this model and then employed in the nonlinear roof "trusses" of the full building model

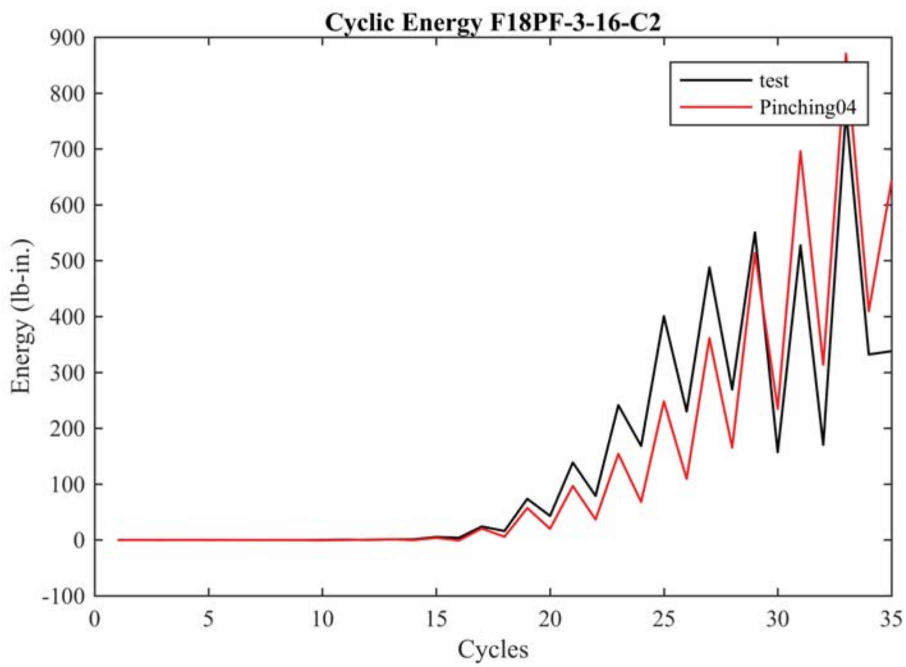
(A-3) Fitting pinching04 model to test results

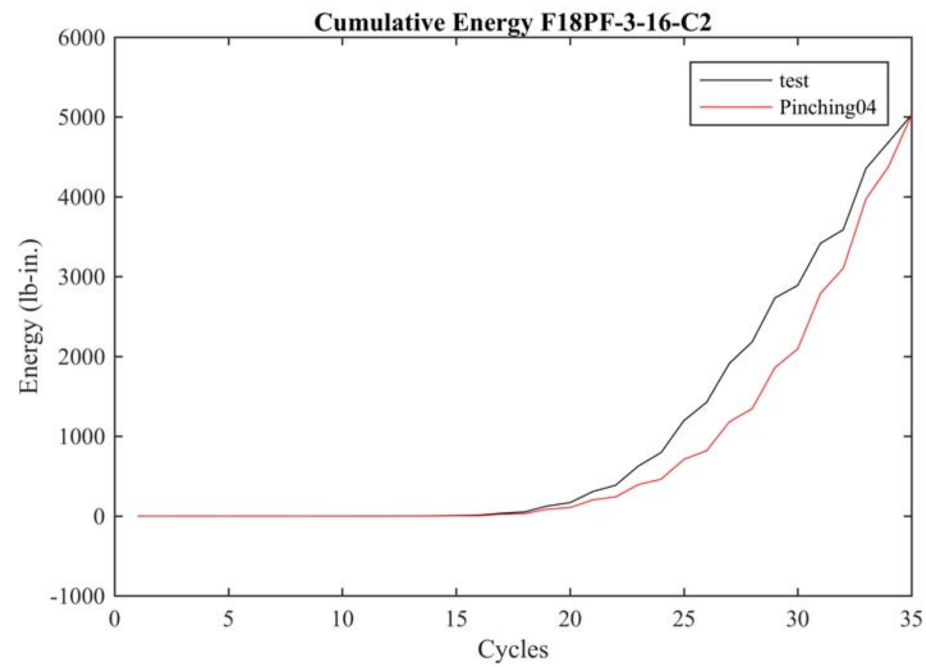
Per-cycle model fit:



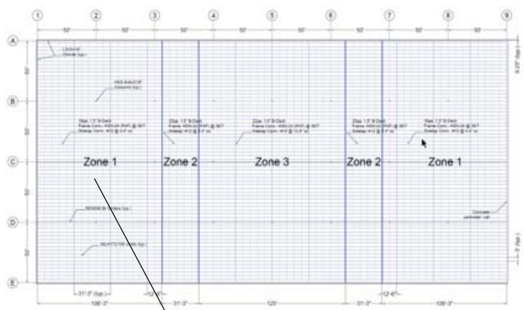
(A-3) Fitting pinching04 model to test results

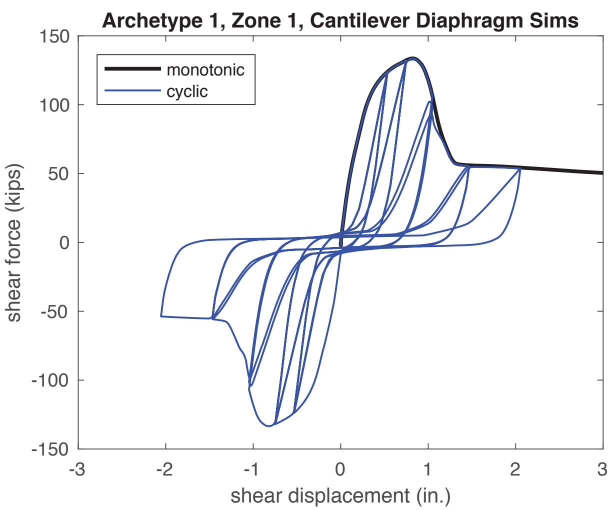
Energy balance



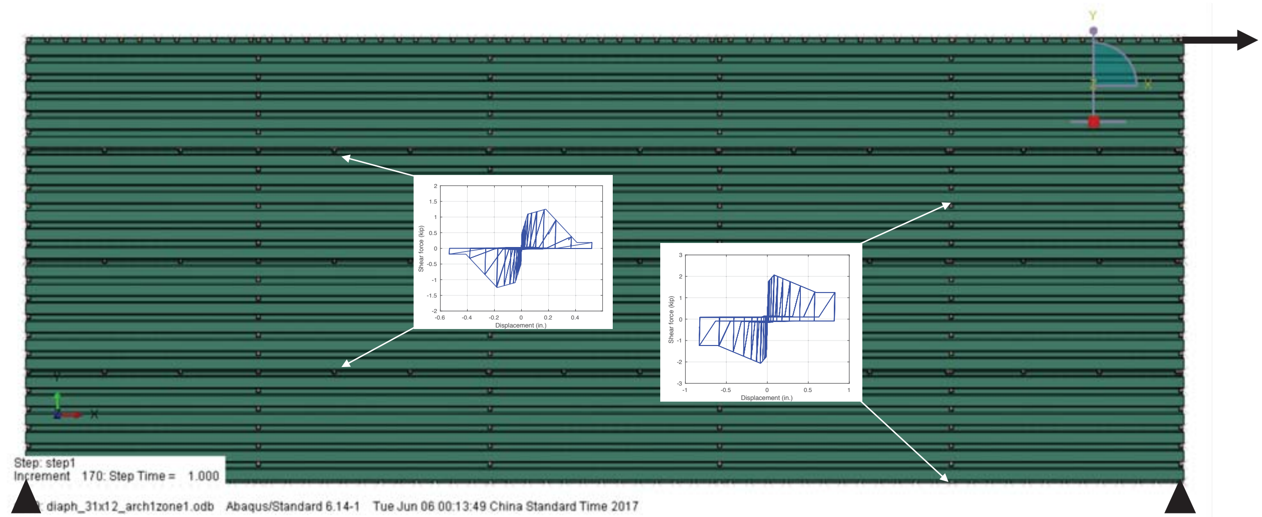


A1Z1: Roof submodel performance



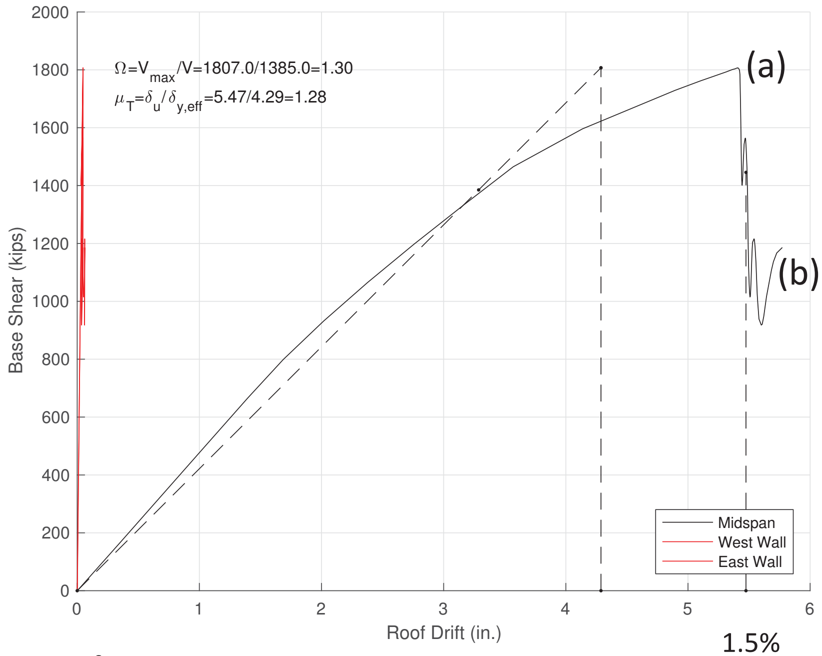


Nonlinear shell FE model of roof segment

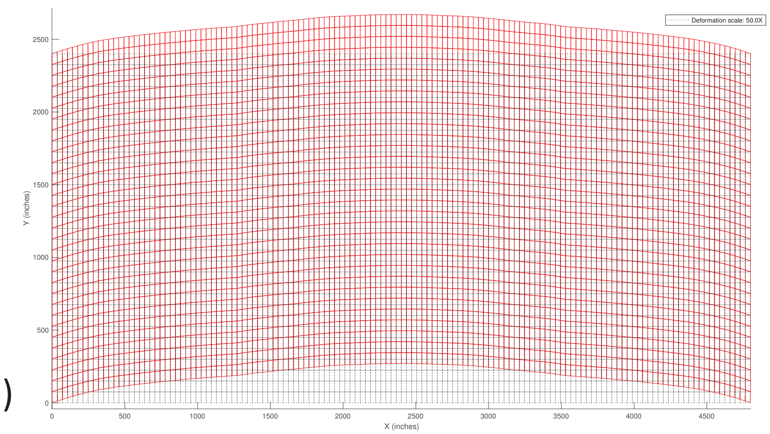


Nonlinear hysteretic springs for connectors called out

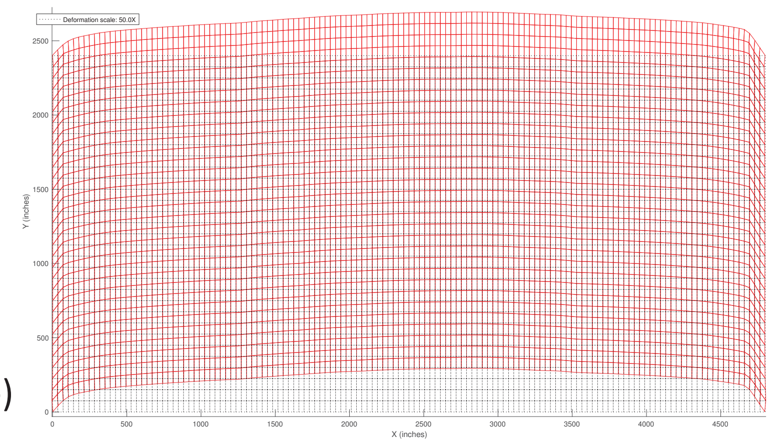
A1: N-S Pushover and Discussion



(a)



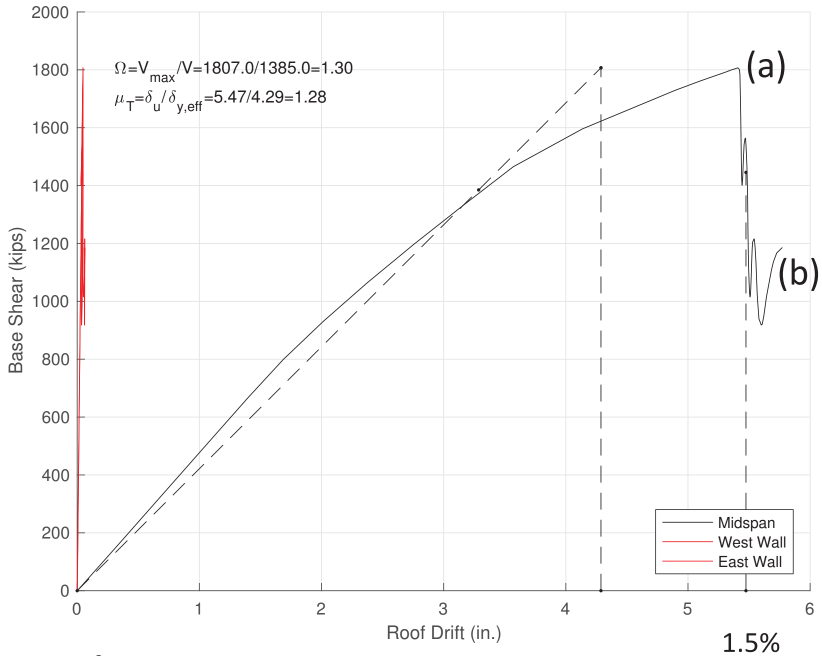
(b)



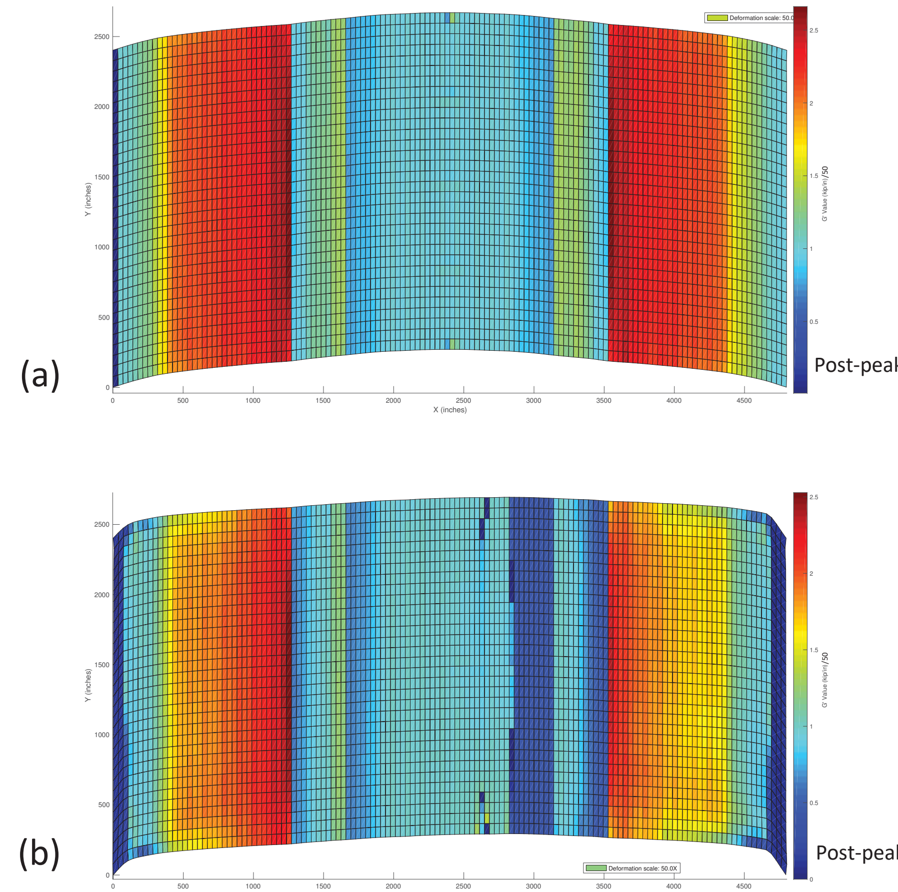
Discussion

- Loading is line load at the edge, could modify
- Capacity OK, but only 1.5% drift & large force drop
- Previous runs show response continues past (b) this may be needed/helpful for interpreting later results
- Note, base shear at first drop is ~1800 kips.

A1: N-S Pushover and Discussion (2)



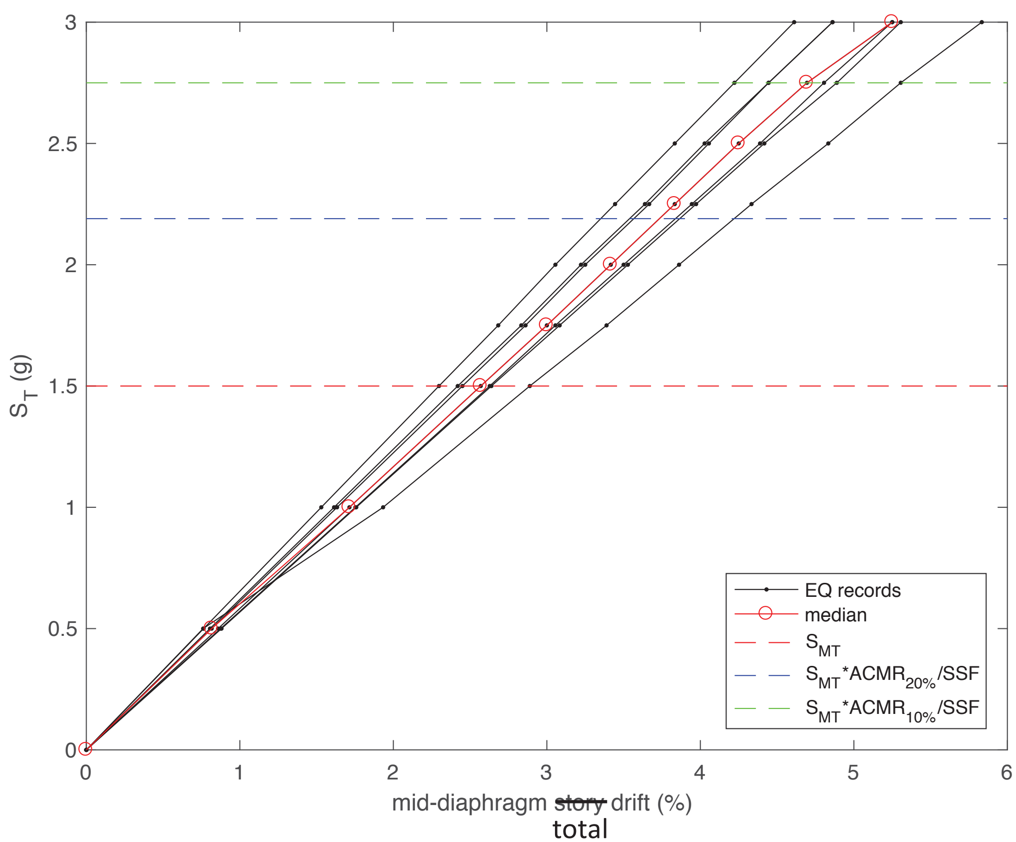
G’ contours



Discussion

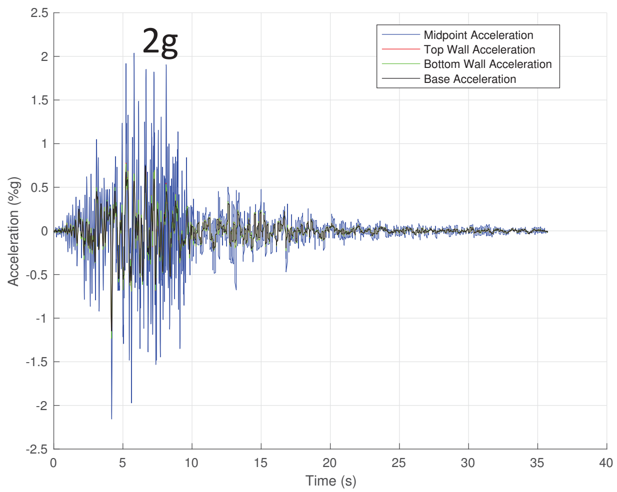
- Examining the failure is challenging, on the right we look at the contour of the roof panel stiffness G’
- Dark blue are portions of the roof that are in the post-peak regime, which happens at edge and at zone boundaries after (a)..

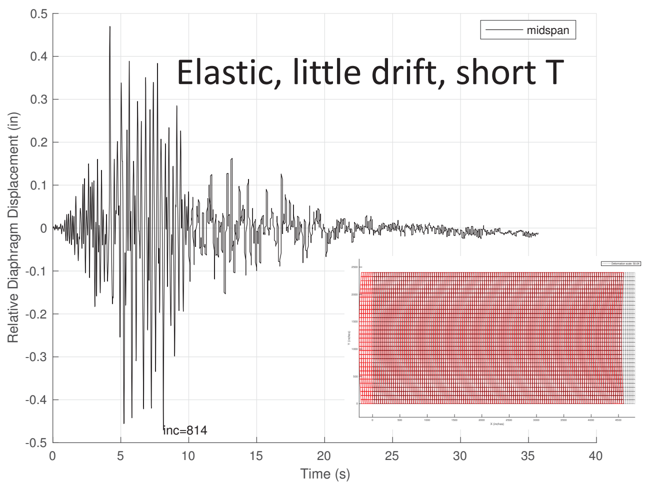
And in the EW direction?

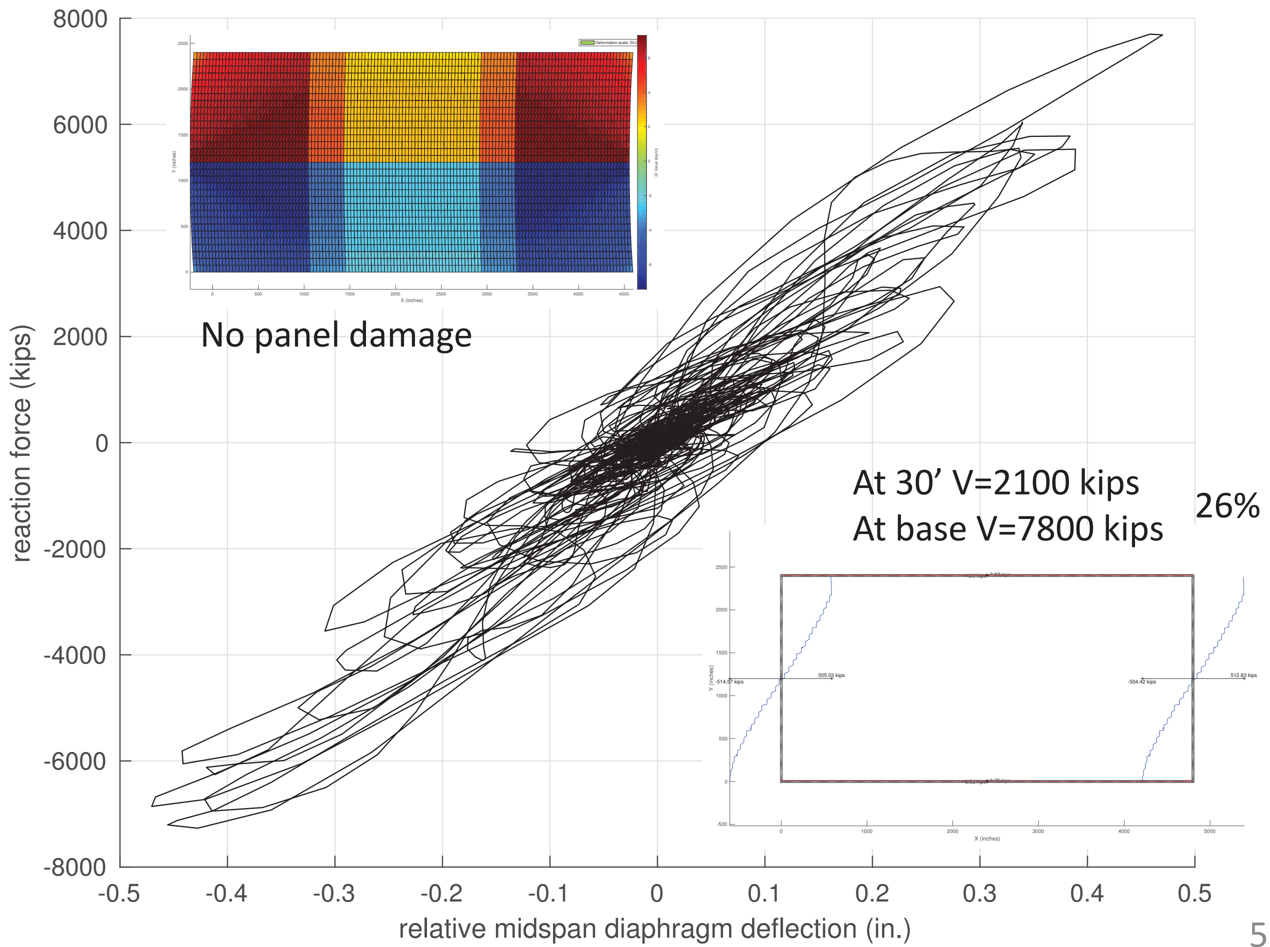


- Same process, ran 44 EQ at $0.5S_T$. Picked median 7. Ran those 7 up through $3.25S_T$.
- You can see the response is largely linear still at these levels of excitation
- Focusing on median EQ at $2.25S_T$.
- Building engages small effective mass in the EW direction

A1: E-W SF2.25 EQ14







What do we conclude E-W IDA

- Short direction response under current design, in this example, is providing essentially elastic performance
- Further study of short direction not likely to be beneficial
- Grouping short and long direction in an archetype family certainly benefits the average archetype response