COLD-FORMED STEEL RESEARCH CONSORTIUM

Advancing seismic design for bare steel deck diaphragms

BSSC PUC Meting

San Francisco, CA

5 December 2018

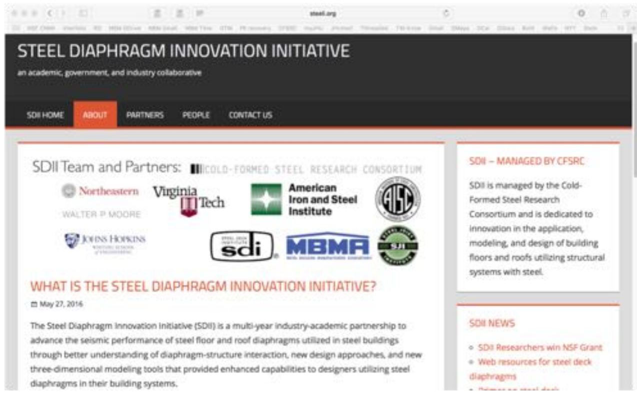
B.W. Schafer

Steel Diaphragm Research

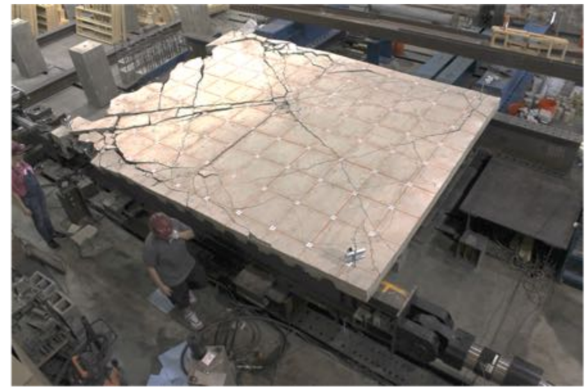
Context for current steel deck diaphragm research and innovation

Steel Diaphragm Innovation Initiative

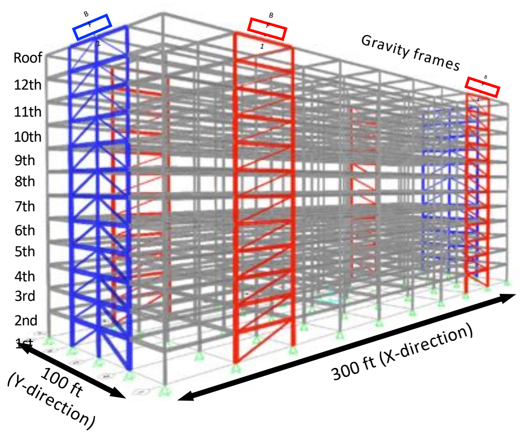
- Innovation and Practice



- Experiments



- Modeling



- Workshop

10 January 2019
Hilton San Francisco Airport Bayfront

Steel - Rigid Wall Flexible Diaphragm Effort

Building upon:

No definitive recommendations for steel deck diaphragms

NBM Technologies (2016-2017)

- Connector Testing
  - Weld, PAF, screw
- Diaphragm and Building Modeling
  - One 3D archetype, two directions, 7 records

Cold-Formed Steel Res. Consortium (2018)

- Expanded Building Modeling
  - Two 3D archetypes, 44 records
- Assessment of Performance
  - Detailed roof ductility demands
- Coordination with Standards Committees

Acknowledgments

- SDII

  CFSRC — Johns Hopkins Whiting School of Engineering, Virginia Tech, Northeastern

  American Iron and Steel Institute, AISC, SDI Steel Deck Institute, SJI Steel Joist Institute, MBMA Metal Building Manufacturers Association, NSF

- RWFD

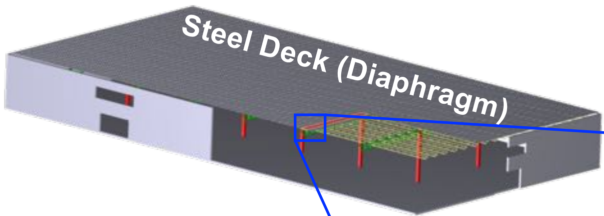
  CFSRC | NBM Technologies, Inc. | American Iron and Steel Institute, SJI Steel Joist Institute, SDI Steel Deck Institute

- Standards and related committees and their participants:
  AISI Sub 31 and Lateral / AISI S310 and AISI S400; AISC TC7 TG on Diaphragms / AISC 342, BSSC IT9 and ATC 135 / NEHRP
- Foundational research: recent work on bare steel deck diaphragms: Tremblay, Rogers et al.; recent work on RWFD: Lawson, Kelly, Filiatrault, and Koliou; recent work on alternative diaphragm design by Restrepo, Fleischman et al.; more
- Numerous research collaborators and students, especially SDII team, NBM RWFD team, and all of Thin-walled Structures Group students

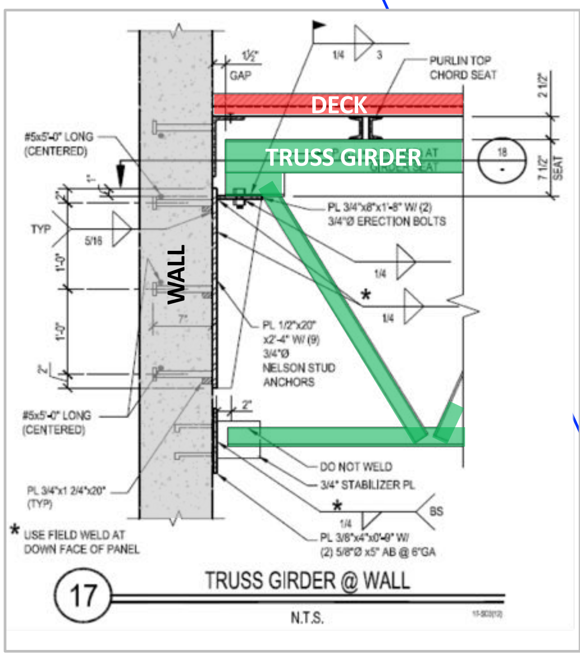
Bare Steel Deck Diaphragms

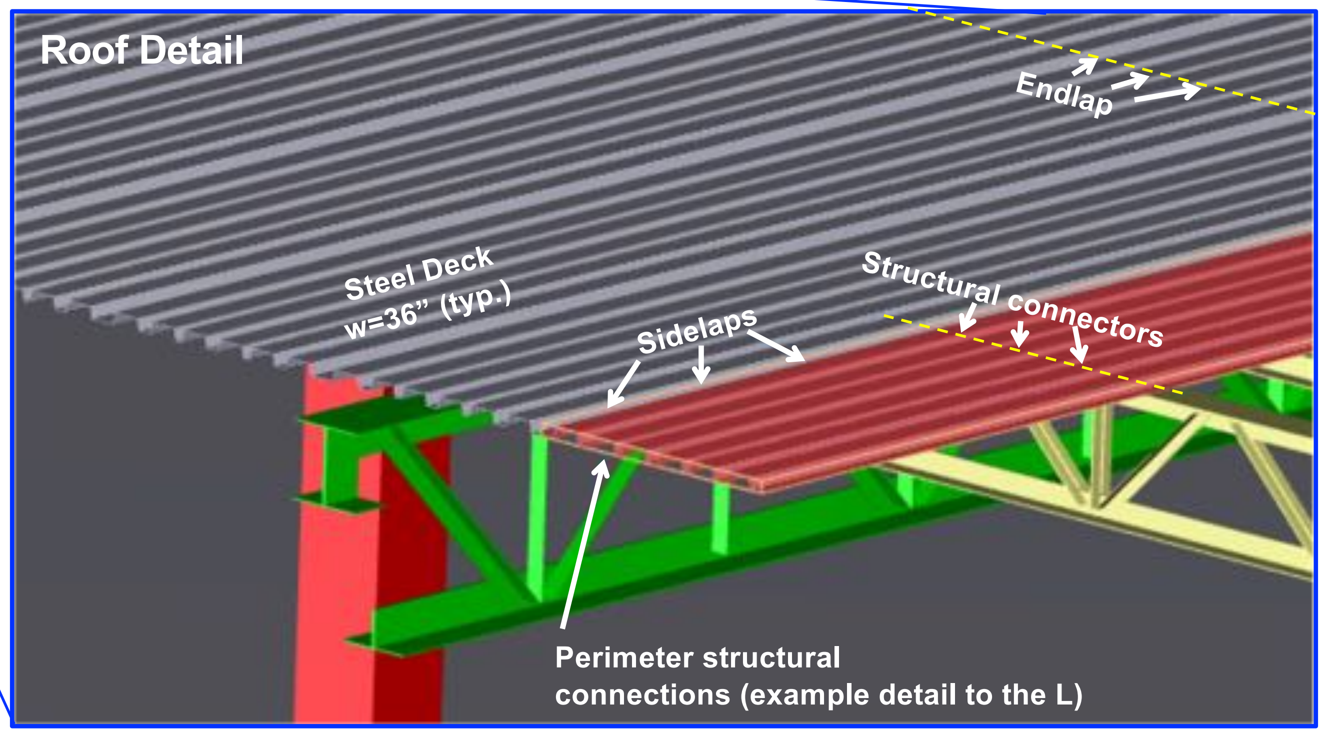
Typical systems and current design

Steel deck diaphragm nomenclature and features



Example
Perimeter Detail





source: Schafer et al. (2018) concept drawing / Verco (2018) private comm. for perimeter detail

Diaphragm Stiffness AISI S310/DDM 04

- By Calculation AISI S310-16 D5

$$G' = \left(\frac{Et}{2(1+\nu)\frac{s}{d} + \gamma_c D_n + C}\right)$$

profile shear   profile warping   connector flexibility

$$C = \left(\frac{Et}{w}\right)\left(\frac{2L}{2\alpha_3 + n_p\alpha_4 + 2n_s S_f/S_s}\right)S_f$$

exterior structural
interior structural
sidelap
sidelap flexibility
frame (structural) flexibility

takeaway: $G' = f(t, \text{profile shape,connectors})$

- By Testing using AISI S310-16 E

AISI S310 Chapter E prescribes how to employ AISI S907-13 to experimentally determine G'

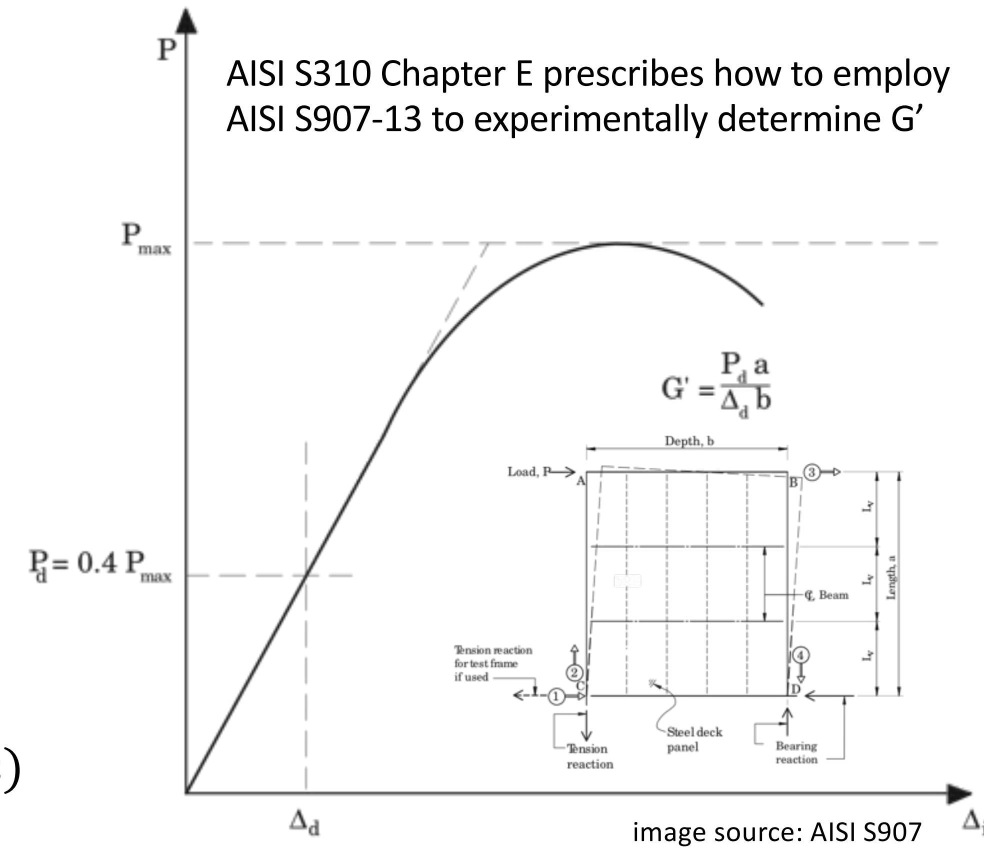


image source: AISI S907

source: AISI S310 (2016) / SDI DDM04 (2015) / AISI S907 (2013)

Diaphragm Strength AISI S310/DDM 04

- By Calculation AISI S310-16 D1

$$\phi S_n = min(\phi S_{nf}, \phi S_{nb})$$

Fastener failure — Panel buckling

$$S_{nf} = min(S_{ni}, S_{nc}, S_{ne})$$

$$\underbrace{\text{interior} \quad \text{corner} \quad \text{exterior}}_{f(P_{nf}, P_{ns}, \text{spacing})}$$

frame (structural) strength — sidelap strength

takeaway: $P_n = f(\text{connector}) or f(\text{profile})$

- By Testing using AISI S310-16 E

$$P_n = P_{max}$$
$$S_n = P_n/b$$
$$\phi S_n \geq S_r$$

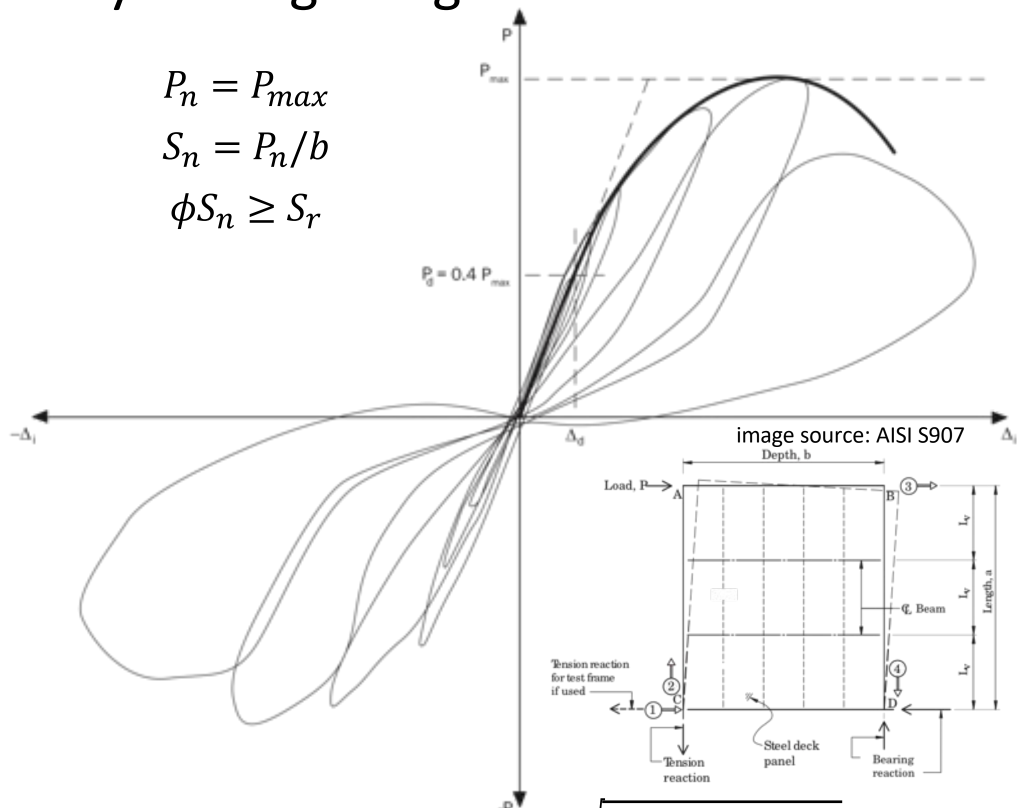


image source: AISI S907

$$\phi = C_\phi M_m F_m P_m e^{-\beta_o \sqrt{V_M^2 + V_F^2 + C_P V_P^2 + V_Q^2}}$$

accounts for variability, sample size, target reliability, etc.

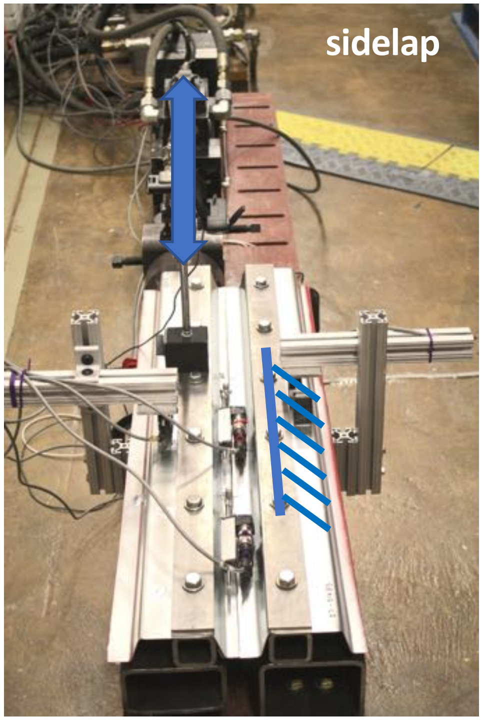
source: AISI S310 (2016) / SDI DDM04 (2015) / AISI S907 (2013)

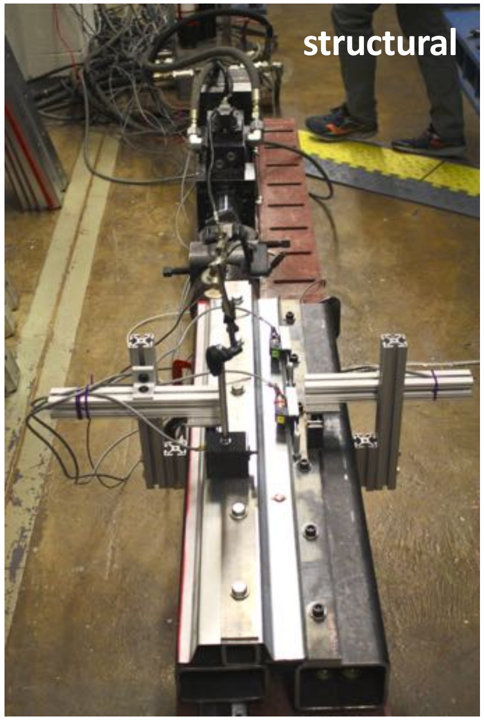
Connector Performance

Testing and performance of sidelap and structural connectors for steel deck diaphragms and potential implications for seismic performance. New testing conducted and reported here due to limitations in existing data.

Cyclic shear deck-connector testing

Test Configuration





AISI S905 test standard

FEMA 461 Protocol 1 Cyclic Profile ($a_{i+1}=1.4a_i$)

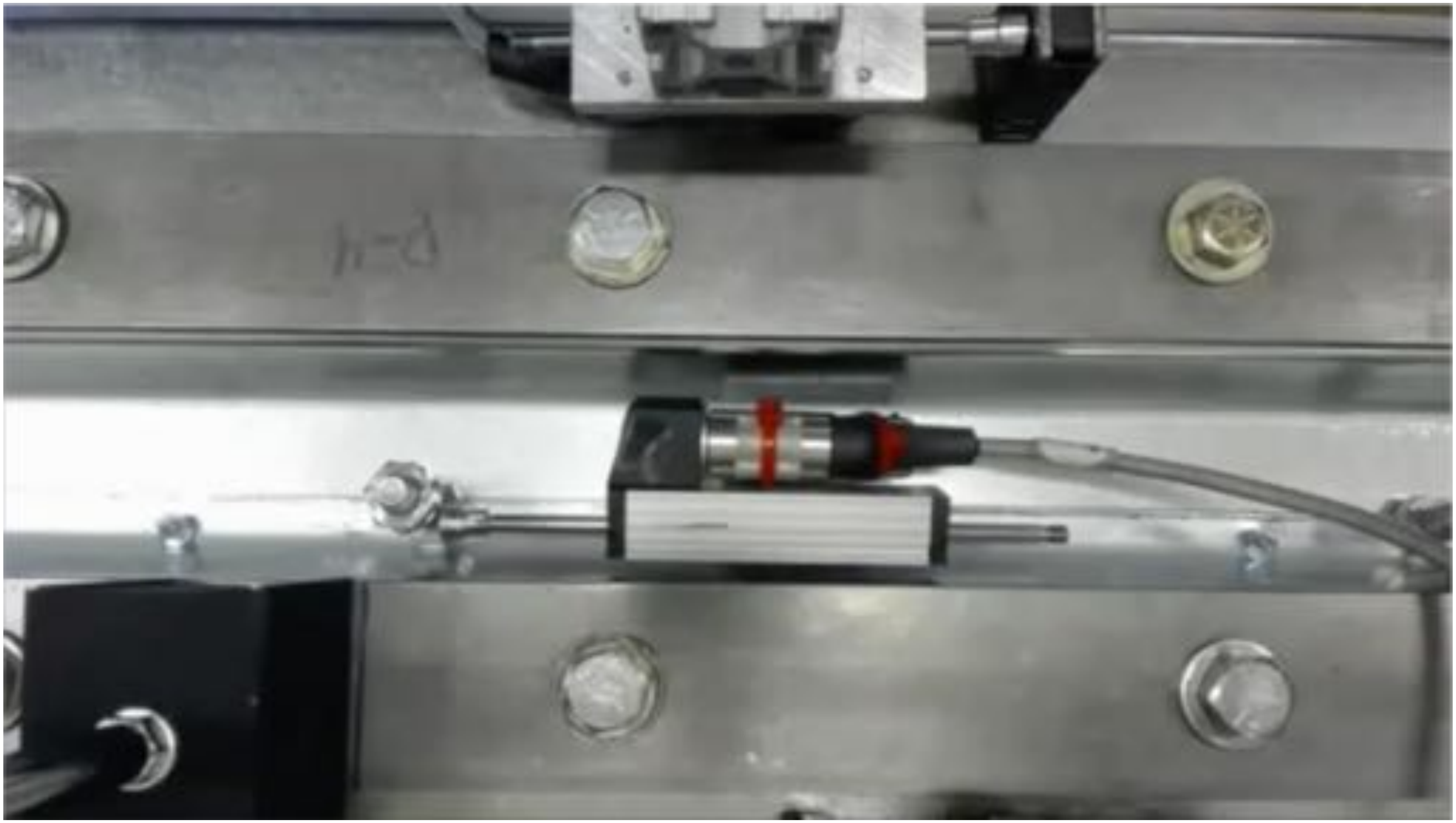
Test Specimens

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.

source: Torabian et al. (2018) / NBM (2017)

Overview of performance



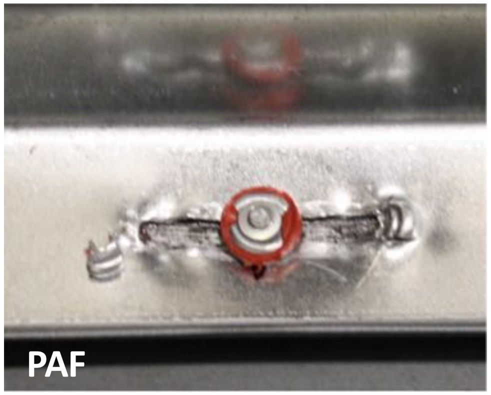
source: Torabian et al. (2018) / NBM (2017)

Visual summary of observed damage

- Sidelap connectors

- Mechanical connectors involve localized deformations and bearing damage with residual capacity if still engaged
- Welds create significant deformations in surrounding deck profile but no residual capacity after fracture

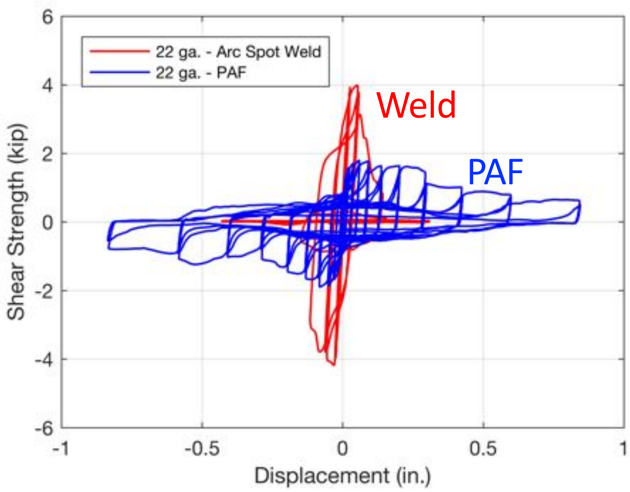
- Structural connectors



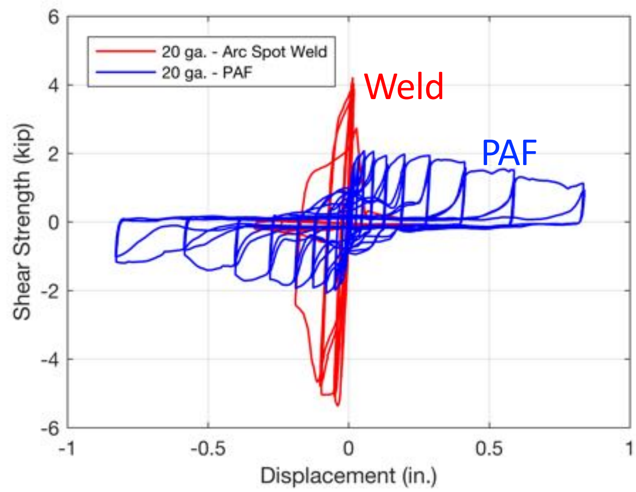
source: Torabian et al. (2018) / NBM (2017)

Arc-Spot Weld vs. PAF Cyclic Structural Conn.

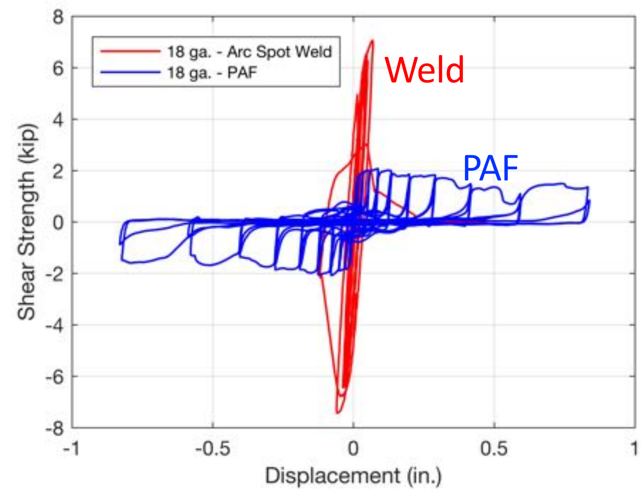
22 gauge



20 gauge



18 gauge



source: NBM (2017) test data – plot original to this presentation

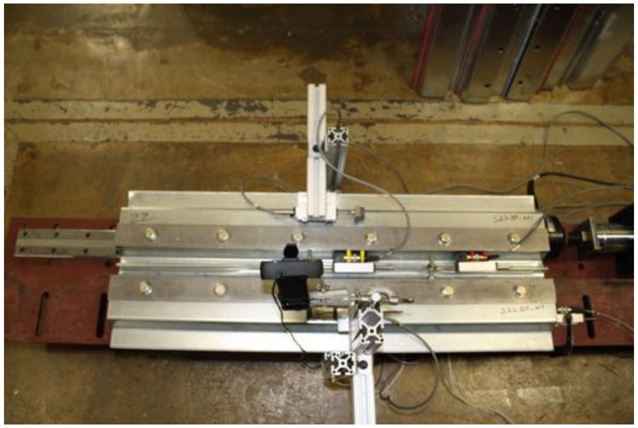
Experimental Connector Ductility

Type	Connector	Deck Gauge	K_i [b]	F_p [b]	δ_{pp80}	μ [a]
			(kip/in.)	(lbf)	(in.)	(-)
Sidelap [d]	Screw [c]	22	59	780	0.303	22.9
		20	60	678	0.145	12.8
		18	135	1251	0.234	25.3
	Top Arc Seam Weld	22	41	2431	0.127	2.1
		20	58	2931	0.118	2.3
		18	102	3638	0.136	3.8
Structural	PAF	22	132	1788	0.231	17.1
		20	174	2041	0.290	24.7
		18	162	2066	0.341	26.7
	Arc Spot	22	168	3993	0.063	2.6
		20	179	4292	0.061	2.5
		18	213	6375	0.068	2.3
	Arc Seam	22	168	4666	0.076	2.7
		20	195	5412	0.082	3.0
		18	221	7669	0.086	2.5

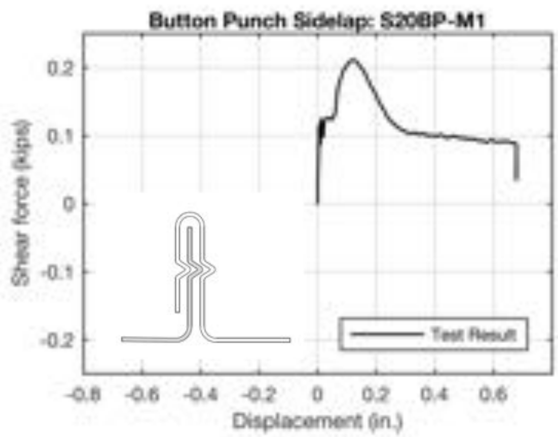
a) $\mu = \delta_{pp80}/(F_p/K_i)$, b) stiffness and strength agree well with AISI S310, see NBM (2017) report for specifics, c) see Torabian et al. 2018b for additional tests on screwed sidelaps, d) see NBM (2018) for tests on button punch sidelaps

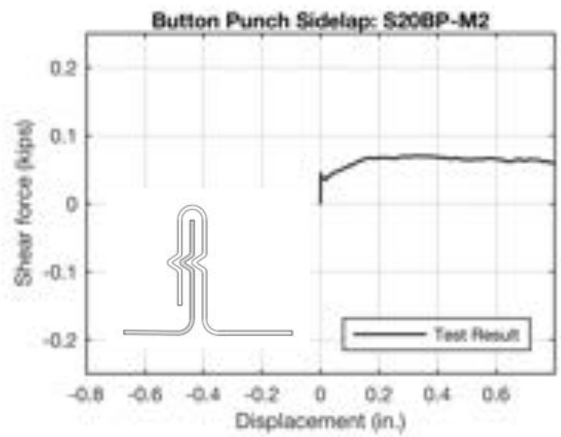
source: NBM (2017) test data – table original to this presentation

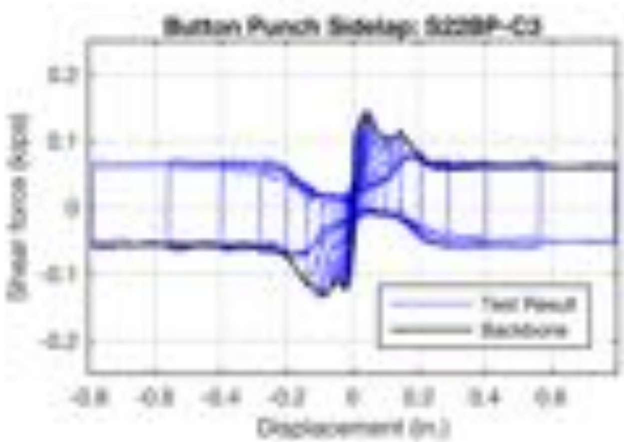
Table note d: Addendum-Button Punch Testing (by NBM)

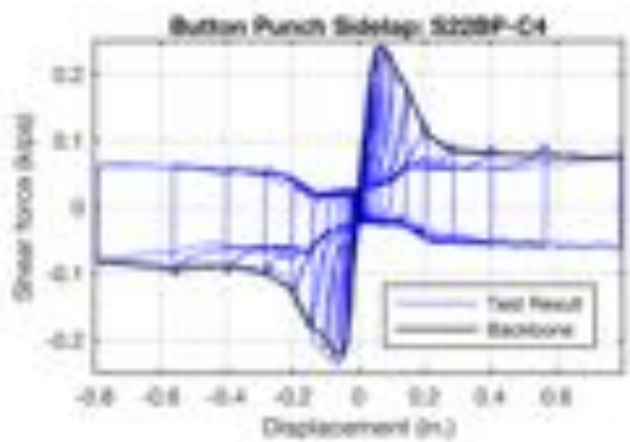


(Preferred configuration as shown in industry literature)









- Small capacity per punch
- Large variation
- Configuration influences
- Friction hysteresis if engaged, but at very small force levels
- Any out-of-plane force readily disengages
- Data used to create spring models for later roof modeling

source: NBM (2018) project for Verco – Verco released to public domain, see references

Cantilever Deck Diaphragm Experimental Performance

Impact of fasteners and other details on ductility performance

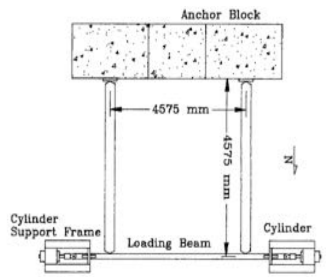
Cantilever Diaphragm Test Database

Overview

Testing Program	# of Specimens
Cornell University, 1950s-1960s	40
S. B. Barnes and Associates, 1950s -1960s	38
West Virginia University, 1960s-70s	246
Development Lab of Inland Ryserson Co.	1
University of Salford, Manchester 1970s-80s	5
ABK, a Joint Venture, California 1980s	3
Iowa State University, 1980s	32
Virginia Tech, 1990s - 2000s	67
Technical Research laboratory in Kobe, Japan, 1990s	6
Nucor – Vulcraft/Verco Group, 1990s-2000s	120
University of Montreal, McGill University, Canada, 2000s	82
Tongji University, China, 2000s	6
Hilti Corporation, Liechtenstein, 2000s-2010s	92
Tokyo Institute of Technology, Japan, 2010s	15
Total:	753

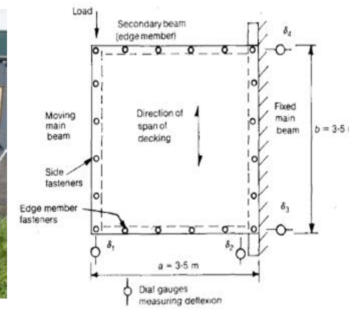
source: O'Brien et al. (2017)

Types of Experimental Studies Included

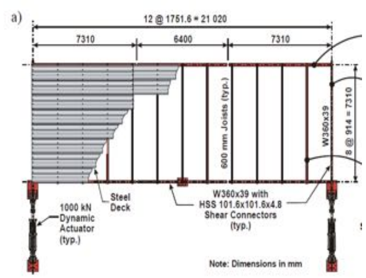


Group from Iowa State in 1980's and 1990's

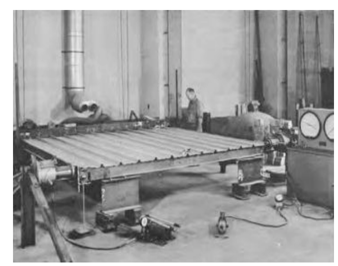
Diaphragm Tests by Industry (e.g. Hilti)



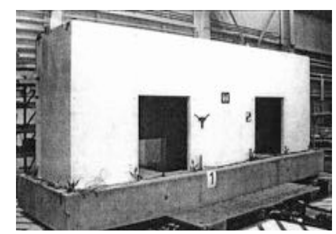
Research from Europe (e.g. Davies and Fisher 1979)



Work by Tremblay and Rogers in Canada



Larry Luttrell's group at West Virginia



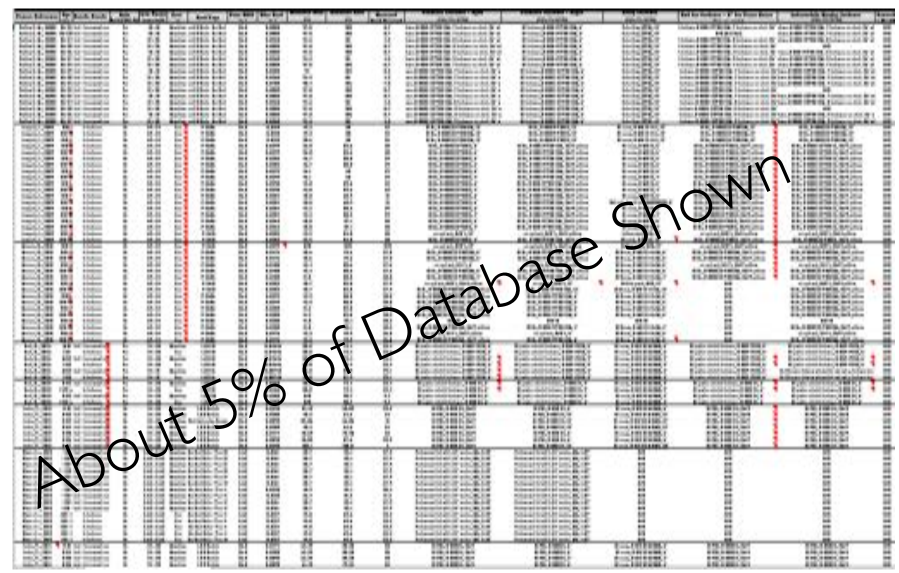
Building Tests (e.g. Cohen et al. 2004)

Cantilever Diaphragm Test Database

Breakdown of database fields:
Test setup fields (26), test result fields (3), calculated fields (11)

Test Setup Fields	Load Type	Measured deck yield strength
	Load protocol	Measured deck percent elongation
	Setup configuration	Type of structural fastener
	Plan dimensions	Size of structural fastener
	Span dimension	Spacing of structural fastener
	Depth dimension	Type of sidelap fastener
	Deck span direction	Size of sidelap fastener
	Deck span length	Spacing of Sidelap Fastener
	Test frame support member sizes	Endlap location
	Test frame interior support member sizes	Concrete unit weight
	Steel deck profile dimensions	Measured concrete fill thickness
	Steel deck manufacturer	28 day concrete compressive strength
	Steel deck thickness	Type of concrete reinforcement
Test Result Fields	Ultimate shear strength	Shear angle at 80% strength degradation
	Shear stiffness	
Calculated Fields	Predicted structural fastener strength	Strength Factors, R_{Ω}
	Predicted sidelap fastener strength	Subassemblage Ductility
	Predicted diaphragm strength	System Ductility
	Predicted structural fastener flexibility	Ductility Factor (medium/long period), R_{μ}
	Predicted sidelap fastener flexibility	Diaphragm Design Force Reduction Factor
	Predicted diaphragm stiffness	(medium and long period), R_s

Available online at:
O'Brien, P., Eatherton, M.R., Easterling, W.S., Schafer, B.W., Hajjar, J.F. (2017) "Steel Deck Diaphragm Test Database v1.0." CFSRC Report R-2017-03, permanent link: jhir.library.jhu.edu/handle/1774.2/40634.



source: O'Brien et al. (2017)

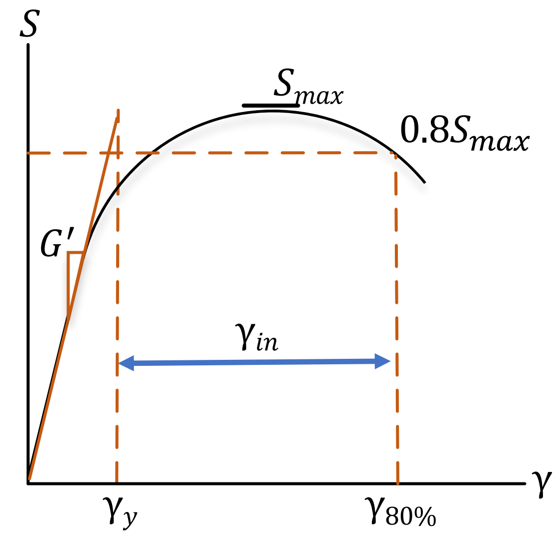
Subsystem Ductility from Database

Summary ductility statistics from O'Brien et al. (2017) database

		Monotonic			Cyclic			
Structural	**Sidelap**	n	μ_m	$\sigma_{\mu m}$	n	μ_c	$\sigma_{\mu c}$	μ_c/μ_m
PAF	Screw	19	**3.6**	1.8	19	**2.9**	1.0	80%
Weld	(all connectors)	28	**3.2**	1.1	8	**1.7**	0.5	
	Button Punch	8	**2.6**	0.4	6	**1.5**	0.4	60%
	Screw	8	**3.4**	1.3	1	**2.0**	-	59%
	Top Arc Seam	7	**3.9**	1.0	1	**2.6**	-	68%
	Seam	5	**3.2**	1.3	-	**-**	-	

n: number of samples, σ: standard deviation, Note Tremblay et al. (2004) has developed a system using spot welds with washers, for structural connections, when welded sidelaps are used this system has moderate ductility and little cyclic degradation. Related data is not included in this table under "weld" since the details are non standard.

$$\mu = \frac{\gamma_{80\%}}{\gamma_y}$$



source: O'Brien et al. (2017), summary calculations original

Monotonic vs. Cyclic and Ductility

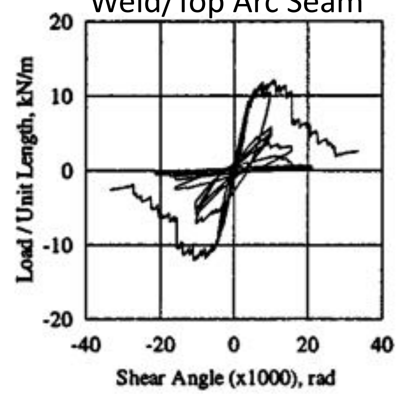
Wider Database Results

Structural	**Sidelap**	μ_c/μ_m
PAF	Screw	80%
Weld	(any connector)	60%
	Button Punch	60%
	Screw	59%
	Top Arc Seam	68%

"categories: **low ductility** (Tests 2 and 14) {Weld/BP and Weld Screw}, **moderate ductility** (Tests 12, 13, 16, and 6) {Weld/Weld, Weld with Washer/Weld, Weld with Washer/Screw, Screw/Screw} and **good ductility** (Tests 7, 8, and 18) {PAF/Screw, PAF/Screw, PAF/Screw}."
Essa et al. (2003) {BWS additions}

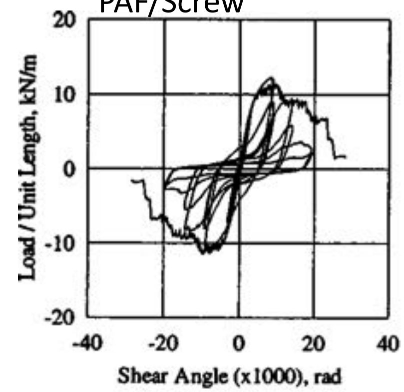
Essa et al. (2003) from original (results in database)

Weld/Top Arc Seam



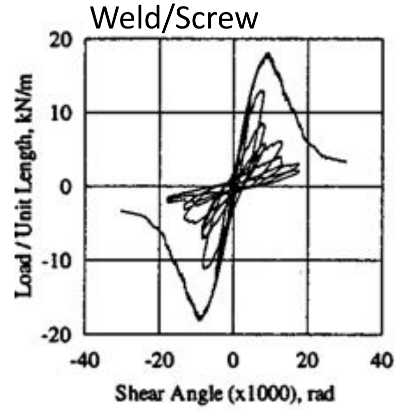
(a)

PAF/Screw

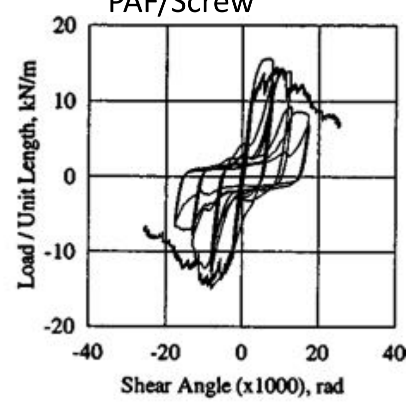


(a)

Weld/Screw



PAF/Screw



source: Essa et al. (2003), O'Brien et al. (2017), summary calculations original

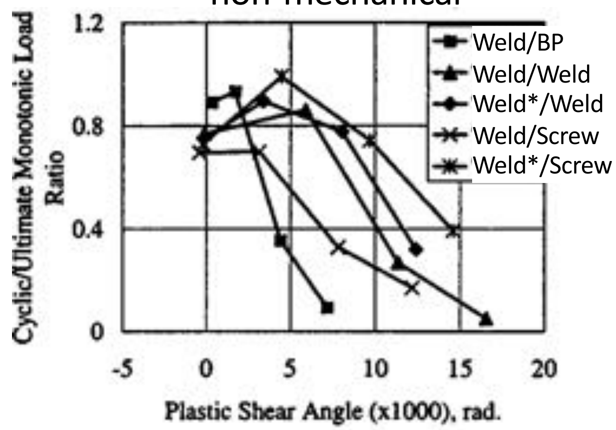
Monotonic vs. Cyclic and Ductility (Cont.)

Wider Database Results

Structural	**Sidelap**	μ_c/μ_m
PAF	Screw	80%
Weld	(any connector)	60%
	Button Punch	60%
	Screw	59%
	Top Arc Seam	68%

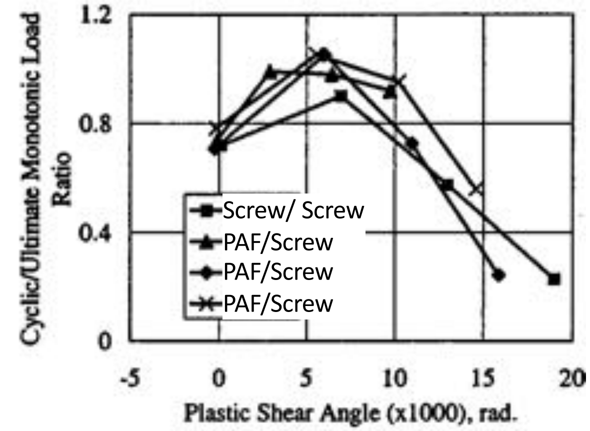
Essa et al. (2003) from original (results in database)

non-mechanical



*denotes weld with washer

mechanical

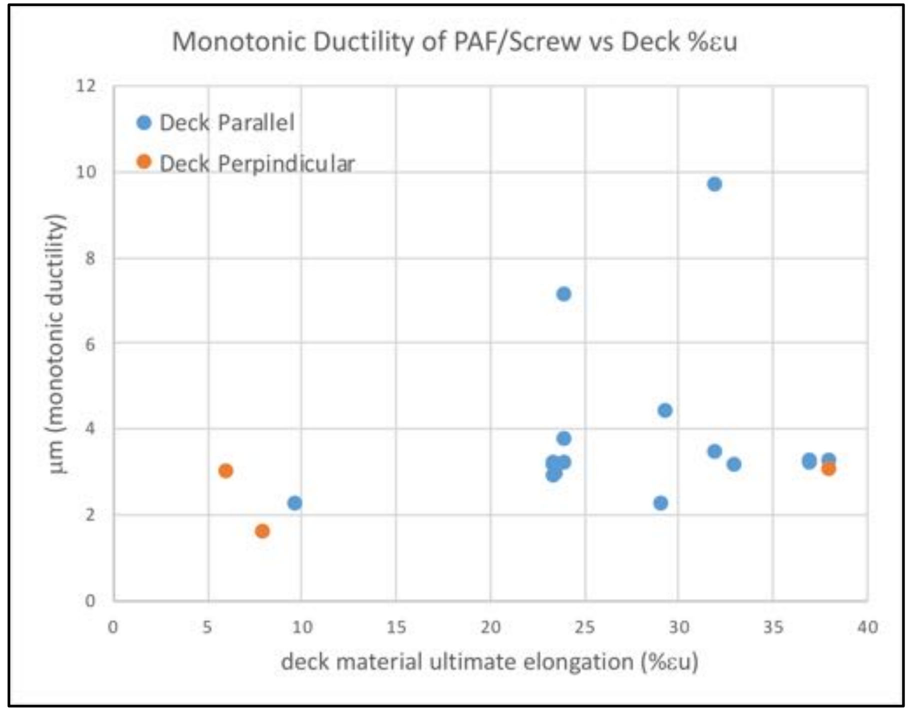


Although some non-mechanical (weld) systems can achieve similar levels of ductility to mechanical systems, cyclic degradation is larger and residual capacities at large shear strains are smaller. The post-peak performance of the mechanical systems is preferred - this could potentially be achieved with different detailing/connectors or specialized deck profiles, but in current non-proprietary systems this is not common/available.

source: Essa et al. (2003), O'Brien et al. (2017), summary calculations original

Impact of Deck orientation

- Conceptually “shear” does not have a direction as for equilibrium force and deformations are required on all sides (i.e. both parallel and perpendicular to the deck).
- Nonetheless, in the database only the PAF/screw condition under monotonic loading includes testing with the deck both parallel to and perpendicular to the load.
- However, like specimens have not been tested – Bagwell (2007) tested with the deck perpendicular and this can be compared to tests of others with the deck parallel. However, 2 of his 3 relevant specimens used full hard deck steel.



- Little evidence that dir. of deck relevant
- Some evidence that full hard low ductility deck steel (ε_u<10%, F_u/F_y~1, F_y>80ksi) should potentially be avoided when chasing ductility

source: O’Brien et al. (2017) (also see for Bagwell 2007), ductility plot original

Impact of endlaps on ductility

CFSRC

	Monotonic			Cyclic			
Condition	n	μ_m		n	μ_c		μ_c/μ_m
No Endlap	17	3.36		18	2.47		73%
PAF Endlap	9	3.08		12	2.46		80%
Weld Endlap	21	3.55		1	2.36		66%

- Overall endlaps have only minor influence on ductility of tested cantilever diaphragms
- Interestingly, if you drill into the data further endlaps are
  ..slightly beneficial for specimens with welded structural connectors (presumably providing additional shear deformation at high load in the system), and
  ..slightly detrimental with mechanical structural connectors (presumably applying additional out-of-plane forces on the connector).

source: O'Brien et al. (2017) database, ductility table original

Stiffness and Strength Prediction Accuracy

CFSRC

- AISI S310 Stiffness and Strength Predictions are compared to the available test database
- Comparison includes 82 specimens monotonic and cyclic with digitized load-disp. data
- AISI S310 developed based on wider data, but, summary provides some insight

Connector				$S_{test}/S_{predicted}$		$G'_{test}/G'_{predicted}$	
Structural	sidelap	count	ϕ	mean	stdev	mean	stdev
Weld	any	42	0.55	0.89	0.23	0.85	0.20
PAF or Screw or Bolt	screw	40	0.70	1.20	0.22	0.68	0.24

- Variability in welds is high, at fastener level have observed the opposite of this strength test-to-predicted ratio (i.e. > 1.0) ; low ϕ factor accounts for variability
- Stiffness error can be related to use of stiffness at 40% maximum, in addition data includes monotonic and cyclic tested G'; error may also be due to this inclusion
- AISI S310 (2016) is adequate, but after further study refinements may be needed

source: O'Brien et al. (2017) database, strength and stiffness table original

Cyclic PAF/Screw - Database Characteristics

Thinking about possible prescriptive characteristics for the best performing deck, we note the following from cyclic PAF/Screw tests:

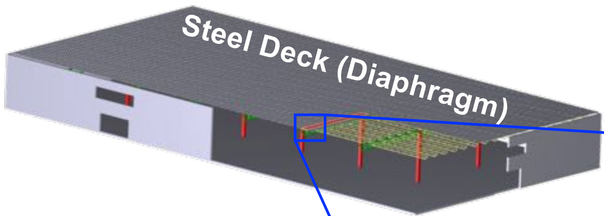
- Deck
  - 36 in. wide B deck
  - t=0.0276 in. to 0.05748 in. (24 to 16 gauge)
  - F_y=36 to 56 ksi, ε_u>20% (one specimen - F_u=96ksi, ε_u=10% specimen)
  - (Note cellular deck removed from dataset)
- Structural Connectors
  - Hilti X-HSN 24, X-ENP-19L15, X-EDNK22-THQ12; Buildex BX12
  - 3, 6, 9, 12 in. spacing
- Sidelap
  - #12
  - 6, 12 in. spacing

source: O'Brien et al. (2017) database

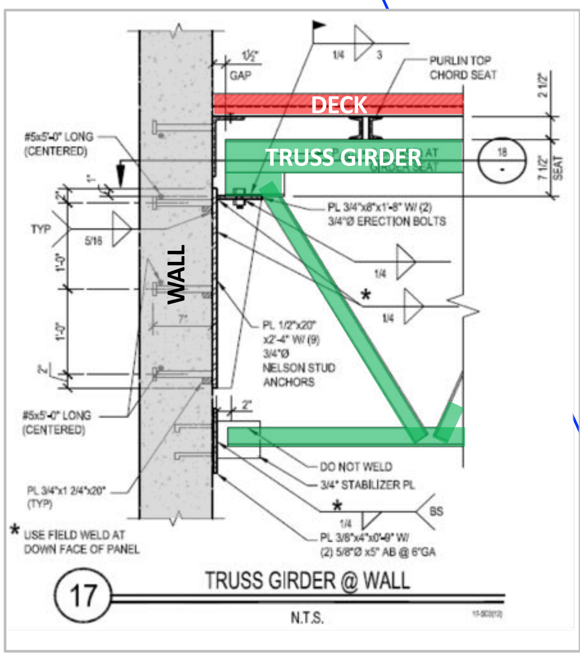
Building Applications
Steel RWFD Buildings

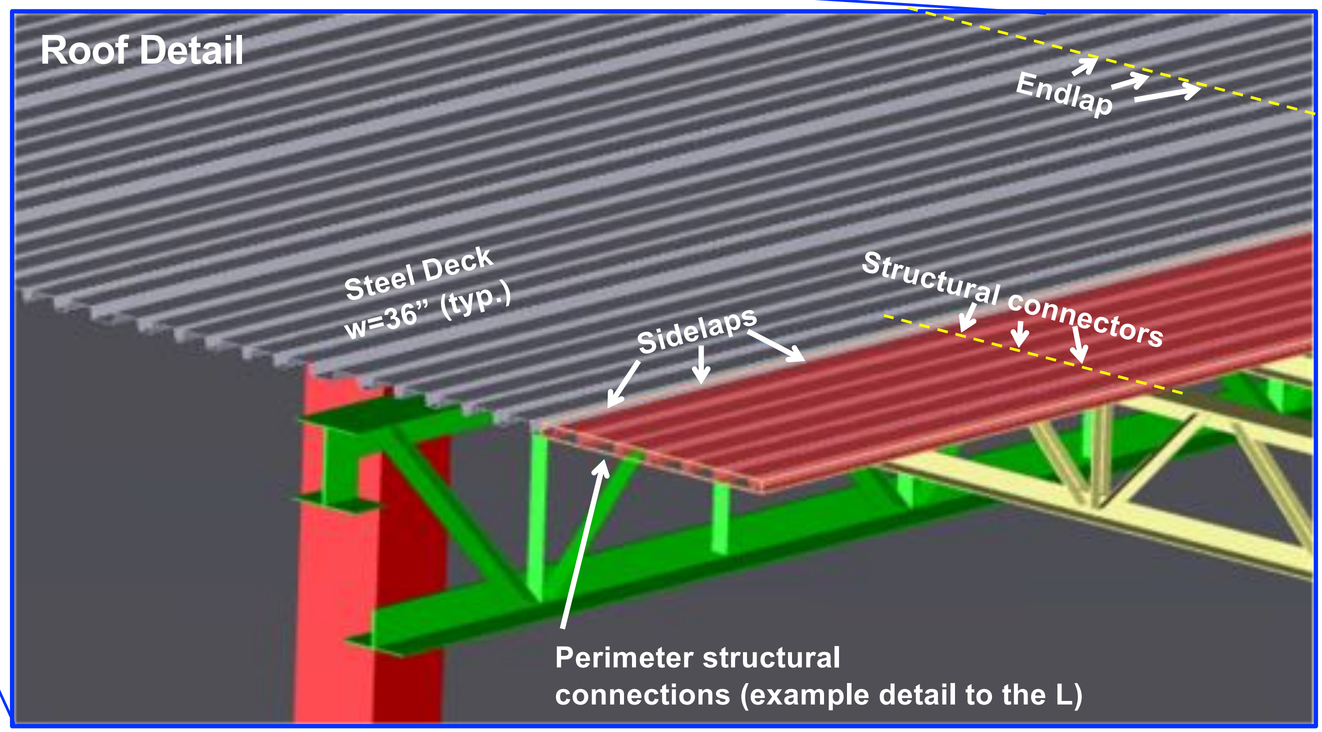
Implications of deck diaphragm performance on building performance.
FEMA P-1026 investigation and new investigations and modeling.

RWFD Building – Steel Deck Roof



Example
Perimeter Detail





source: Schafer et al. (2018) concept drawing / Verco (2018) private comm. for perimeter detail

RWFD Buildings

Summary of need from P-1026

- RWFD is a common building type
- Inelasticity in diaphragm often important to successful building performance for RWFD bldg.
- Inelasticity in diaphragm violates basic assumptions of conventional ELF-based design
- Past performance creates concern

Current Status

- Conventional design and alternative solution examined
- IT9 has brought the fruits of its labor for wood roof diaphragms to the BSSC PUC

FEMA P-1026 simulation engine


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

FEMA P-1026/March 2015

FEMA

nehrp


SEISMIC ANALYSIS AND DESIGN OF RIGID WALL – FLEXIBLE ROOF DIAPHRAGM BUILDINGS

by

Maria Koliou

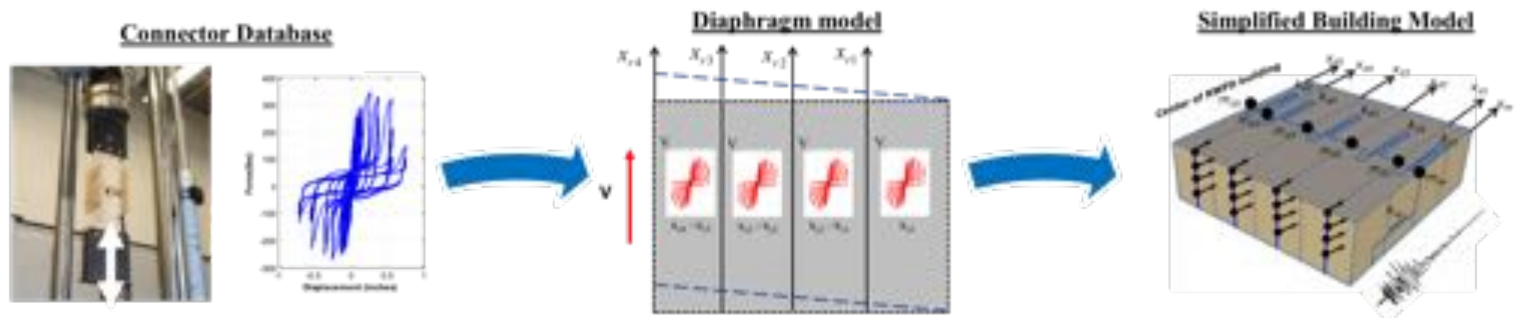
June 05, 2014

A dissertation submitted to the
Faculty of the Graduate School of
the University at Buffalo, State University of New York
in partial fulfillment of the requirements for the
degree of

Doctor of Philosophy

Department of Civil, Structural and Environmental Engineering

- Simulation Framework



Employed Tremblay and Rogers (2003a,b) data, similar to testing reported here, but not on full length deck specimens per AISI S905. Results in different response for some cases. Discussed more in later slide.

Verified model against Tremblay and Rogers PAF/screw cantilever test and SAP 2000 shell model. Energy dissipation and hysteretic behavior deemed acceptable.

Verified model against existing 3D building model completed in PERFORM. Fragility output from IDA determined to be sufficiently accurate in comparison.

source: FEMA P-1026 (2015), Koliou (2014), Koliou et al. papers

FEMA P-1026 archetypes

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

FEMA P-1026/March 2015

FEMA

nehrp

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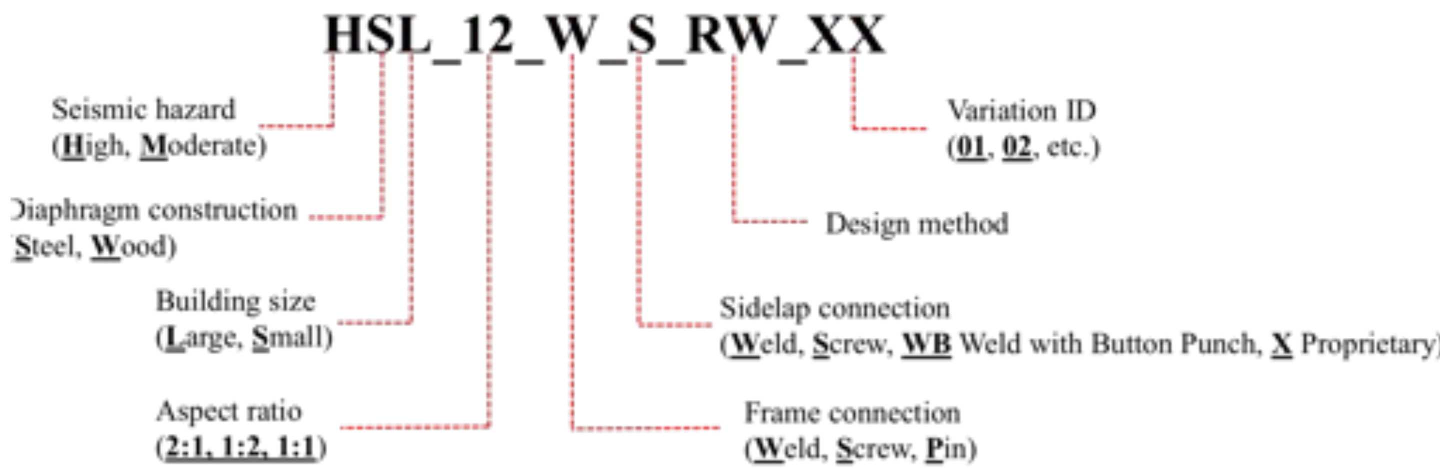
Department of Civil, Structural and Environmental Engineering

- Employed P695 procedure to investigate response
- RWFD Building Archetypes
  Wood in FEMA P-1026, Steel complete in Koliou (2014)

Nomenclature:



source: FEMA P-1026 (2015), Koliou (2014), Koliou et al. papers

FEMA P-1026 archetype performance

CFSRC

Conventional Design

Archetype ID	Design configuration				Collapse margin parameters				Acceptance check	
	Building size	Diaphragm aspect ratio	Diaphragm construction	Seismic SDC	CMR	μ_T	SSF	ACMR	Accept. ACMR	Pass/Fail
Performance Group No. PG-5E (Steel, Large Building, Welds and Button Punches as sidelap Connectors, Existing Design)										
HSL_21_W_WB_RW4_01	Large	2:1	Steel	D_{max}	0.99	8.09	1.34	1.33	1.73	Fail
HSL_12_W_WB_RW4_01	Large	1:2	Steel	D_{max}	1.90	8.26	1.33	2.53	1.73	Pass
HSL_11_W_WB_RW4_01	Large	1:1	Steel	D_{max}	0.95	8.16	1.33	1.27	1.73	Fail
Mean of Performance Group:					**1.28**	**8.17**	**1.33**	**1.71**	**2.30**	**Fail**
Performance Group No. PG-6E (Steel, Large Building, Screws as sidelap Connectors, Existing Design)										
HSL_21_P_S_RW4_01	Large	2:1	Steel	D_{max}	1.23	8.24	1.35	1.67	1.73	Fail
HSL_12_P_S_RW4_01	Large	1:2	Steel	D_{max}	2.07	8.14	1.33	2.75	1.73	Pass
HSL_11_P_S_RW4_01	Large	1:1	Steel	D_{max}	1.13	8.26	1.36	1.53	1.73	Fail
HSL_11_S_S_RW4_01	Large	1:1	Steel	D_{max}	1.15	8.01	1.33	1.53	2.73	Fail
Mean of Performance Group:					**1.40**	**8.16**	**1.34**	**1.87**	**2.30**	**Fail**
Performance Group No. PG-7E (Steel, Small Building, Button Punches as sidelap Connectors, Existing Design)										
HSS_11_W_B_RW4_01	Small	1:1	Steel	D_{max}	1.73	7.94	1.32	2.28	1.73	Pass
HSS_21_W_B_RW4_01	Small	2:1	Steel	D_{max}	1.42	8.05	1.33	1.89	1.73	Pass
HSS_12_W_B_RW4_01	Small	1:2	Steel	D_{max}	1.90	7.91	1.32	2.51	1.73	Pass
Mean of Performance Group:					**1.68**	**7.97**	**1.32**	**2.23**	**2.30**	**Fail**
Performance Group No. PG-8E (Steel, Small Building, Screws as sidelap Connectors, Existing Design)										
HSS_11_P_S_RW4_01	Small	1:1	Steel	D_{max}	1.55	8.02	1.33	2.07	1.73	Pass
HSS_11_S_S_RW4_01	Small	1:1	Steel	D_{max}	1.43	8.15	1.33	1.91	1.73	Pass
HSS_21_P_S_RW4_01	Small	2:1	Steel	D_{max}	1.33	8.33	1.33	1.76	1.73	Pass
HSS_12_P_S_RW4_01	Small	1:2	Steel	D_{max}	1.71	8.25	1.33	2.27	1.73	Pass
HSS_21_S_S_RW4_01	Small	2:1	Steel	D_{max}	1.25	7.85	1.32	1.65	1.73	Fail
HSS_12_S_S_RW4_01	Small	1:2	Steel	D_{max}	1.42	8.06	1.33	1.89	1.73	Pass
Mean of Performance Group:					**1.45**	**8.11**	**1.33**	**1.92**	**2.30**	**Fail**

source: FEMA P-1026 (2015), Koliou (2014)

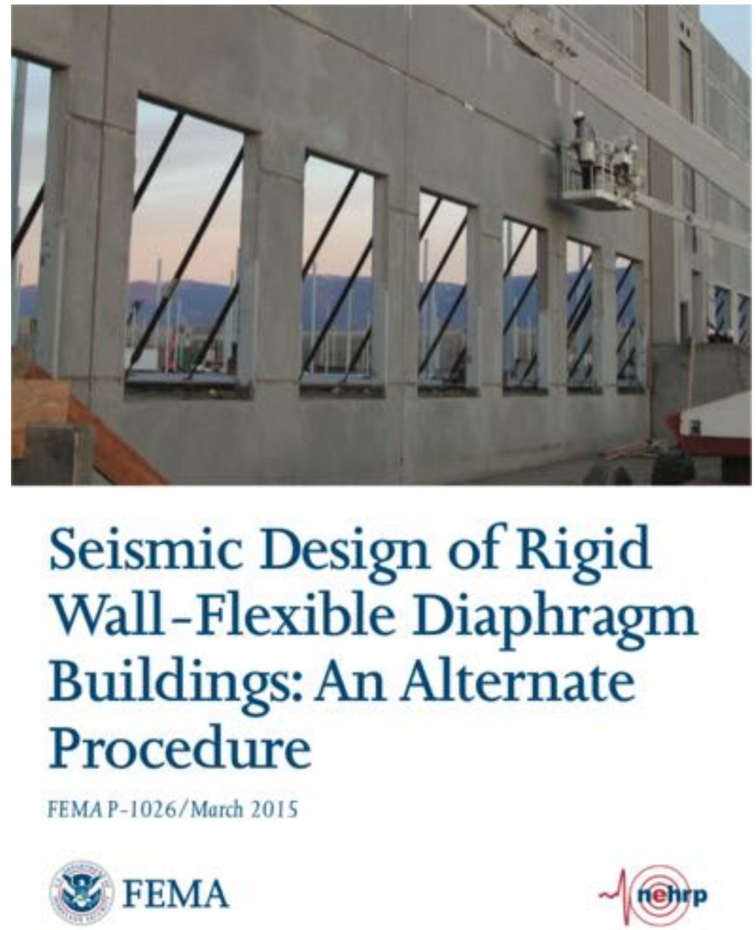
- All steel performance groups are predicted to have unacceptable CMR
- Large buildings have lower CMR than small buildings **(large roof more critical)**
- Short direction (1:2) always results in acceptable performance, focus on weak direction (2:1) aspect ratios **(weak dir. more critical)**
- Not shown – SDC C models perform better than SDC D models **(SDC D more critical)**

FEMA P-1026 and steel

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

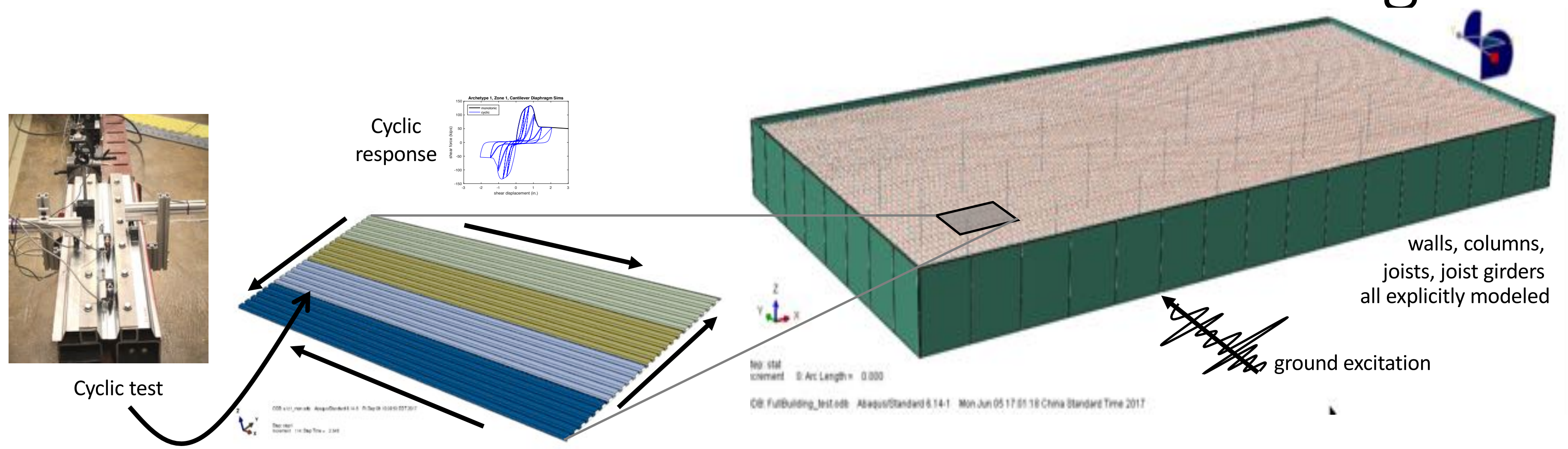
(1) tests results of a large scale diaphragm showed significantly less distribution of yielding than analyses ...,
(2) … design strengths are based on monotonic tests,
(3) data for reverse cyclically loaded connections is sparse …,
(4) the post-yield stiffness of connectors is positive for only a small 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



source: FEMA P-1026 (2015)

New 3D simulation of RWFD steel buildings



(a) Connector tests

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

(b) 3D Roof submodel

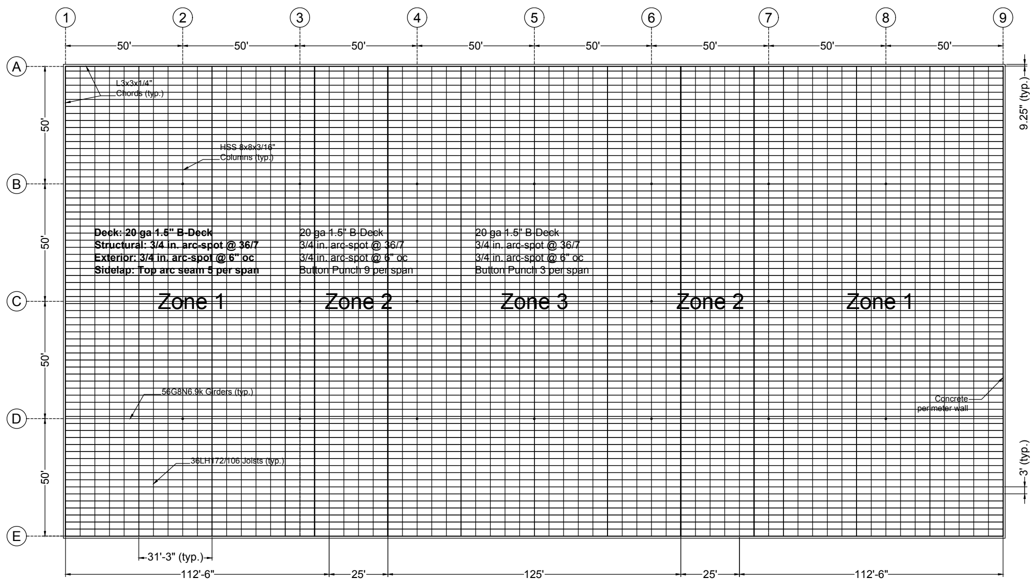
- Shell FE model, material and geometric nonlin.
- Similar to cantilever diaphragm testing
- Nonlinear connectors
- Establish cyclic perform-ance of roof segment
- Validated against testing

(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

source: NBM see Schafer et al. (2018) summary

Archetypes: A1 (PAF/Screw), A3 (Weld/Weld(/BP))



- ”Large” 200x400 building design, SDC D
- Design per AISI S310-16 and ASCE7-16
- Summary of A1 and now A3 designs to the right
- Roof designed in three zones

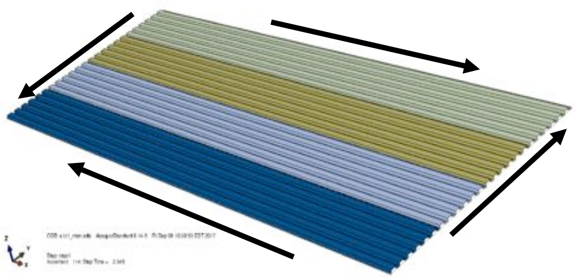
	A1	A3
• Zone 1	PAF/Screw	Weld/Weld
• Zone 2	PAF/Screw	Weld/BP
• Zone 3	PAF/Screw	Weld/BP

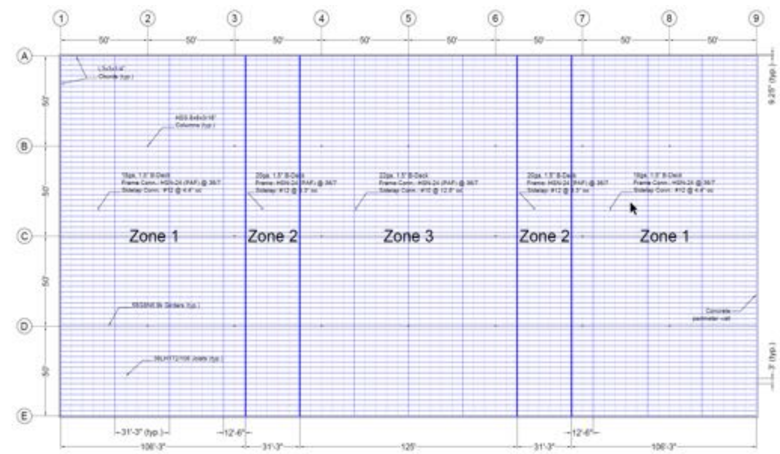
source: Schafer et al. (2018), A3 new

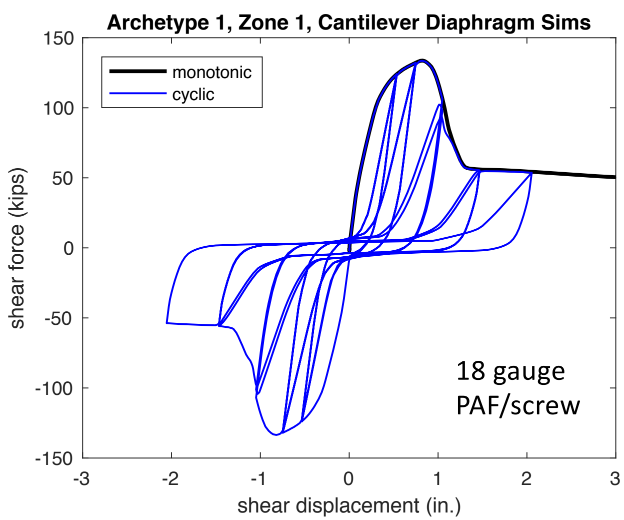
Roof Zones	A1	A3
Zone 1	**Bodwell PAF/SCREW D**	**WELD/BP Design**
Location from edge (ft)	0	0
LRFD Demand (plf)	1641	1641
Deck	18 ga 1.5" B-Deck	20 ga 1.5" B-Deck
Structural Connector	HSN-24 (PAF) @ 36/7	3/4 in. arc-spot @ 36/7
Exterior Edge Spacing	HSN-24 (PAF) @ 6" oc	3/4 in. arc-spot @ 6" oc
Sidelap Connector	16 #12 per 6.25' span	Top arc seam 5 per span
Nominal capacity, v_n (plf)	2914	3136
Design capacity, ϕv_n (plf)	1894	1725
D/C	0.87	0.95
Zone 2		
Location from edge (ft)	106.25	112.5
LRFD Demand (plf)	769	718
Deck	20 ga 1.5" B-Deck	20 ga 1.5" B-Deck
Structural Connector	HSN-24 (PAF) @ 36/7	3/4 in. arc-spot @ 36/7
Edge Spacing	HSN-24 (PAF) @ 6" oc	3/4 in. arc-spot @ 6" oc
Sidelap Connector	9 #12 per 6.25' span	Button Punch 9 per span
Nominal capacity, v_n (plf)	1621	1344
Design capacity, ϕv_n (plf)	1054	739
D/C	0.73	0.97
Zone 3		
Location from edge (ft)	137.5	137.5
LRFD Demand (plf)	513	513
Deck	20 ga 1.5" B-Deck	20 ga 1.5" B-Deck
Structural Connector	HSN-24 (PAF) @ 36/7	3/4 in. arc-spot @ 36/7
Edge Spacing	HSN-24 (PAF) @ 6" oc	3/4 in. arc-spot @ 6" oc
Sidelap Connector	6 #12 per 6.25' span	Button Punch 3 per span
Nominal capacity, v_n (plf)	1001	1049
Design capacity, ϕv_n (plf)	651	577
D/C	0.79	0.89

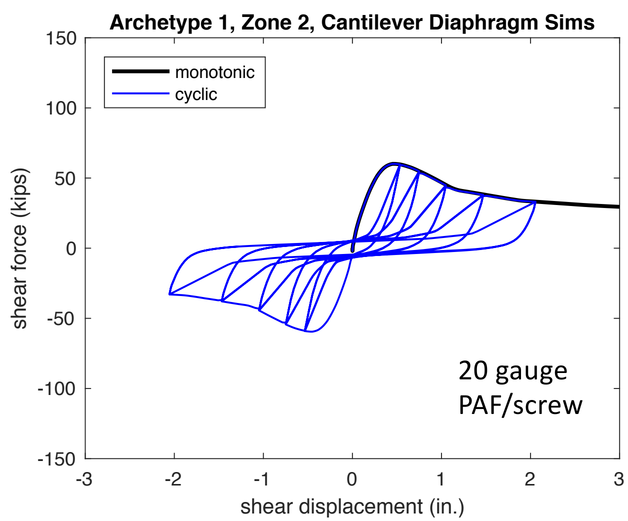
A1: Results of Roof Zone Modeling

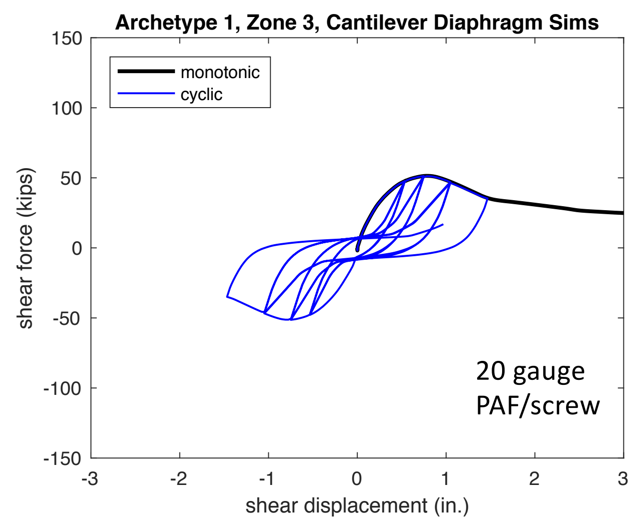
CFSRC







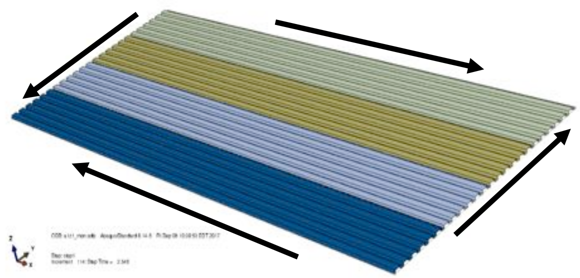


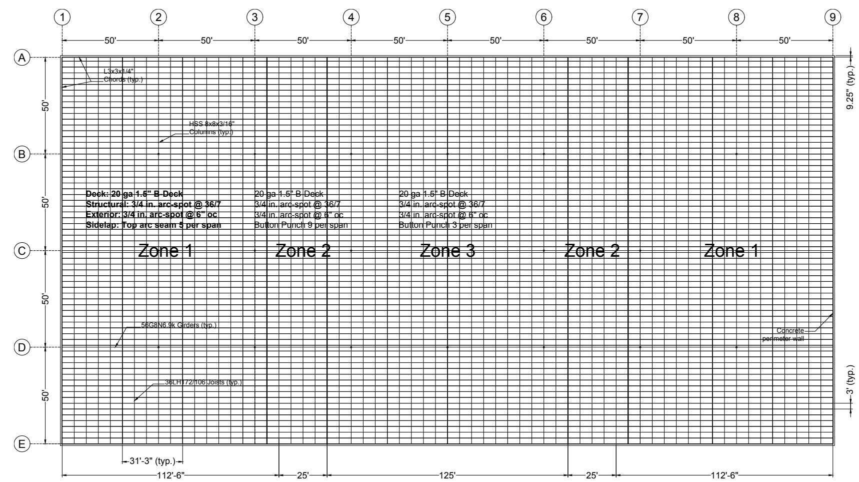


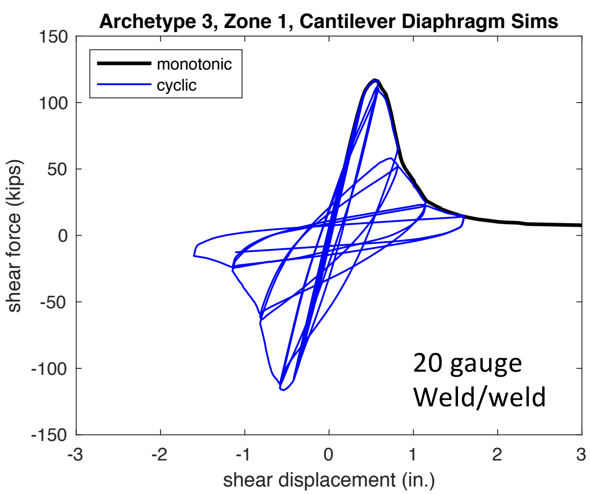
source: Schafer et al. (2018)

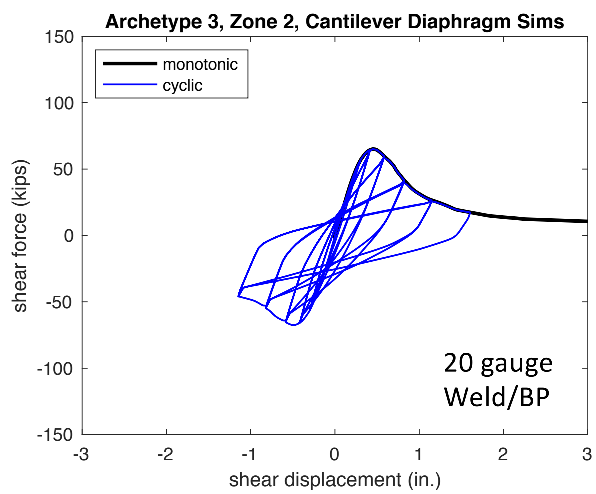
1 in. = 0.7% shear angle

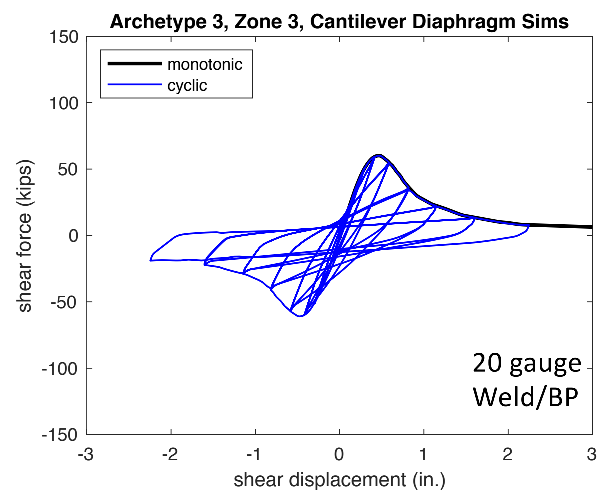
A3: Results of Roof Zone Modeling











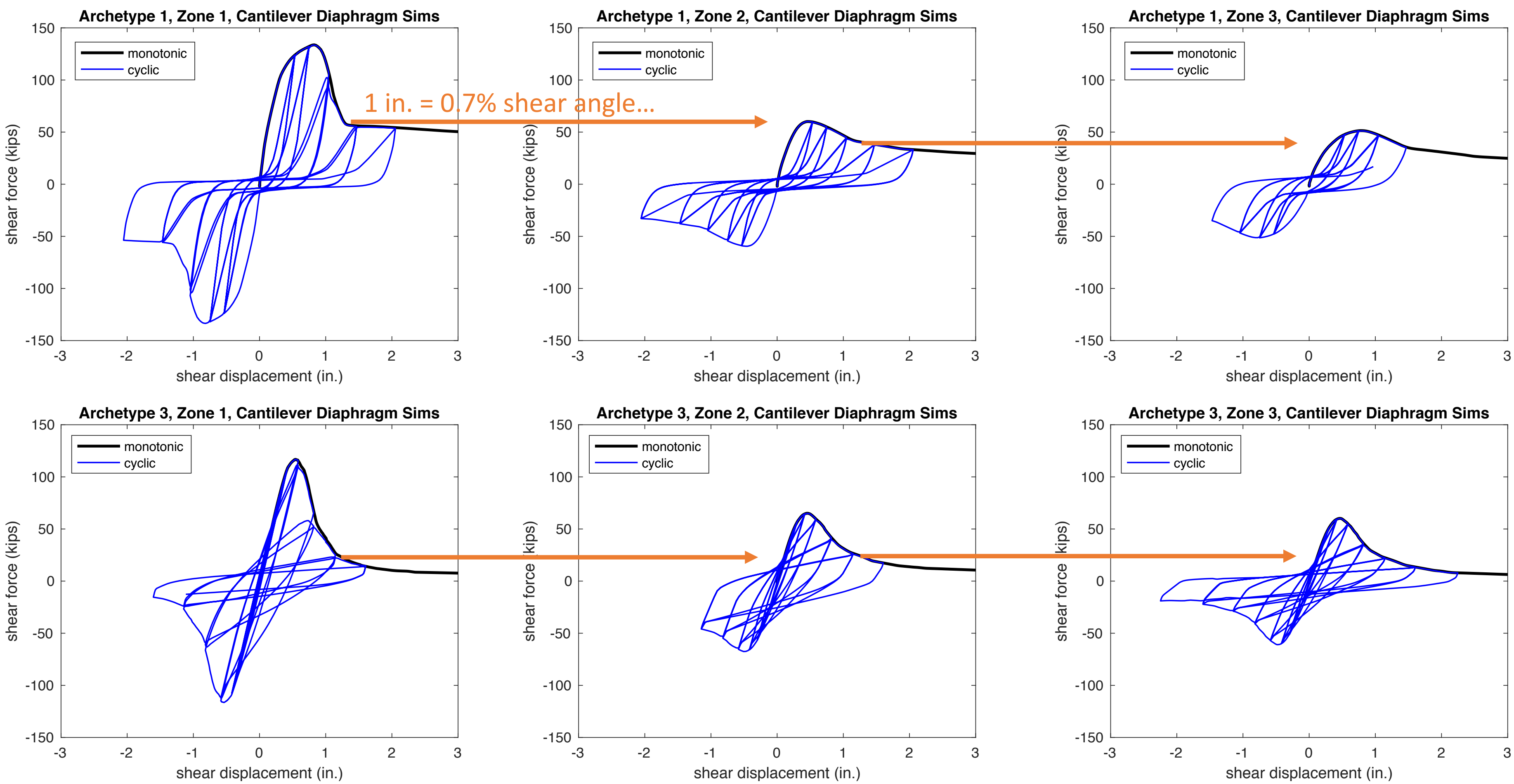
1 in. = 0.7% shear angle

source: new work

Comparison of A1 and A3 roof performance

PAF/Screw

Weld/Weld or BP



Building Simulation Details (P695 details)

- Apply FEMA P695 11.3 Collapse Evaluation of Individual Buildings

  Typical P695: $(SSF)(CMR)>ACMR_{10\%}$

  Noting: $(SSF)(S_{CT}/S_{MT})>ACMR_{10\%}$

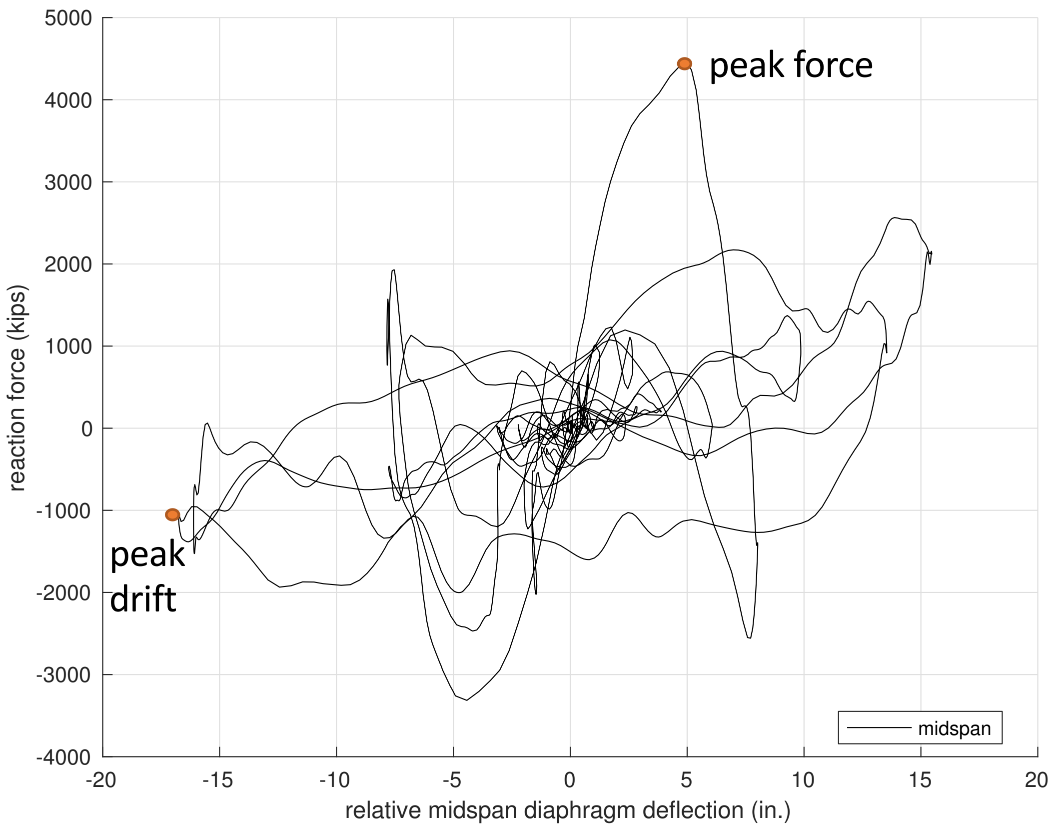
  Results in: $S_{CT}>S_{MT}(ACMR_{10\%}/SSF)$

  - Run 44 P695 earthquake motions at this scale factor
  - If median is acceptable then building "passes" examination
- Still must include uncertainty through β, selected values

		FEMA P-1026		This analysis	
		Value	Description	Value	Description
EQ record:	β_{RTR}	0.4	upperbound	0.2~0.4	P695 formula
Design:	β_{DR}	0.2	Good	0.2	Good
Test:	β_{TD}	0.35	Fair	0.2	Good
Model:	β_{MDL}	0.35	Fair	0.2	Good
	β_{TOT}	0.67		0.40~0.53	
	$ACMR_{20\%}$	1.75		1.40~1.56	
	$ACMR_{10\%}$	2.35		1.67~1.97	

source: new work

Example A1: N-S SF2.25 EQ4 Base Shear-Roof Drift Trace

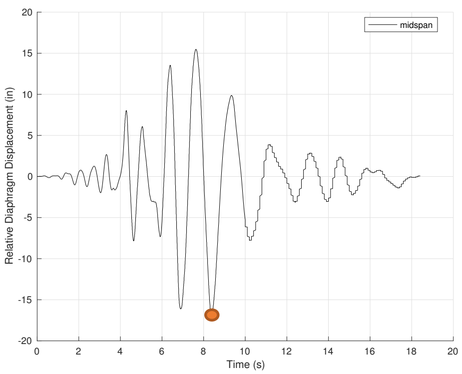


Discussion

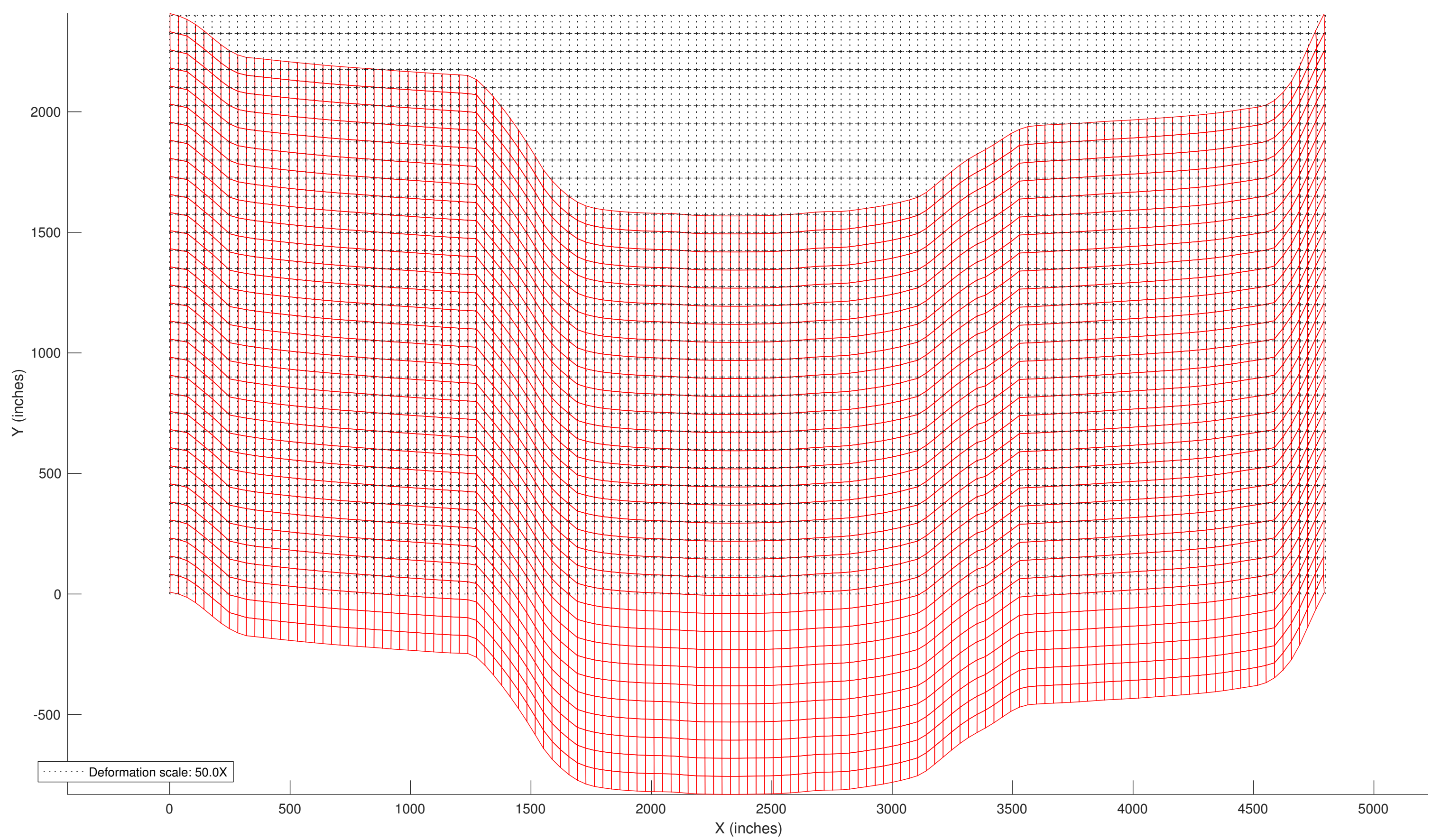
- What we see is a large cycle that led to damage and heavily degraded stiffness
- Response still dissipating energy, still zero centered (not drifting away even at high demand)
- Examined peak force and peak drift response, focusing on peak drift in the following slides

source: Schafer et al. (2018)

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



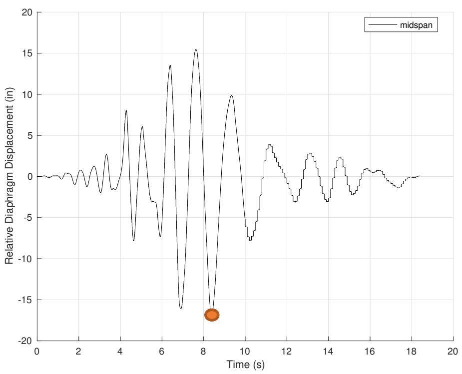
Magnified Roof Displaced Shape



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

source: Schafer et al. (2018)

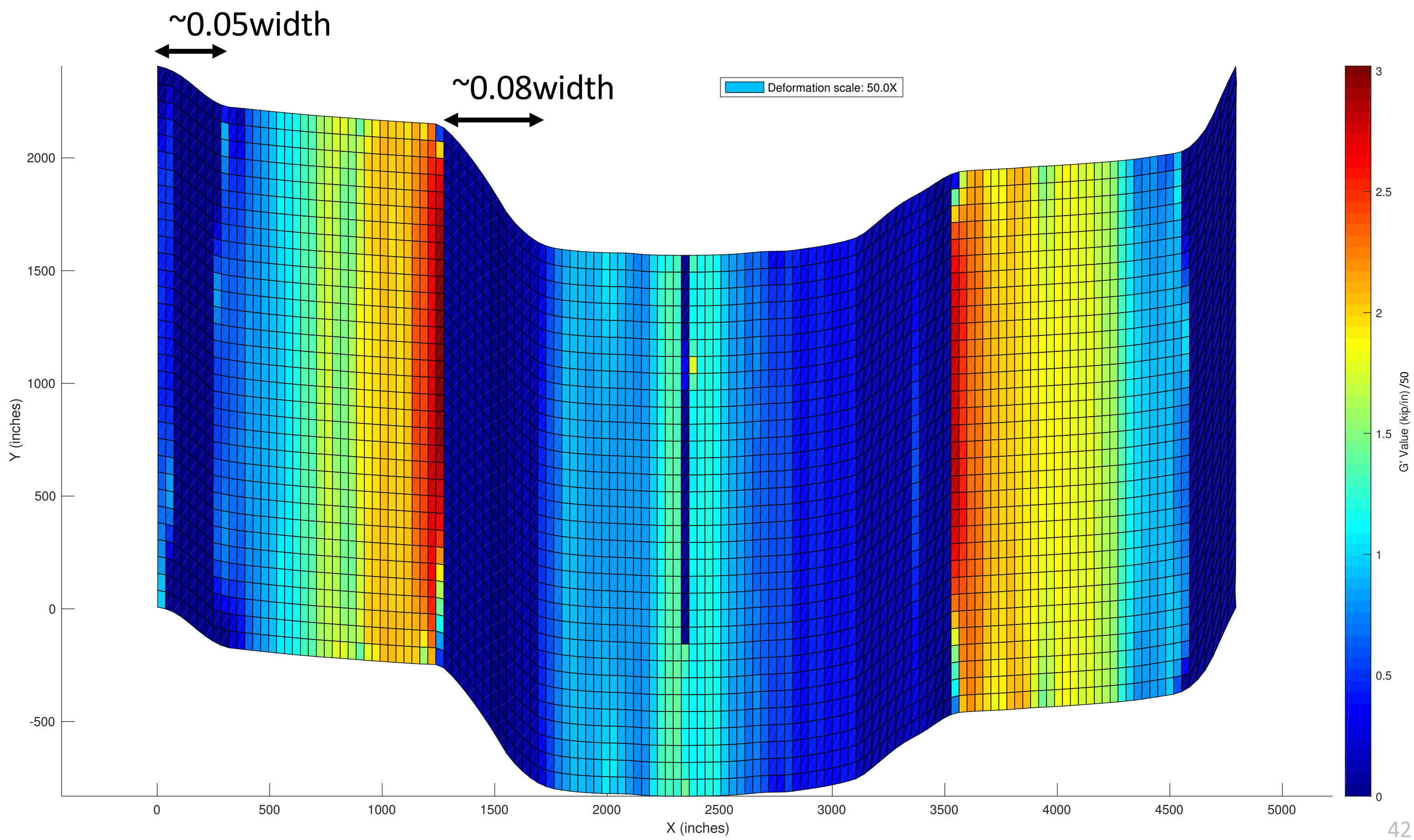
Example A1: N-S SF2.25 EQ4 at Peak Drift



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)

source: Schafer et al. (2018)

G’ contour

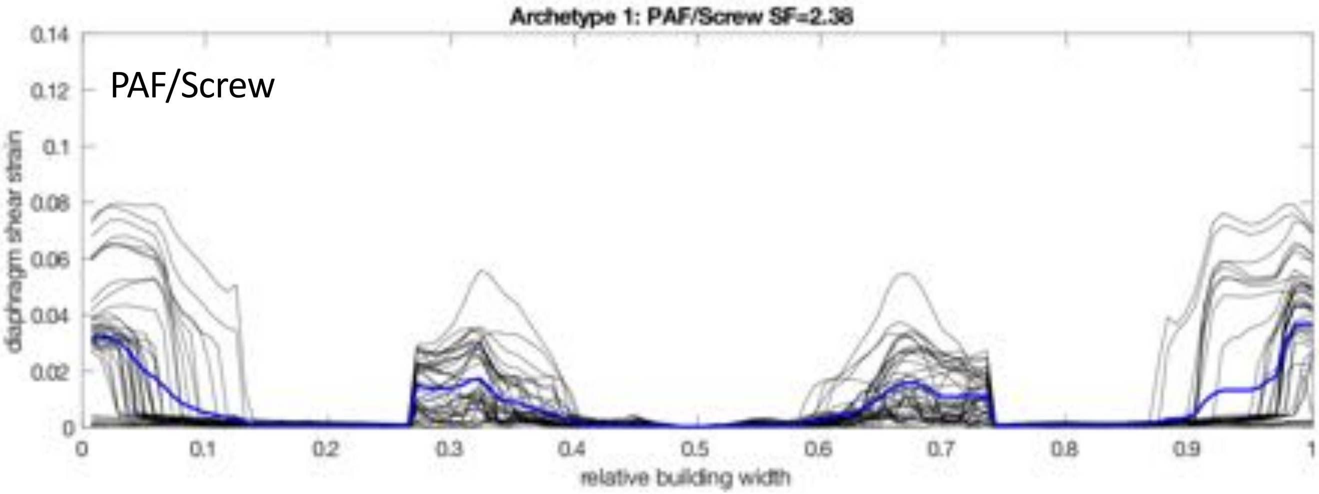


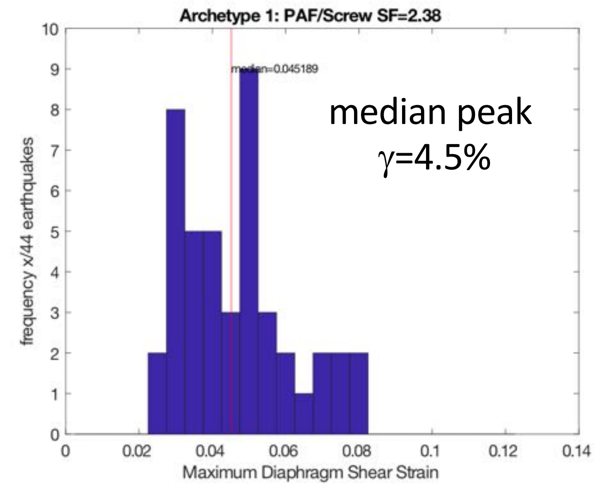
Results across EQ suite

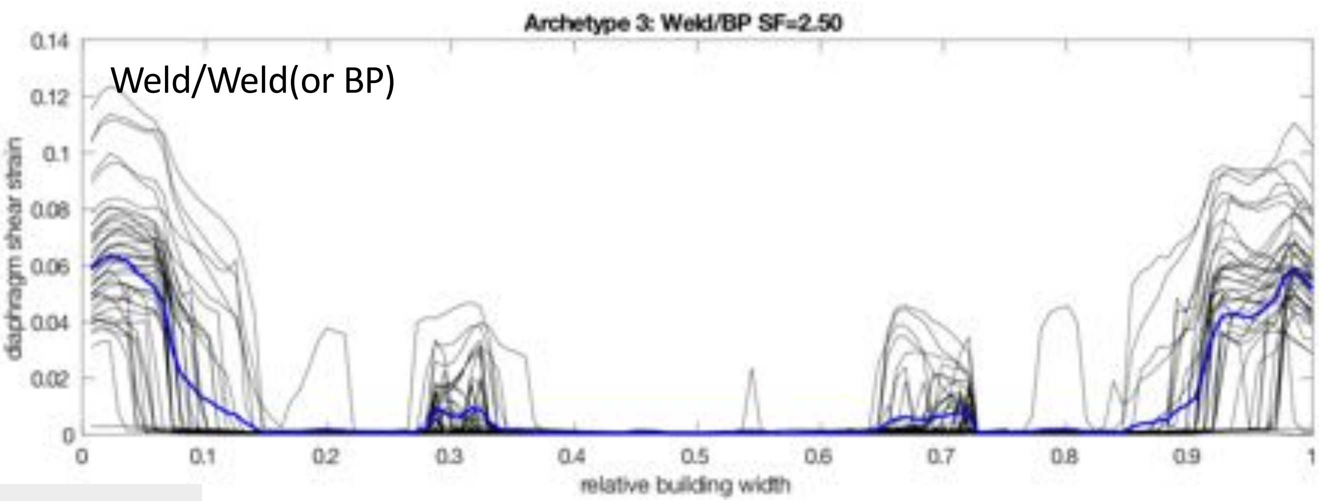
Now transiting to results across both archetypes and the 44 P695 EQ suite

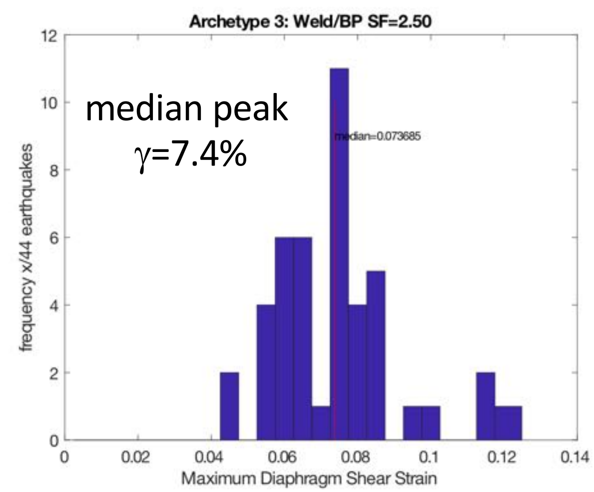
Archetype 1 at Scale Factor=2.38, Archetype 3 at Scale Factor=2.5 to meet $ACMR_{10\%}$

Maximum diaphragm shear strain





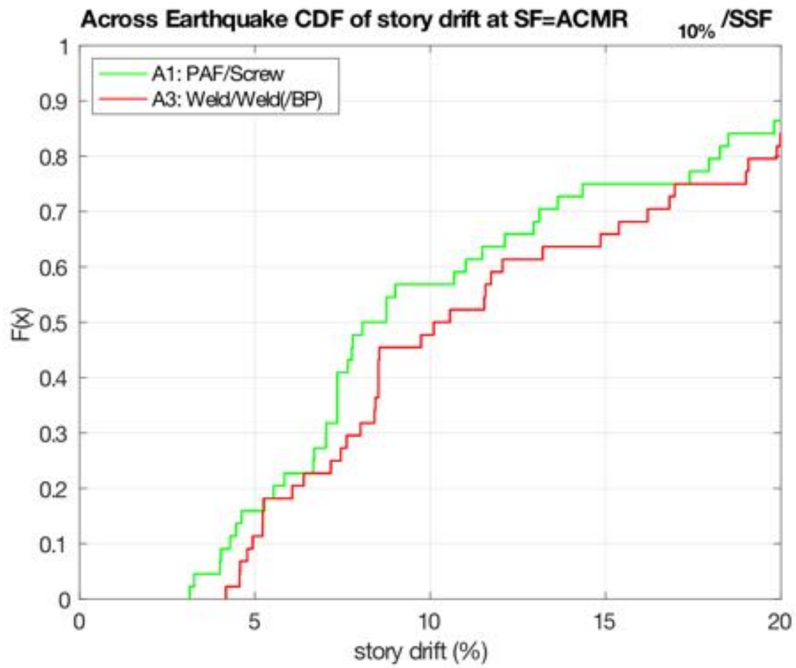




source: new work

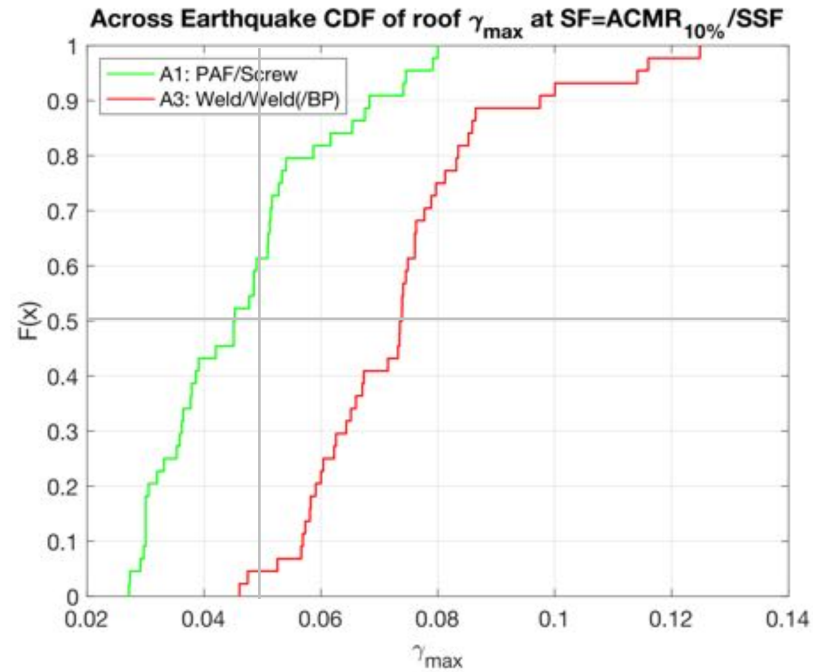
Failure Criteria

Story Drift



Discussion:
Not a good failure criteria for this collapse
Vertical system still must sustain this drift

Roof Shear Strain



Discussion:
γ=5% separates PAF/Screw from Weld/Weld
Implies considerable roof damage

source: new work

Predicted Performance vs. P-1026

Conventional Design

Archetype ID	Design configuration				Collapse margin parameters				Acceptance check	
	Building size	Diaphragm aspect ratio	Diaphragm construction	Seismic SDC	CMR	μ_T	SSF	ACMR	Accept. ACMR	Pass/Fail
Performance Group No. PG-5E (Steel, Large Building, Welds and Button Punches as sidelap Connectors, Existing Design)										
HSL_21_W_WB_RW4_01	Large	2:1	Steel	D_{max}	0.99	8.09	1.34	1.33	1.73	Fail
HSL_12_W_WB_RW4_01	Large	1:2	Steel	D_{max}	1.90	8.26	1.33	2.53	1.73	Pass
HSL_11_W_WB_RW4_01	Large	1:1	Steel	D_{max}	0.95	8.16	1.33	1.27	1.73	Fail
Mean of Performance Group:					**1.28**	**8.17**	**1.33**	**1.71**	**2.30**	**Fail**
Performance Group No. PG-6E (Steel, Large Building, Screws as sidelap Connectors, Existing Design)										
HSL_21_P_S_RW4_01	Large	2:1	Steel	D_{max}	1.23	8.24	1.35	1.67	1.73	Fail
HSL_12_P_S_RW4_01	Large	1:2	Steel	D_{max}	2.07	8.14	1.33	2.75	1.73	Pass
HSL_11_P_S_RW4_01	Large	1:1	Steel	D_{max}	1.13	8.26	1.36	1.53	1.73	Fail
HSL_11_S_S_RW4_01	Large	1:1	Steel	D_{max}	1.15	8.01	1.33	1.53	2.73	Fail
Mean of Performance Group:					**1.40**	**8.16**	**1.34**	**1.87**	**2.30**	**Fail**
Performance Group No. PG-7E (Steel, Small Building, Button Punches as sidelap Connectors, Existing Design)										
HSS_11_W_B_RW4_01	Small	1:1	Steel	D_{max}	1.73	7.94	1.32	2.28	1.73	Pass
HSS_21_W_B_RW4_01	Small	2:1	Steel	D_{max}	1.42	8.05	1.33	1.89	1.73	Pass
HSS_12_W_B_RW4_01	Small	1:2	Steel	D_{max}	1.90	7.91	1.32	2.51	1.73	Pass
Mean of Performance Group:					**1.68**	**7.97**	**1.32**	**2.23**	**2.30**	**Fail**
Performance Group No. PG-8E (Steel, Small Building, Screws as sidelap Connectors, Existing Design)										
HSS_11_P_S_RW4_01	Small	1:1	Steel	D_{max}	1.55	8.02	1.33	2.07	1.73	Pass
HSS_11_S_S_RW4_01	Small	1:1	Steel	D_{max}	1.43	8.15	1.33	1.91	1.73	Pass
HSS_21_P_S_RW4_01	Small	2:1	Steel	D_{max}	1.33	8.33	1.33	1.76	1.73	Pass
HSS_12_P_S_RW4_01	Small	1:2	Steel	D_{max}	1.71	8.25	1.33	2.27	1.73	Pass
HSS_21_S_S_RW4_01	Small	2:1	Steel	D_{max}	1.25	7.85	1.32	1.65	1.73	Fail
HSS_12_S_S_RW4_01	Small	1:2	Steel	D_{max}	1.42	8.06	1.33	1.89	1.73	Pass
Mean of Performance Group:					**1.45**	**8.11**	**1.33**	**1.92**	**2.30**	**Fail**

Archetype Findings <u>Extrapolated</u> Back to FEMA P-1026 Study

	Comment
	Expect insufficient CMR
	Elastic design b/c of zones in other dir.
	Expect insufficient CMR
	Group will need a design change
	Models suggest acceptable CMR
	Elastic design b/c of zones in other dir.
	1:1 not appreciably different, expect success
	Expect regular fasteners insufficient
	If PAF/Screw only, sufficient CMR
	Large building more critical
	Weld/BP nearly passed with 2D model
	Weld/BP may be OK, but cannot assume
	Large building more critical
	PAF/Screw and Screw/Screw nearly passed
	PAF/Screw presumed OK

source: new work

CFSRC

Transitioning to design methods

R only, R and R_s, R and RWFD with R_{diaph}

CFSRC

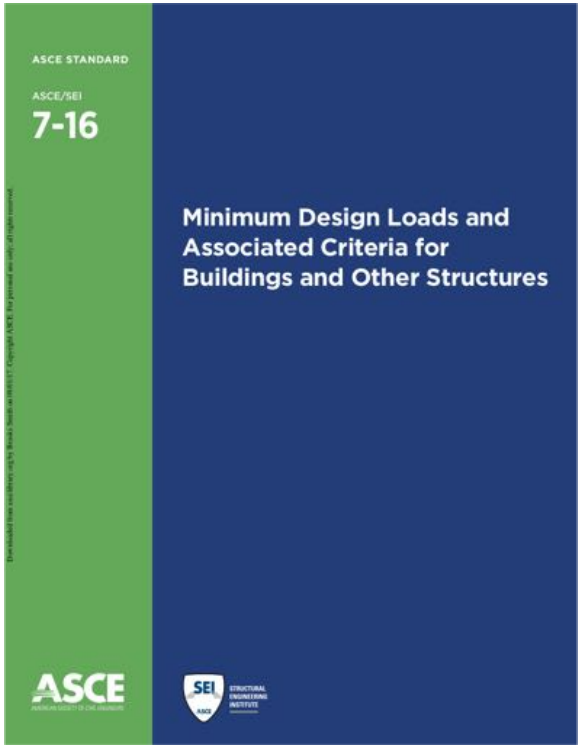
Diaphragm Design - Demand

- For the purposes of this presentation, assuming quite a bit of familiarity with the three diaphragm demand options currently available

- Traditional Diaphragm Design (R)
  - ASCE 7 12.10.1

- Alternative Diaphragm Design (R_s)
  - ASCE 7 12.10.3

- New RWFD Diaphragm Design (R_{diaph})
  - FEMA P-1026
  - BSSC IT9 Ballot



Diaphragm Design – Capacity

Relevant diaphragm design guidance does, and will in the near future, exist across a wide variety of standards

AISI S310/AISI S400 is the long term planned home for materials

ASCE 7

- General guidance
- Ch. 14 call outs

ASCE 41

- Ch. 9 = AISC 342

AISI

- AISI S310
- AISI S400
- AISI Test Standards

AISC

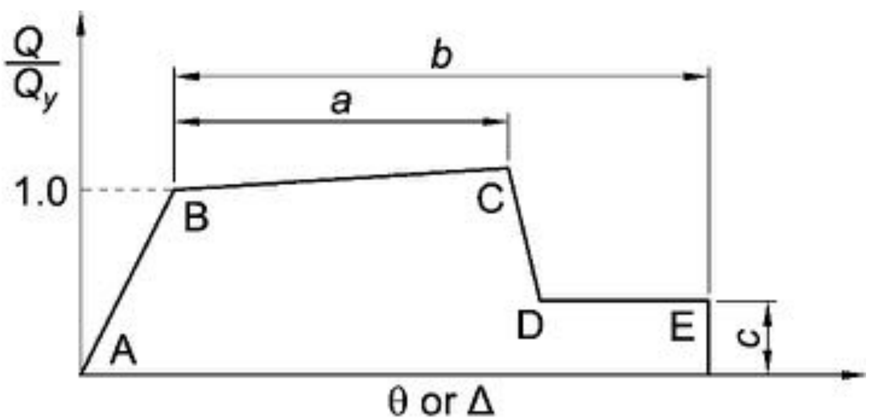
- AISC 341
- AISC 360

AISC 342/ASCE 41

m-factors and nonlinear modeling parameters – recently passed COS ballot one

Establishing nonlinear modeling parameters

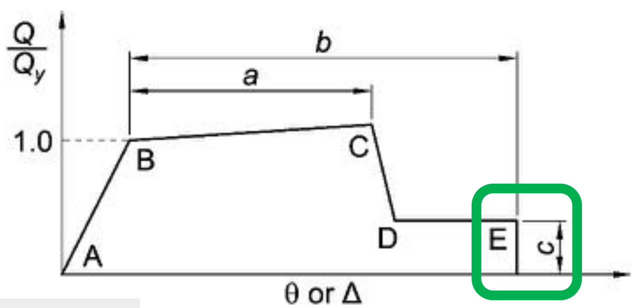
- AISC 342 will replace ASCE 41 Chapter 9 for structural steel
- Based on findings herein, specifically the SDII cantilever test database, new specification language was recently developed
- This work has passed in the first Specification level ballot, and is expected in the new addition



- AISC 342 Chapter G provides m-factors and modeling parameters
- Semi-rigid diaphragms with full in-roof-plane response provided
- Provides insight on one means to leverage existing data

AISC 342 modeling parameters

- ”Pushover” curve defined as equivalent energy elastic-perfectly plastic in this application, subtly different than earlier μ definition
- Key judgment made by task committee and approved by voters: residual strength ratios driven to near 0 for non-mechanical connectors



source: AISC 342 passed ballot

TABLE G1.2
Modeling Parameters and Acceptance Criteria for Nonlinear Procedures—Bare steel deck diaphragms

Component or Action	Modeling Parameters			Acceptance Criteria		
	Plastic Rotation Angle, rad.		Residual Strength Ratio	Plastic Rotation Angle, rad.		
	a	*b*	*c*	IO	LS	CP
Shear strength controlled by connectors:						
support: PAF; side-lap: screw	$2.7\gamma_y$	$3.7\gamma_y$	0.4	$1.4\gamma_y$	$2.8\gamma_y$	$4.0\gamma_y$
support: weld; side-lap: screw	$2.8\gamma_y$	$4.8\gamma_y$	0.05[b]	$1.4\gamma_y$	$2.8\gamma_y$	$4.0\gamma_y$
support: weld; side-lap: button punch	$1.7\gamma_y$	$3.1\gamma_y$	0.05[b]	$0.9\gamma_y$	$1.7\gamma_y$	$3.1\gamma_y$
support: weld; side-lap: weld	$2.3\gamma_y$	$3.6\gamma_y$	0.05[b]	$1.2\gamma_y$	$2.3\gamma_y$	$3.6\gamma_y$
Shear strength controlled by panel:						
buckling	TBD	TBD	TBD	TBD	TBD	TBD

CP = collapse prevention performance level as defined in ASCE/SEI 41 Chapter 2
IO = immediate occupancy performance level as defined in ASCE/SEI 41 Chapter 2
LS = life safety performance level as defined in ASCE/SEI 41 Chapter 2

[a] Values are for shear walls with stiffeners to prevent shear buckling.
[b] Structural connectors generally control residual strength. Value based on arc spot weld, arc seam weld c=0.15

- Also CP level estimated at about $4\gamma_y$

Special seismic detailing for bare steel deck

Establish a target system that has adequate ductile performance and call out this system whenever ductility is specifically required.

Introduce "special seismic" detail: *in progress*

- Amend AISI S400 in ASCE 7 Chapter 14
- "special seismic" detail for bare steel deck diaphragms created to insure ductile deck performance when explicitly needed
- Path 1: Prescriptive criteria for special seismic
  - Deck thickness and material limits (16-22 gauge ε_u>20%)
  - Structural connector: PAF, limited to tested PAFs
    - Perpendicular to deck no less than 36/7
    - Parallel to deck no more than 18 in. o.c.
  - Sidelap connector: Screw, sized to match gauge
    - Spaced no less than 6 in. and no more than 12 in.

- Path 2: Performance criteria for special seismic
  - Cyclic Cantilever diaphragm test that matches PAF/Screw performance
    - $\gamma_{80\%}/\gamma_y = \mu \geq 3$, 40% residual at max($4\gamma_y$, 2%)
  - Connector testing and diaphragm simulation

Introduce "special seismic" detail: *in progress*

- Amend AISI S400 in ASCE 7 Chapter 14
- "special seismic" detail for bare steel deck diaphragms created to insure ductile deck performance when explicitly needed
- Path 1: Prescriptive criteria for special seismic
  - Best of what we know today
  - Should cover PAF/screw space, could cover Screw/screw…
  - Intended to provide direct non-proprietary solution

- Path 2: Performance criteria for special seismic
  - Encapsulates key features of best performing system
  - Recognizes good performance observed in test database for other systems
  - Provides path for proprietary systems/alternative means to achieve ductility

Improving traditional steel deck diaphragm design

Providing for ductility when needed in conventional diaphragm design

Ductile vs. “non-ductile” roof detailing

- Under conventional design it is possible to design a bare steel deck roof that meets strength and service criteria but have little ductility
  - Such a non-ductile roof should be acceptable unless it is explicitly called upon to develop inelasticity and energy dissipation
- If ductility required in bare steel roof deck then
  - use “special seismic” provisions for selection, or
  - capacity protect deck by designing at Ω_o levels
- What should be the trigger for needing a ductile roof deck in conventional design?
  - R=3? Works for ordinary vertical steel systems, not applicable here
  - R<1? Flags cases where roof ductility likely needed, but misses others
  - **SDC D,E,F**? Coarse, but encompasses key seismic demands – and given lack of explicit knowledge on whether diaphragm needs to be ductile it seems prudent within context of conventional design (<u>currently proposed trigger</u>)

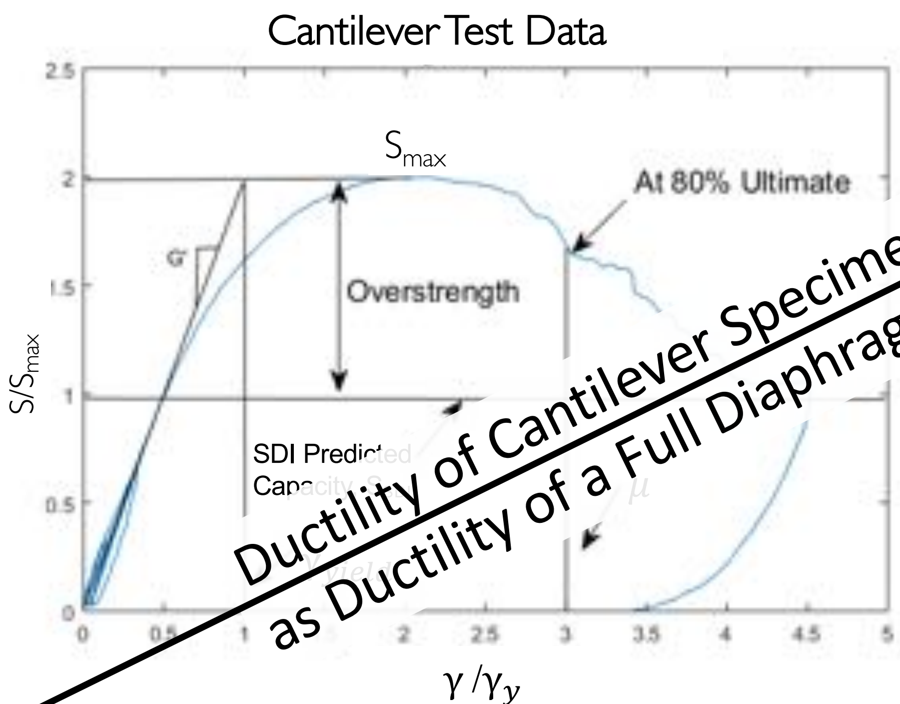
Alternative Diaphragm Design (R_s)

How to bring steel into the new alternative diaphragm design procedures

Alternative Roof Diaphragm Design - R_s

- Two categories should be introduced for bare steel deck:
  - Special (ductile/R_s>1)
  - Ordinary (non-ductile or unknown/R_s=1)
- Special = PAF/Screw or Equivalent Performance
  - Connector has ductility, designated energy dissipating mechanism
  - Cantilever diaphragm has ductility, deck and subsystem provide ductility
  - Building seismic simulations indicate acceptable performance
- Use cantilever diaphragm database to establish R_s for this system

First idea for estimating R_s



Ductility of Cantilever Specimen Not Same as Ductility of a Full Diaphragm System

~~Calculating R_s using ATC 19 Approach:~~

$R_s = R_\mu R_\Omega$

$R_\Omega = S_{max} / S_{SDI}$

$$\mu = \frac{\gamma_{ult}}{\gamma_{yield}}$$

$$R_\mu = \sqrt{2\mu - 1} \; or \; \mu$$

Depending on period

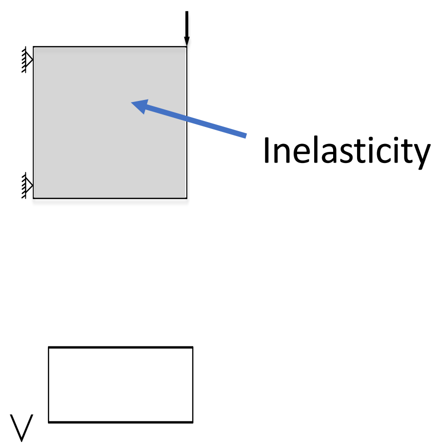
Issue: ductility of cantilever test is larger than ductility of a full diaphragm system
Task: develop method to use cantilever test data to calculated system ductility

source: SDII

Source of difference in ductility

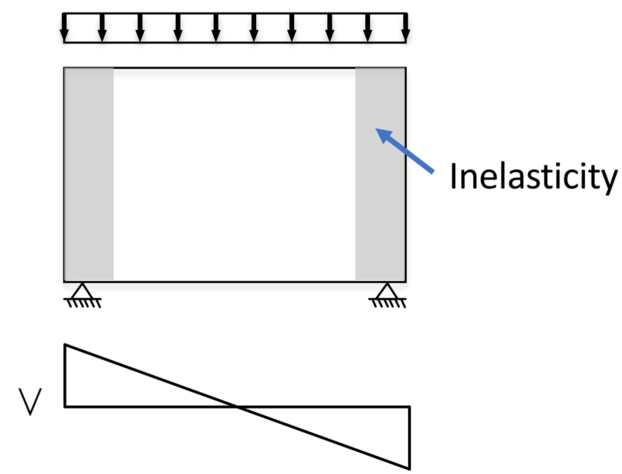
Cantilever specimen – constant shear and distributed inelasticity throughout
Diaphragm system – varying shear and inelasticity will concentrate in end regions

Cantilevered diaphragm test



Shear distribution: Uniform shear

Simply supported diaphragm



Shear distribution: linear variation

Conclusion: $\mu_{subassembly} > \mu_{system}$

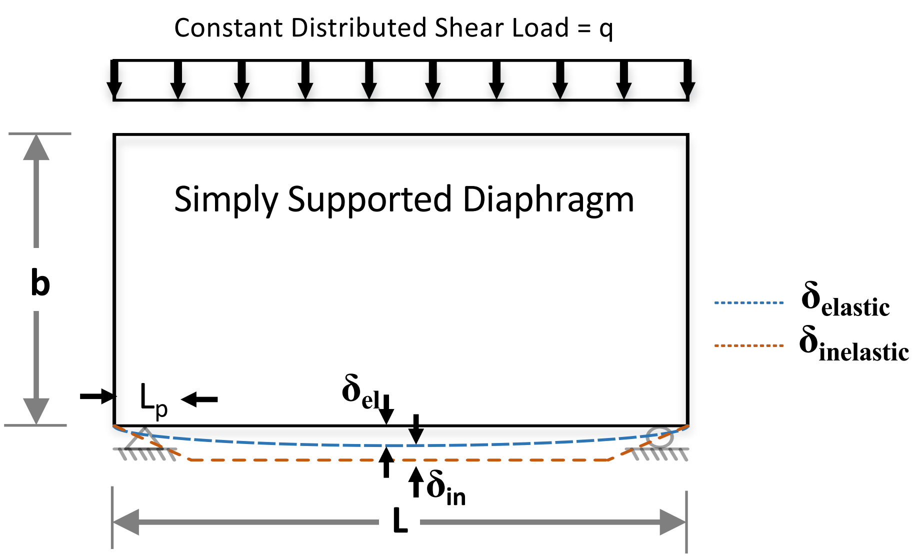
source: SDII

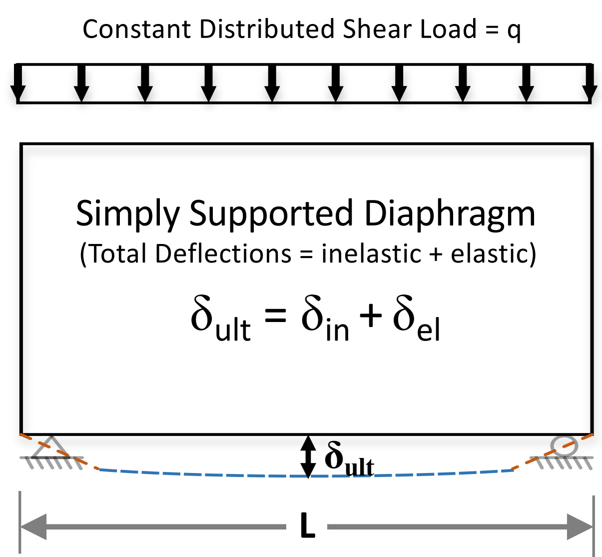
Resolution: estimate elastic and inelastic δ

Deflections and ductility will differ from subassembly to system → $\mu_{subassembly} \neq \mu_{system}$

$$\mu_{\text{system}} = \frac{\delta_{ult}}{\delta_y} = \frac{\delta_{in} + \delta_{el}}{\delta_{el}}$$

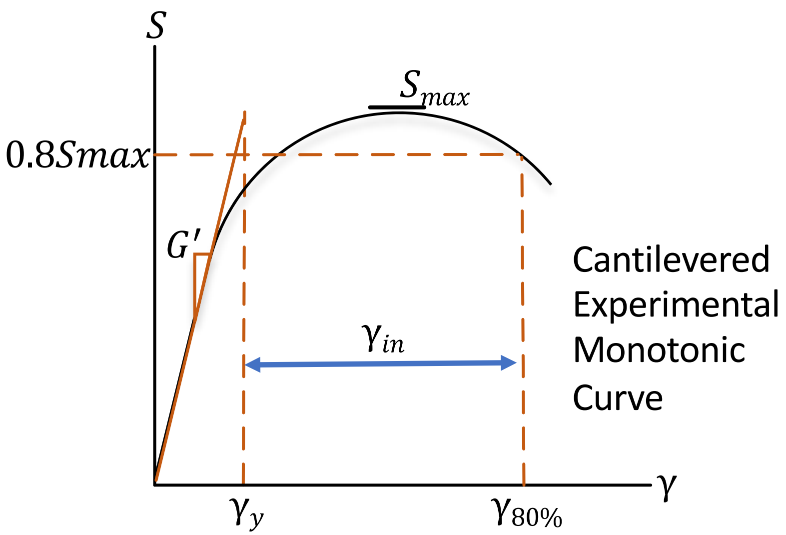
Find δ_{in} and δ_{el}





source: SDII

Resulting Equation for Ductility and R_s



Obtain $\mu_{sub} = \gamma_{80\%} / \gamma_y$ from test

$$\gamma_{in} = \gamma_y(\mu_{sub} - 1)$$

$$\delta_{in} = \gamma_{in} L_p = \gamma_y L_p (\mu_{sub} - 1)$$

$$\underbrace{\delta_{in} = \gamma_y L_p (\mu_{sub} - 1) \qquad \delta_{el} = \frac{S_{max} L}{4G'} = \frac{\gamma_y L}{4}}_{\delta_{ult} = \delta_{in} + \delta_{el}}$$

$$\boxed{\boldsymbol{\mu_{system} = \frac{\delta_{ult}}{\delta_{el}} = 1 + 4(\mu_{sub} - 1)\left(\frac{L_p}{L}\right)}}$$

- System ductility depends on L_p/L, not L
- Will need to assume a plastic zone length L_p/L

$$\mathbf{R}_{\mu\_system} = \sqrt{2\mu_{system} - 1} \ \boldsymbol{or}\ \mu_{system}$$

(depending on period)

$\mathbf{R_s = R_\Omega R}_{\mu\_system}$ $\quad$ R_Ω same as test

source: SDII

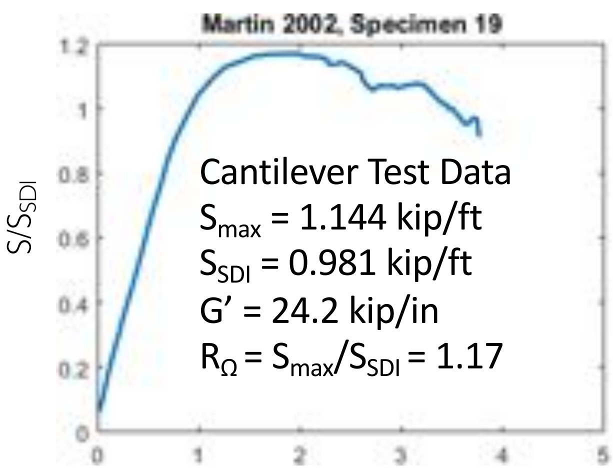
R_S – Example, Mechanical Fasteners Bare Deck Diaphragm (1/2)

$R_s = R_\mu R_\Omega$

- 12" fastener spacings
- 20 gauge deck
- Monotonic loading
- 12' span, 20' depth

<u>PAF Structural Fasteners, Screwed Sidelap</u>

Martin 2002, spec. 19



γ / γ_y

Ductility of subassembly alone:

$\mu_{sub} = 3.76$

$$\boldsymbol{\mu_{system} = 1 + 4(\mu_{sub} - 1)\left(\frac{L_p}{L}\right)}$$

Assume plastic zone is 10% of the diaphragm span, Lp/L = 0.10

$$\mu_{system} = 1 + 4(3.76 - 1)(0.10)$$

$$\mu_{system} = 2.10$$

Ductility of the full diaphragm system

source: SDII

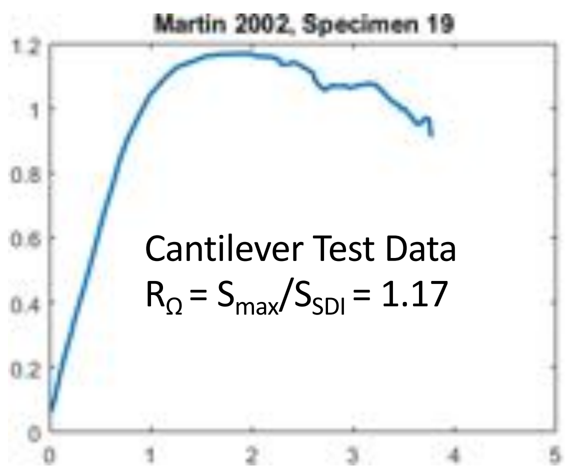
R_s – Example, Mechanical Fasteners Bare Deck Diaphragm (2/2)

$\mathbf{R_s = R_\mu R_\Omega}$

- 12" fastener spacings
- 20 gauge deck
- Monotonic loading
- 12' span, 20' depth

<u>PAF Structural Fasteners, Screwed Sidelap</u>

Martin 2002, spec. 19



S/S_{SDI}

γ / γ_y

Cantilever Test Data

$R_\Omega = S_{max}/S_{SDI} = 1.17$

$\mu_{system} = 2.11$

Medium Period

$R_\mu = \sqrt{2\mu_{system} - 1}$

$= \sqrt{2 * 2.10 - 1} =$

1.79

$\mathbf{R_s = R_\mu R_\Omega = 2.09}$

Long Period

$R_\mu = \mu_{system}$

$= 2.10$

$\mathbf{R_s = R_\mu R_\Omega = 2.46}$

source: SDII

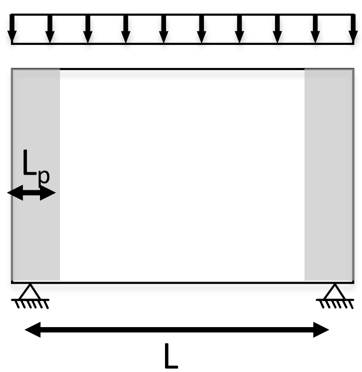
R_s based on cantilever test database

PAF/screw data only

$\mu_{sub} = \mu_c = 2.9$ for PAF/Screw in SDII database

$R_{s\Omega}$ =1.2 for PAF/Screw in SDII database

	$R_s=R_{s\mu}R_{s\Omega}$	
L_p/L	medium T	long T
0.05	1.7	1.6
0.1	2.2	1.9
0.15	2.6	2.2
0.2	3.1	2.4



source: new work

- Literature review and engineering judgment set initial L_p/L as 0.1
- Simulations conducted herein show L_p/L in the range 0.05 to 0.15 in Zone 1 and additional inelastic deformation in other roof zones.
- Within only first zone if we consider $L_p/2L_{Zone1}$ to be the relevant length for ductility and L_p=0.05L to 0.15L then we get $L_p/2L_{Zone1}$ =0.09 to 0.28
- R_s of 2.5 is proposed currently for the ballot, this is less than the subsystem R, but not unduly so.

RWFD Diaphragm Design (R_{diaph})

..

FEMA P-1026 Alternative Design

Key Features

- Roof is its own SDOF system
  - Roof T far enough from vertical period that elastic behavior is distinct
  - Use roof T and separate spectra
  - Assume forces from roof must be carried down to building after diaphragm ductility accounted for (two-stage analysis)
- Protect the perimeter of the roof to drive inelasticity inward/away from walls
  - Account for inelasticity in the roof and allow the roof forces to be reduced by R_{diaph}=4.5
  - Near the edge, create a zone that has 50% higher demands

source: FEMA P-1026

FEMA P-1026 archetype performance

Conventional Design

Archetype ID	Design configuration				Collapse margin parameters				Acceptance check	
	Building size	Diaphragm aspect ratio	Diaphragm construction	Seismic SDC	CMR	μ_T	SSF	ACMR	Accept. ACMR	Pass/Fail
Performance Group No. PG-5E (Steel, Large Building, Welds and Button Punches as sidelap Connectors, Existing Design)										
HSL_21_W_WB_RW4_01	Large	2:1	Steel	D_{max}	0.99	8.09	1.34	1.33	1.73	Fail
HSL_12_W_WB_RW4_01	Large	1:2	Steel	D_{max}	1.90	8.26	1.33	2.53	1.73	Pass
HSL_11_W_WB_RW4_01	Large	1:1	Steel	D_{max}	0.95	8.16	1.33	1.27	1.73	Fail
Mean of Performance Group:					**1.28**	**8.17**	**1.33**	**1.71**	**2.30**	**Fail**
Performance Group No. PG-6E (Steel, Large Building, Screws as sidelap Connectors, Existing Design)										
HSL_21_P_S_RW4_01	Large	2:1	Steel	D_{max}	1.23	8.24	1.35	1.67	1.73	Fail
HSL_12_P_S_RW4_01	Large	1:2	Steel	D_{max}	2.07	8.14	1.33	2.75	1.73	Pass
HSL_11_P_S_RW4_01	Large	1:1	Steel	D_{max}	1.13	8.26	1.36	1.53	1.73	Fail
HSL_11_S_S_RW4_01	Large	1:1	Steel	D_{max}	1.15	8.01	1.33	1.53	2.73	Fail
Mean of Performance Group:					**1.40**	**8.16**	**1.34**	**1.87**	**2.30**	**Fail**
Performance Group No. PG-7E (Steel, Small Building, Button Punches as sidelap Connectors, Existing Design)										
HSS_11_W_B_RW4_01	Small	1:1	Steel	D_{max}	1.73	7.94	1.32	2.28	1.73	Pass
HSS_21_W_B_RW4_01	Small	2:1	Steel	D_{max}	1.42	8.05	1.33	1.89	1.73	Pass
HSS_12_W_B_RW4_01	Small	1:2	Steel	D_{max}	1.90	7.91	1.32	2.51	1.73	Pass
Mean of Performance Group:					**1.68**	**7.97**	**1.32**	**2.23**	**2.30**	**Fail**
Performance Group No. PG-8E (Steel, Small Building, Screws as sidelap Connectors, Existing Design)										
HSS_11_P_S_RW4_01	Small	1:1	Steel	D_{max}	1.55	8.02	1.33	2.07	1.73	Pass
HSS_11_S_S_RW4_01	Small	1:1	Steel	D_{max}	1.43	8.15	1.33	1.91	1.73	Pass
HSS_21_P_S_RW4_01	Small	2:1	Steel	D_{max}	1.33	8.33	1.33	1.76	1.73	Pass
HSS_12_P_S_RW4_01	Small	1:2	Steel	D_{max}	1.71	8.25	1.33	2.27	1.73	Pass
HSS_21_S_S_RW4_01	Small	2:1	Steel	D_{max}	1.25	7.85	1.32	1.65	1.73	Fail
HSS_12_S_S_RW4_01	Small	1:2	Steel	D_{max}	1.42	8.06	1.33	1.89	1.73	Pass
Mean of Performance Group:					**1.45**	**8.11**	**1.33**	**1.92**	**2.30**	**Fail**

FEMA P-1026 archetype performance

CFSRC

Revised Design

Archetype ID	Design configuration				Collapse margin parameters				Acceptance check	
	Building size	Diaphragm aspect ratio	Diaphragm construction	Seismic SDC	CMR	β_T	SSF	ACMR	Accept. ACMR	Pass/Fail
Performance Group No. PG-5N (Steel, Large Building, Welds and Button Punches as sidelap Connectors, New Design)										
HSL_11_W_WB_RD4.5-1.5_01	Large	1:1	Steel	D_{max}	1.55	8.35	1.36	2.10	1.73	Pass
HSL_12_W_WB_RD4.5-1.5_01	Large	1:2	Steel	D_{max}	2.53	8.14	1.33	3.36	1.73	Pass
HSL_21_W_WB_RD4.5-1.5_01	Large	2:1	Steel	D_{max}	1.41	7.84	1.36	1.89	1.73	Pass
Mean of Performance Group:					**2.04**	**8.11**	**1.35**	**2.73**	**2.30**	**Pass**
Performance Group No. PG-6N (Steel, Large Building, Screws as sidelap Connectors, New Design)										
HSL_21_P_S_RD4.5-1.5_01	Large	2:1	Steel	D_{max}	1.47	8.38	1.37	2.02	1.73	Pass
HSL_12_P_S_RD4.5-1.5_01	Large	1:2	Steel	D_{max}	2.56	8.15	1.36	3.48	1.73	Pass
HSL_11_P_S_RD4.5-1.5_01	Large	1:1	Steel	D_{max}	2.12	8.51	1.37	2.90	1.73	Pass
HSL_11_S_S_RD4.5-1.5_01	Large	1:1	Steel	D_{max}	1.96	8.11	1.36	2.67	1.73	Pass
Mean of Performance Group:					**2.03**	**8.29**	**1.37**	**2.77**	**2.30**	**Pass**
Performance Group No. PG-7N (Steel, Small Building, Button Punches as sidelap Connectors, New Design)										
HSS_11_W_B_RD4.5-1.5_01	Small	1:1	Steel	D_{max}	2.11	7.88	1.32	2.79	1.73	Pass
HSS_21_W_B_RD4.5-1.5_01	Small	2:1	Steel	D_{max}	1.85	7.94	1.32	2.45	1.73	Pass
HSS_12_W_B_RD4.5-1.5_01	Small	1:2	Steel	D_{max}	2.44	7.74	1.32	3.22	1.73	Pass
Mean of Performance Group:					**2.14**	**7.85**	**1.32**	**2.82**	**2.30**	**Pass**
Performance Group No. PG-8N (Steel, Small Building, Screws as sidelap Connectors, New Design)										
HSS_11_P_S_RD4.5-1.5_01	Small	1:1	Steel	D_{max}	1.98	8.17	1.33	2.63	1.73	Pass
HSS_11_S_S_RD4.5-1.5_01	Small	1:1	Steel	D_{max}	1.72	7.99	1.33	2.29	1.73	Pass
HSS_21_P_S_RD4.5-1.5_01	Small	2:1	Steel	D_{max}	1.59	8.09	1.33	2.11	1.73	Pass
HSS_12_P_S_RD4.5-1.5_01	Small	1:2	Steel	D_{max}	2.01	8.14	1.33	2.67	1.73	Pass
HSS_21_S_S_RD4.5-1.5_01	Small	2:1	Steel	D_{max}	1.63	7.96	1.33	2.17	1.73	Pass
HSS_12_S_S_RD4.5-1.5_01	Small	1:2	Steel	D_{max}	1.92	7.98	1.33	2.55	1.73	Pass
Mean of Performance Group:					**1.81**	**8.06**	**1.33**	**2.40**	**2.30**	**Pass**

source: Koliou (2014)

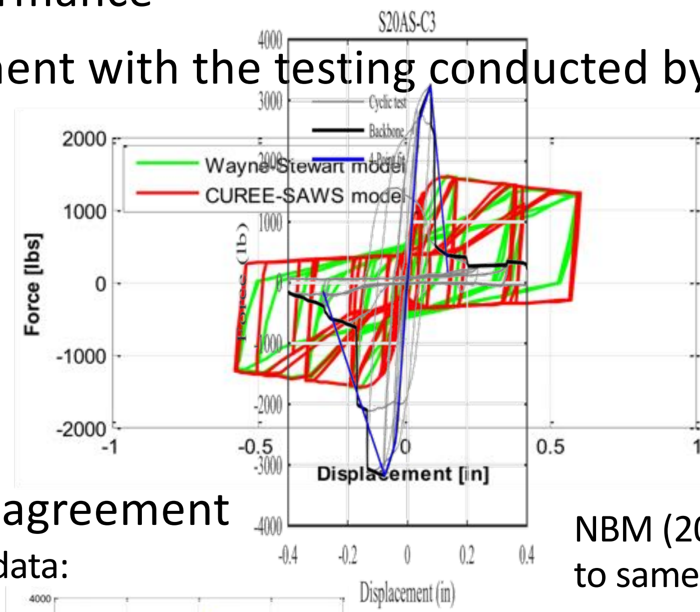
- All steel performance groups are predicted to have acceptable CMR
- Large, 2:1 dir., SDC D remains the most critical

- FEMA P-1026 model supports the use of the alternative diaphragm design approach, R_{diaph}=4.5

Details in the FEMA P-1026/Koliou (2014) model

- Connector models integral to the model performance
- PAF and screw models in good general agreement with the testing conducted by NBM (2018) and reported herein
- Sidelap weld models not in good agreement

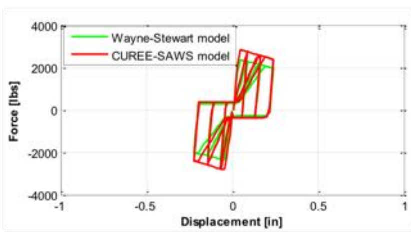
Koliou (2014) model fit to Rogers and Tremblay (2003a) data available at the time:



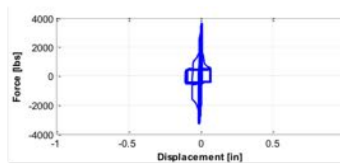
NBM (2018) test to same scale.

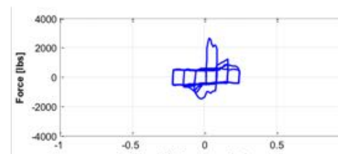
- Some structural weld models also not in good agreement

Fit used in model:



source data:





Residual in test data causing fit to have too slow degradation

- Conclusion, set aside the weld models. Could be re-run, for now rely on only the mechanically fastened deck and related modeling

FEMA P-1026 archetype performance

Revised Design

Archetype ID	Design configuration				Collapse margin parameters				Acceptance check	
	Building size	Diaphragm aspect ratio	Diaphragm construction	Seismic SDC	CMR	β_T	SSF	ACMR	Accept. ACMR	Pass/Fail
Performance Group No. PG-5N (Steel, Large Building, Welds and Button Punches as sidelap Connectors, New Design)										
HSL_11_W_WB_RD4.5-1.5_01	Large	1:1	Steel	D_{max}	1.55	8.35	1.36	2.10	1.73	Pass
HSL_12_W_WB_RD4.5-1.5_01	Large	1:2	Steel	D_{max}	2.53	8.14	1.33	3.36	1.73	Pass
HSL_21_W_WB_RD4.5-1.5_01	Large	2:1	Steel	D_{max}	1.41	7.84	1.36	1.89	1.73	Pass
Mean of Performance Group:					**2.04**	**8.11**	**1.35**	**2.73**	**2.30**	**Pass**
Performance Group No. PG-6N (Steel, Large Building, Screws as sidelap Connectors, New Design)										
HSL_21_P_S_RD4.5-1.5_01	Large	2:1	Steel	D_{max}	1.47	8.38	1.37	2.02	1.73	Pass
HSL_12_P_S_RD4.5-1.5_01	Large	1:2	Steel	D_{max}	2.56	8.15	1.36	3.48	1.73	Pass
HSL_11_P_S_RD4.5-1.5_01	Large	1:1	Steel	D_{max}	2.12	8.51	1.37	2.90	1.73	Pass
HSL_11_S_S_RD4.5-1.5_01	Large	1:1	Steel	D_{max}	1.96	8.11	1.36	2.67	1.73	Pass
Mean of Performance Group:					**2.03**	**8.29**	**1.37**	**2.77**	**2.30**	**Pass**
Performance Group No. PG-7N (Steel, Small Building, Button Punches as sidelap Connectors, New Design)										
HSS_11_W_B_RD4.5-1.5_01	Small	1:1	Steel	D_{max}	2.11	7.88	1.32	2.79	1.73	Pass
HSS_21_W_B_RD4.5-1.5_01	Small	2:1	Steel	D_{max}	1.85	7.94	1.32	2.45	1.73	Pass
HSS_12_W_B_RD4.5-1.5_01	Small	1:2	Steel	D_{max}	2.44	7.74	1.32	3.22	1.73	Pass
Mean of Performance Group:					**2.14**	**7.85**	**1.32**	**2.82**	**2.30**	**Pass**
Performance Group No. PG-8N (Steel, Small Building, Screws as sidelap Connectors, New Design)										
HSS_11_P_S_RD4.5-1.5_01	Small	1:1	Steel	D_{max}	1.98	8.17	1.33	2.63	1.73	Pass
HSS_11_S_S_RD4.5-1.5_01	Small	1:1	Steel	D_{max}	1.72	7.99	1.33	2.29	1.73	Pass
HSS_21_P_S_RD4.5-1.5_01	Small	2:1	Steel	D_{max}	1.59	8.09	1.33	2.11	1.73	Pass
HSS_12_P_S_RD4.5-1.5_01	Small	1:2	Steel	D_{max}	2.01	8.14	1.33	2.67	1.73	Pass
HSS_21_S_S_RD4.5-1.5_01	Small	2:1	Steel	D_{max}	1.63	7.96	1.33	2.17	1.73	Pass
HSS_12_S_S_RD4.5-1.5_01	Small	1:2	Steel	D_{max}	1.92	7.98	1.33	2.55	1.73	Pass
Mean of Performance Group:					**1.81**	**8.06**	**1.33**	**2.40**	**2.30**	**Pass**

Archetype Findings Extrapolated Back to FEMA P-1026 Study

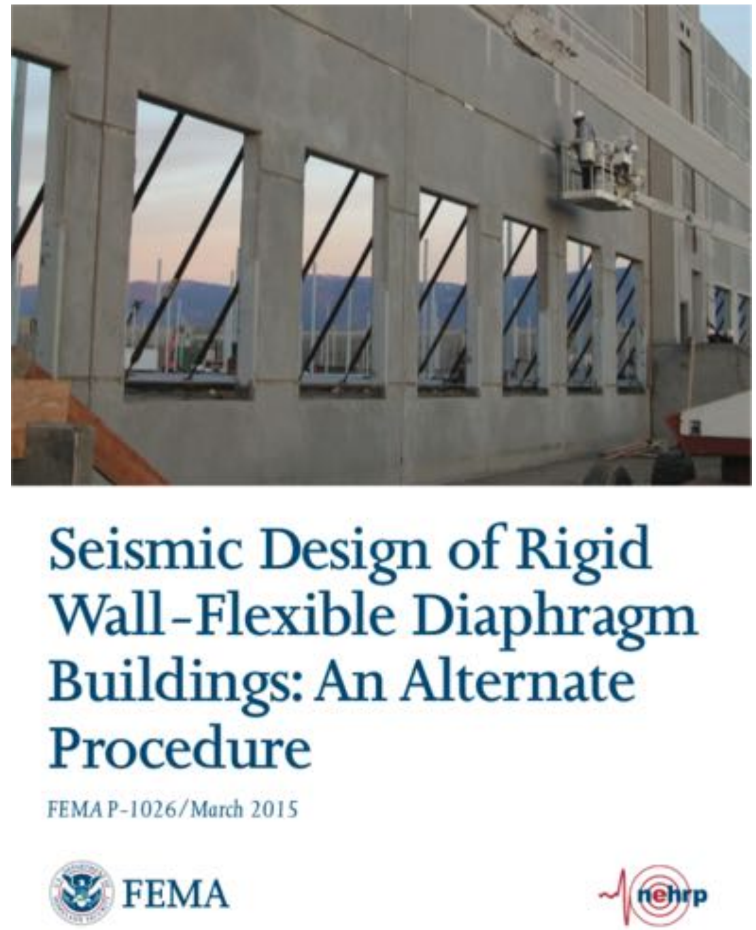
	Comment
PG-5N	Given recent fastener testing, set this result aside, consider for future model updates.
PG-6N	
PG-7N	Given recent fastener testing, set this result aside, consider for future model updates.
PG-8N	

FEMA P-1026 and steel concerns

"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



source: FEMA P-1026 (2015)

Adressing FEMA P-1026 report concerns

CFSRC

concerns

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

resolution

1. Created 3D model to more fully explore large scale diaphragms, identified conditions where ductility is lost and separated
2. Examined test-to-predicted strength for cyclic results
3. Increased the cyclic test database substantially
4. Identified connectors with best ductility and integrated real behavior into model
5. Compiled available testing and utilized data to inform modeling and design results
6. Creating a performance pathway for proprietary systems to be included

Wrapping Up

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Forthcoming Ballots for Bare Steel Deck Diaphragms

CFSRC

- Definition of Special Seismic Detailing
  - Prescriptive PAF/Screw
  - Performance-Based: Cyclic Cantilever Test or Connectors + Simulation
- Conventional Diaphragm Design (R)
  - If ductility needed - SDC trigger for this? (otherwise no change)
  - Special – no change,
  - Ordinary – design at Ωo levels
- Modifications for Alternative Diaphragm Design (R_s)
  - Special R_s=2.5
  - Ordinary R_s=1.0
- Modifications for RWFD Design
  - Special R_{diaph} = 4.5
  - Ordinary R_{diaph} = 1.5
  - Follow same procedure as adopted for wood

Conclusions

- We have a path forward
- Setting a target for ductile steel deck diaphragm performance and pegging it to the favorable behavior of typical PAF/screw assemblies provides a useful organizing principle, implemented correctly it should benefit the practice and the public, and not stifle innovation
- Even with the proposals a number of issues need (at least long term) resolution: diaphragm collapse criteria, diaphragm drift vs. vertical (gravity system) drift, anchorage forces, more consideration of out-of-plane forces on connectors
- Existing data shows that there is more and varied potential for inelastic steel deck diaphragm performance than is currently being exploited; modified details, profiles, roof zoning, all warrant study
- Existing (R) and new design philosophies (R_s, R_{diaph}) rely on largely conservative and isolated ideas of inelastic building-diaphragm interaction, these deserve further study going forward

References

- Not a complete literature review – only references to support materials in the presentation, referenced standards not detailed here.
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