COLD-FORMED STEEL RESEARCH CONSORTIUM

Steel Diaphragm Innovation Initiative
Workshop Report

B.W. Schafer, W.S. Easterling, R. Sabelli, M.R. Eatherton, J.F. Hajjar

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SDII
Steel Diaphragm Innovation Initiative

CFSRC Information

The Cold-Formed Steel Research Consortium (CFSRC) is a multi-institute consortium of university researchers dedicated to providing world-leading research that enables structural engineers and manufacturers to realize the full potential of structures utilizing cold-formed steel. More information can be found at www.cfsrc.org. All CFSRC reports are hosted permanently by the Johns Hopkins University library in the DSpace collection: https://jscholarship.library.jhu.edu/handle/1774.2/40427.

SDII Information

The Steel Diaphragm Innovation Initiative (SDII) is a multi-year industry-academic partnership to advance the seismic performance of steel floor and roof diaphragms utilized in steel buildings through better understanding of diaphragm-structure interaction, new design approaches, and new three-dimensional modeling tools that provided enhanced capabilities to designers utilizing steel diaphragms in their building systems. SDII was created through collaboration between the American Iron and Steel Institute and the American Institute of Steel Construction with contributions from the Steel Deck Institute, the Metal Building Manufacturers Association, and the Steel Joist Institute in partnership with the Cold-Formed Steel Research Consortium; including, researchers from Johns Hopkins University, Virginia Tech, Northeastern University, and Walter P Moore.

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1 Executive Summary

Researchers from the Steel Diaphragm Innovation Initiative (SDII) led a one day workshop in Burlingame California on 10 January 2019 for thirty-five engineering participants to discuss progress to date in the SDII effort, receive feedback on existing and planned future work, and to collectively identify key challenges and innovation opportunities related to the seismic performance of buildings employing bare or concrete-filled steel deck diaphragms.

The SDII research team summarized current efforts in structural experiments across a variety of scales, modeling across scales, codes and standards for demand and capacity, and innovation opportunities. The presentation slides are provided in Appendix 1 and 2. For bare steel deck diaphragms existing testing, new testing, and simulation have been employed to develop improved design provisions for AISC 342/ASCE 41, AISI S310, AISI S400, and NEHRP/ASCE7. These new provisions recognize the conditions in which bare steel deck diaphragms can provide adequate ductility, deformation, and residual force capacity – and when these performance conditions are met, provide appropriate reductions in diaphragm demands. For concrete-filled steel deck diaphragms, new testing including: monotonic pushout tests, cyclic pushout tests, and full-scale cantilever diaphragm tests are all underway. Combined with existing testing the results are providing improved stiffness and strength provisions for AISI S310, and will also impact AISC 341, AISC 360, and ASCE7. The workshop participants were brought up to speed on all of these issues and more, expressed support for the SDII effort, and then engaged in an active exercise to explore challenges and opportunities in steel deck diaphragms.

Workshop participants were provided a questionnaire in advance and given time during the meeting to individually answer ten questions related to challenges and nine questions related to innovation (see Appendix 3). Participants provided their complete response to the SDII team for later analysis, and then during the workshop engaged in small groups to develop an initial set of priorities. The prioritized challenges developed during the workshop covered: codification needs related to capacity prediction; improved models, particularly for diaphragm demands; workflow and practice-oriented (time and fee) challenges, detailing challenges, and how to better handle irregularities. The deeper analysis of the complete participant responses highlighted two major additional specific challenges: (1) even the nation's most accomplished seismic building engineers do not have a consistent understanding of whether or not inelasticity is expected in the seismic response of building diaphragms, (2) while some engineers rely extensively on supplemental reinforcement in concrete-filled steel deck diaphragms both to improve the strength and provide the necessary chord and collector capacity, other engineers have specific concerns about confinement in these systems and will not employ them in their designs.

A similar process was followed during the workshop and in later analysis for the questions related to innovation. During the workshop the prioritized points regarding innovation centered on three groups: technological innovation, overall innovation, and engineer support/workflow innovations. The primary ideas for technological innovation focused on improved connectors, and the potential for the integration of discrete energy dissipation devices (structural fuses). A significant point of discussion with respect to innovation is the need to have strong engineering support and efficient and simple workflows. Engineers found that the tools to model diaphragms were lacking in nearly every regard, and innovation is needed. A deeper analysis of the participant responses identified that innovation in diaphragms is hampered by a definitive lack of knowledge with respect to the behavior of building systems with inelasticity in both the vertical and horizontal lateral force resisting system. If this behavior is understood then software improvements (that support design) and specific technological innovation (isolation, improved damping, optimized deck profiles) can have impact.

2 Workshop Overview

On 10 January 2019, 35 participants, and 5 presenters convened in Burlingame, CA near SFO airport to discuss challenges and innovation in steel deck diaphragms, as shown in Figure 1. The workshop provided an overview of research conducted to date associated with the Steel Diaphragm Innovation Initiative (SDII) and provided an opportunity for the participants to give feedback on current research, future research plans, and current and proposed proposals for related codes and standards. Attendees participated in a detailed questionnaire related to steel deck diaphragm challenges and innovation. Results of the questionnaire were prioritized during the workshop and investigated in detail as reported herein.

Figure 1. Workshop venue and participants

SDII is a multi-year industry-academic partnership to advance the seismic performance of steel floor and roof diaphragms utilized in steel buildings through better understanding of diaphragm-structure interaction, new design approaches, and new three-dimensional modeling tools that provided enhanced capabilities to designers utilizing steel diaphragms in their building systems. SDII was created through collaboration between the American Iron and Steel Institute (AISI) and the American Institute of Steel Construction (AISC) with contributions from the Steel Deck Institute (SDI), the Metal Building Manufacturers Association (MBMA), and the Steel Joist Institute (SJI) in partnership with the Cold-Formed Steel Research Consortium (CFSRC); including, researchers from Johns Hopkins University (JHU), Virginia Tech (VT), Northeastern University (NEU), and Walter P Moore (WPM).

2.1 Schedule

The schedule for the workshop was as follows:

8:00 – 8:05	Introduction (Sputo)
8:05 – 10:00	Overview of SDII (Schafer)
	Compiling and analyzing existing data (Eatherton)
	New cyclic testing to characterize performance across scales
	Connector (fastener shear) (Schafer)
	Interface (pushout) (Hajjar)
	Diaphragm (cantilever) (Easterling)
	Planned large scale testing (Hajjar)
	Leveraging Simulation
	Vertical vs. horizontal LFRS (Schafer)
	Building scale archetype simulations (Eatherton)
	Bringing fracture into models (Hajjar)
	Optimization (Schafer)
	Conclusions (Schafer)
10:00 – 10:30	**Break**
10:30 – 11:30	SDII Codes and Standards - Proposals and Future Pathways
	Overview (Schafer)
	This code cycle
	Bare deck (Schafer), Concrete-filled (Easterling, Eatherton)
	Future code cycles (many questions here!)
	ELF Demands (Schafer), Model/performance-based (Eatherton)
	P695 for diaphragms?, Testing standards?, Irregularities?, C&C?
11:30 – 11:35	Introduction to SDII Questionnaires – Challenges and Innovation
11:35 – 12:00	Individual Time to work on questionnaires
12:00 – 12:45	**Lunch**
12:45 – 1:15	Facilitated small group work, posting of key points (All)
1:15 – 1:30	Designers Perspective on Challenges (Sabelli)
1:30 – 2:10	Discussion and consensus on challenges (Sabelli + Eatherton)
2:10 – 2:25	Designers Perspective on Innovation (Sabelli)
2:25 – 2:55	Discussion and consensus on innovation (Sabelli + Hajjar)
2:55 – 3:00	Wrap-up and next steps (Schafer)

2.2 Participants

The SDII workshop attendees included the following participants:

- Rafael Sabelli, Walter P. Moore
- Ben Schafer, JHU
- Matt Eatherton, VT
- Sam Easterling, VT
- Jerrry Hajjar, NEU
- Jim Fisher, Steel Joist Institute
- Pat McManus, Martin/Martin
- Emily M. Guglielmo Martin/Martin
- John Hooper, MKA
- David Bonneville, Degenkolb
- Jim Malley, Degenkolb
- Tom Xia, DCI Engineers
- Ron Hamburger, SGH
- Kevin Moore, SGH
- Kelly Cobeen, WJE
- Tom Sabol, Englekirk
- Rob Madsen, Devco
- Bob Bachman, retired Fluor
- Jim Harris, Harris and Co.
- John Rolfes, CSD
- John Lawson, CalPoly San Luis Obispo
- Robert Tremblay, Polytechnique Montreal
- Colin Rogers, McGill University
- Chia-Ming Uang, UCSD
- Greg Deierlein, Stanford
- Robert Fleischman, UA
- Roy Lobo, OSHPD
- Carrie Johnson, Wallace Engineering
- Dave Durington, Johnson & Burkholder
- Igor Marinovic, Blue Scope Buildings
- Mark Detwiler, NCI
- Jeff Martin, Verco
- Patrick Bodwell, Verco
- Dave Golden, ASC
- Bob Hanson, UMich, FEMA

In addition, the following industry representatives attended the workshop:

- Tom Sputo, SDI
- JP Cardin, AISI
- Bonnie Manley, AISI
- Devin Huber, AISC
- Lee Shoemaker, MBMA

The following individuals were invited, but unable to attend the workshop:

- Ken Charles, SJI
- Dominic Kelly, SGH
- Larry Kruth, AISC
- Tom Schlafly, AISC
- Mike Mahoney, FEMA
- Mike Tong, FEMA
- Ayse Hortacsu, ATC
- Walter Schultz, Vulcraft

3 SDII Research Summary

The SDII research team provided a summary of research to date. The slides from this presentation are provided in Appendix 1. The overall topics covered included the following:

- Overview of SDII (Schafer)
- Compiling and analyzing existing data (Eatherton)
- New cyclic testing to characterize performance across scales
  - Connector (fastener shear) (Schafer)
  - Interface (pushout) (Hajjar)
  - Diaphragm (cantilever) (Easterling)
  - Planned large scale testing (Hajjar)
- Leveraging Simulation
  - Vertical vs. horizontal LFRS (Schafer)
  - Building scale archetype simulations (Eatherton)
  - Bringing fracture into models (Hajjar)
  - Optimization (Schafer)
- Conclusions (Schafer)

In brief discussion following the presentation the participants expressed an overall appreciation for the work that had been engaged to date and the direction of the effort. The participants were interested in why OCBF had been considered for one of the SDII building archetypes - and there was disagreement as to whether this was a good or bad decision. Building systems where the vertical systems were in the interior as well as the perimeter were called out as being of specific interest. There was a great deal of interest in re-thinking the capacity calculations for steel deck with concrete fill – and whether a more mechanics-oriented strut and tie model could be provided. Comments were made regarding the planned large scale testing – particularly with respect to challenges in separating the columns. In addition, comments were made expressing interest in learning more about the simulation results comparing inelasticity in the vertical and horizontal lateral force resisting systems.

4 SDII Codes and Standards Efforts

The SDII research team provided a comprehensive summary of efforts related to codes and standards adoption. The slides from this presentation are provided in Appendix 2. The overall topics covered including the following

SDII Codes and Standards - Proposals and Future Pathways
Overview (Schafer)
This code cycle
    Bare deck (Schafer),
    Concrete-filled (Easterling, Eatherton)
Future code cycles (many questions here!)
    ELF Demands (Schafer), Model/performance-based (Eatherton)
    P695 for diaphragms?, Testing standards?, Irregularities?, C&C?

Additional discussion from the participants focused on understanding the bare steel deck diaphragm proposals since they were the most developed. Participants expressed how times have changed for bare steel deck – where once welds were the preferred solution and mechanical fastening considered secondary, the situation is now reversed.

Participants also discussed the proposed strength predictions for steel deck with concrete fill in AISI S310 and whether these would provide an appreciably different strength prediction from the use of ACI 318 and only considering the concrete above the deck flutes. This question was not addressed at the time, but was examined at the February meeting of the AISI subcommittee in charge of AISI S310 development. At that meeting it was shown that for fill with only temperature and shrinkage steel the difference in the predictions is significant. Work is underway to finalize the proposed provisions in the AISI subcommittee.

5 Questionnaire and Small Group Work

All of the workshop participants were asked to complete the Questionnaire provided in Appendix 3. The questionnaire covered topics related to major challenges and potential for innovation in steel deck diaphragms. Participants were provided the questionnaire in advance, and given some time to complete the questionnaire during the workshop.

Upon completion of the questionnaire participants were placed in small groups of 4. The groups were asked to identify one or two key challenges and one or two key innovations and put these thoughts onto post-it notes and post for all participants to see on the boards shown in Figure 2.

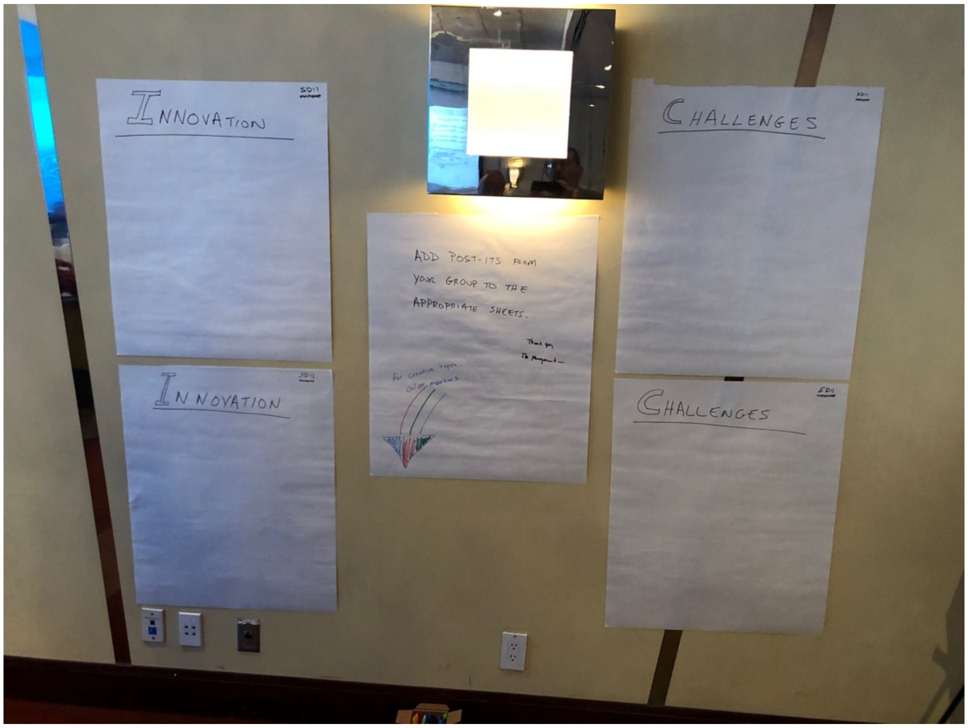


Figure 2 Innovation and Challenge Summary Boards

6 Priorities for Challenges and Innovation as Facilitated at the Workshop

6.1 Challenges

For the identified challenges Rafael Sabelli and Matt Eatherton organized the responses and facilitated a discussion. The original post-it notes in their groupings from the workshop are provided in Figure 3. The post-it notes were typed up and the groupings labeled in the following:

Group 1: Codification Needs

- More codification of strut and tie analysis
- Horizontal truss diaphragms need to be addressed (tension-only x-braces)
- Architectural impacts, Non-uniform structures, irregularities
- Are we limiting or precluding the use of welds in seismic areas for diaphragms?

Group 2: Models and Diaphragm Demands

- Develop reasonable model to determine diaphragm interaction with vertical elements and diaphragm design forces
- Not considering the 3D interaction of horizontal or vertical elements
- Extracting forces from diaphragm models
- Redistributing forces in single story parallel moment frames should be addressed

Group 3: Workflow and Practice Challenges

- Schedule/time constraints to do 3D/nonlinear analysis
- Low rise buildings <4 stories do not need complex 3D analysis
- What real damage has occurred to justify large increases in demand – show me the bodies.

Group 4: Details, Identifying when Diaphragms Matter, More

- Collector design and analysis
- Understanding when diaphragm behavior controls
- Resiliency and inspect-ability
- No automated design checks into current analysis programs

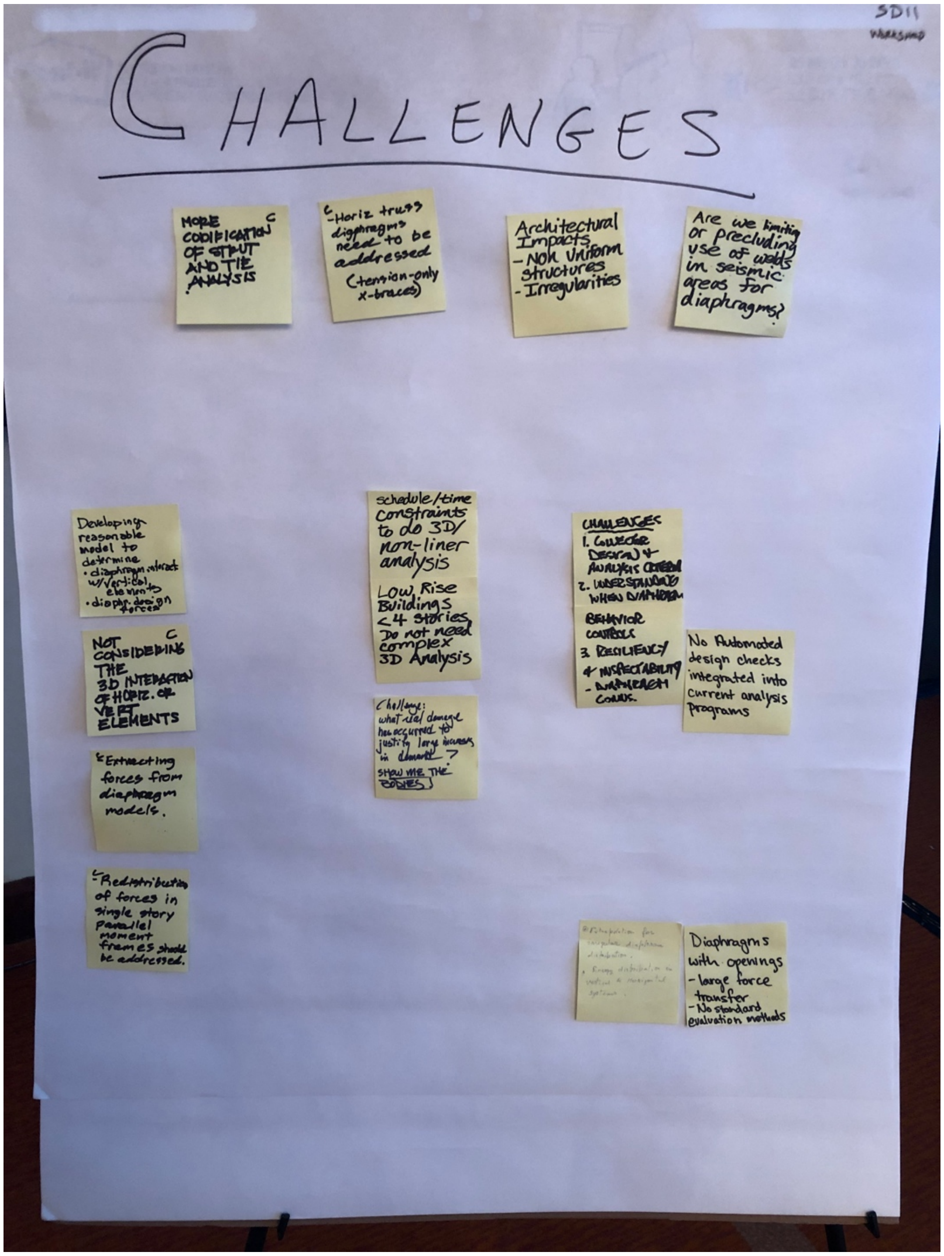
Group 5: Irregularities

- Diaphragms with openings or large force transfers
- Extrapolation for irregular diaphragm distribution
- Energy distribution in vertical and horizontal systems

Sabelli and Eatherton summarized the responses and led a discussion. The participants covered a wide variety of challenges directly related to the identified areas and more broadly.

The participants highlighted that few engineers understand diaphragm design – and that in the main diaphragm design seems disconnected from the general building design process.

The participants highlighted that none of the mainstream structural engineering software provides diaphragm forces. This situation become even more problematic with irregularities. So, engineers do their best to make conservative assumptions.



SDII WORKSHOP

CHALLENGES

MORE CODIFICATION OF STRUT AND TIE ANALYSIS

-Horiz truss diaphragms need to be addressed (tension-only x-braces)

Architectural Impacts
- Non Uniform structures
- Irregularities

Are we limiting or precluding use of welds in seismic areas for diaphragms?

Developing reasonable model to determine
- diaphragm interact w/vertical elements
- diaphr. design forces

NOT CONSIDERING THE 3D INTERACTION OF HORIZ. OR VERT ELEMENTS

Extracting forces from diaphragm models.

-Redistribution of forces in single story parallel moment frames should be addressed.

schedule/time constraints to do 3D/non-liner analysis

Low Rise Buildings <4 stories Do not need complex 3D Analysis

Challenge: what real damage has occurred to justify large increases in demand? SHOW ME THE BODIES

CHALLENGES
1. COLLECTOR DESIGN & ANALYSIS CRITERIA
2. UNDERSTANDING WHEN DIAPHRAGM BEHAVIOR CONTROLS
3. RESILIENCY
4. INSPECTABILITY
- DIAPHRAGM CONNS.

No Automated design checks integrated into current analysis programs

- Extrapolation for irregular diaphragm distribution.
- Energy distribution in vertical & horizontal systems.

Diaphragms with openings
- large force transfer
- No standard evaluation methods

Figure 3. Challenges identified during the workshop

One challenge that was brought forth multiple times and has its own built-in inconsistencies is that (a) many participants don’t trust diaphragm forces coming from ELF-based design, while at the same time (b) equilibrating diaphragm forces with ELF forces, particularly when dealing with transfers and irregularities is important and a major tool that engineer attempt to use in many situations.

The participants struggled to see how capacity-based design is intended to proceed for diaphragms. Is the diaphragm designed to deliver necessary forces to the vertical LFRS only?

What about multiple load paths through the diaphragm? How to assess what element is the fuse and what should be capacity protected in such an indeterminate system?

Participants re-emphasized the challenge with irregularities, particularly plan irregularities. Discussion of plan irregularities led to a highlight of an over-arching pressure: building floor plates are getting more complex, but engineers don't have the fees or tools to deal with this so they desire the simplest possible methods to solve in the shortest possible time. This tension was recognized with respect to many issues surrounding design for irregularities.

Participants could see a future where analysis may be more sophisticated and capable, but felt that the present challenge was to provide simple, preferably linear methods that were accurate enough. Providing performance triggers that might lead the engineer to more sophisticated models was expressed as a potential path to overcome this challenge. Even given these analysis comments several participants expressed that there was still a stark lack of knowledge with respect to the expected demands.

6.2 Innovation

For the identified ideas in innovation Rafael Sabelli and Jerry Hajjar organized the responses and facilitated a discussion. The original post-it notes in their groupings from the workshop is provided in Figure 4. The post-it notes were typed up and the groupings labeled in the following:

Group 1: Technological Innovation
- Improved connections – find a way to make welds safe and economical for seismic
- Attachments of deck edges to sloped and cambered members – need products that are rested and address these well
- Consider use of rebar in composite beam/slabs
- Concept of fuses is a great idea

Group 2: Overall Innovation
- Research that results in more efficient and effective systems
- Bare deck diaphragm system where ductility not always coming from fasteners

Group 3: Engineering Support and Workflow Innovation
- Clear design aids and charts
- Design tools/software to capture real behavior of diaphragms and vertical LFRS
- Maintain a simple design method to address competency and resources of majority of engineers and installers
- Development of simple design methodologies
- Better tools to model diaphragms

Sabelli and Hajjar summarized the responses and led a discussion. The participants covered a wide variety of issues related to innovation and also revisited the challenges with advancing steel deck diaphragms.

Specifically addressing technological innovation beyond the points highlighted above participants expressed an interest in clamped diaphragm systems – and any improvements that could be made for clamping. This was considered a system with positive potential independent from ideas related to deconstruction. Participants also emphasized the idea of moving deformation in bare steel deck diaphragms out of the fasteners and into the deck profile. Challenges with drift amplification in such ductile systems were noted. Participants also emphasized that innovation can come from considering new objectives, e.g considering multiple earthquakes and resiliency and reparability. It was discussed that this would seem to favor separation of the vertical and horizontal system – but this is challenged by the needs of the gravity system.

It was also noted that there is a lack of evidence that steel deck composite floor systems have any over-arching problem and a call to focus on where problems are known, e.g. response of big-box buildings under seismic demand. The counter point to this comment being that the lack of knowledge and long list of challenges identified suggest that current systems may not be efficient nor perform as we intend or expect.

Figure 4. Innovation identified during the workshop

7 Detailed Summary of Questionnaire Response

In addition to the work during the workshop all of the individual questionnaire responses were cataloged and considered. Thus, responses were delved into more detail than was possible during the in person workshop, highlighting quite specific challenges and potential innovations. An organized summary of the identified issues is provided in Appendix 3. Here the overall response of the participants is summarized for each question. First for the challenges, then innovation:

C1. Thinking broadly about the *design* of diaphragms for steel buildings, particularly under seismic demands, what are key challenges engineers face from your perspective?

Engineers noted a large variety of challenges w.r.t diaphragms in steel buildings, major issues exist for: stiffness, strength, demand, irregularities, workflow, training, and best practices. Key among the long list are (a) how to properly calculate and distribute the demand and equilibrate with ELF forces, and (b) how to handle plan irregularities and interior supports. Metal building diaphragms were highlighted as a special case needing improved understanding.

C2. For your design/analysis workflow tell us about how you *model* building diaphragms:

Structural modeling of diaphragms is largely in its infancy. Engineers hope to use rigid or flexible idealizations wherever possible. In-plane linear elastic models are applied for semi-rigid diaphragms sparingly, or when large transfers, openings or other irregularities give the design engineer concern.

C3. When *modeling* the diaphragm of a steel building, what challenges do you face? What challenges do you face/perceive when interpreting your model results to your satisfaction?

Engineers creating models for their diaphragms do not have a high level of confidence that their models are valid and have concerns about how to model almost all aspects of a diaphragm. Even for engineers seeking to use simplified models the preceding concerns are valid. When a model is constructed challenges exist with making the output relevant to the engineer's design.

C4. To what extent do *non-structural* constraints/demands (e.g., fire, acoustics, aesthetics) drive your floor or roof assembly and ultimately your diaphragm design? What challenges do you face with respect to meeting non-structural demands as they relate to the diaphragm?

Fire and sometimes acoustics drives major choices in concrete-filled deck diaphragm design. Vibration can control the design of bare steel deck diaphragms. In general, non-structural demands play an equal or greater role than structural concerns in the typical final design of diaphragms.

C5. Considering the most prominent available floor diaphragm system: *steel deck diaphragms with headed shear studs and concrete fill*, what <u>challenges</u> do you face in making this specific system meet your design constraints?

Steel deck diaphragms with headed shear studs and concrete fill are common, but even still engineers are unclear on how to combine load cases on shear studs - and in particular how gravity and diaphragm shear demands should both be accounted for (or not). It is not clear what the best solution is for chord and collector design of this system.

C6. Again considering *steel deck diaphragms with headed shear studs and concrete fill*, do you include supplemental reinforcement/rebar (beyond temperature and shrinkage steel) in the fill to meet diaphragm demands or serve as chords/collectors? Please explain why/why not.

Engineers today are completely split on the use of supplemental reinforcement in the deck fill. One group of engineers finds supplemental reinforcement to be the most efficient solution particularly for chords and collectors and relies on this as a standard design. A second group is not satisfied that predicted strength will be present in thin slabs on deck and instead uses discrete steel members for C&C design. This is a major point of disagreement that deserves resolution. (Note SDII archetype designs considered both approaches as both methods are in the AISC seismic design manual.)

C7. Again for *steel deck diaphragms with studs and fill*, what is your typical slab edge detail?

Slab edges are not simply bare. At least a CFS or bent angle pour stop exists. Slab edges depend on the cladding system - if the cladding load is connected to the slab then embeds or supplemental reinforcement will exist on slab edge. This edge condition may influence SDII testing, both cantilever and pushout testing w.r.t edge condition.

C8. Considering a roof diaphragm system utilizing *bare steel deck diaphragms*, what challenges do you face in making this specific system meet your design constraints?

Bare steel deck roofs today may use proprietary deck, proprietary fasteners, and delegated design. As a result some EORs may feel that the system is hard to understand, and too complex to design. Though a commonly used system, engineers can find it challenging to meet basic strength requirements, detail the chords and collectors, and detail openings and other irregularities. Vibrations related to MEP are an ongoing concern.

C9. Considering *chords and collectors* specifically, what challenges do you face in the design and detailing of these members? (clarify if you are addressing floor or roof diaphragms specifically)

Engineers have a large number of questions about the details of making chords and collectors work successfully. Transferring load from the diaphragm/slab to the chords and collectors is an issue with a great deal of unknowns for current engineering. There is also question as to the impact of the C&C details on the performance of the vertical LFRS - and the creation of unintentional moment frames. In the main, it is not felt that engineers typically have much training nor particularly strong tools or solutions to tackle these issues.

C10. Please make any *additional comments* you would like with respect to challenges as related to diaphragms for steel buildings (e.g. codes and standards disconnects, modular buildings, large openings, floor plate shape, transfers, stiffness-mass eccentricities, etc.):

From a seismic perspective it is not yet clear what the consequence of diaphragm "failure" is for the structure - more specifically, what is the consequence in steel deck with concrete filled diaphragms? Design philosophies for irregularities are sorely lacking, and needed.

Feedback on Innovation

N1. Thinking broadly, what innovations would you suggest to improve the design, detailing, construction, or behavior of diaphragms in steel buildings under seismic demands?

Innovation in diaphragms is hampered by a definitive lack of knowledge with respect to the behavior of building systems with inelasticity in both the vertical and horizontal LFRS. If this behavior is understood then software improvements (that support design) and specific technological innovation (isolation, improved damping, optimized deck profiles) can have impact.

N2. What is your reaction to the idea of having targeted seismic energy dissipation systems (e.g. replaceable *shear fuses*) in floors/roofs instead of, or in addition to, the vertical LFRS?

The idea of fuses in diaphragms brings a number of concerns to the forefront: cost, incompatibility between the vertical and lateral systems, and fire separation concerns chief among them. Nonetheless for unique, high end, structures the excellent potential for such a solution may be worth exploring.

N3. Based on your understanding of current seismic steel building design (ASCE 7-16, AISC 341-16) do you expect inelastic demands in your building diaphragms at DBE level? MCE level?

DE level	MCE level
no -12	no -4
some -5	some -6
yes -3	yes -10

Expected seismic behavior of steel diaphragms lacks clear objectives and engineers are not operating under a consistent set of assumptions. Some engineers believe the code explicitly, others not at all.

N4. Commonly, diaphragms are treated separately from the vertical LFRS. What is your reaction to design of buildings as 3D structures with seismically designed and detailed components in both the vertical and horizontal planes? What challenges do you see in this approach?

The notion of enabling fully 3D building analysis has support in the engineering community but the notion of requiring such analysis for typical design does not. Engineers understand that the vLFRS and hLFRS may interact, but except in special cases there is not enough time in the design process to consider this complication. The code should provide 3D building analysis as an option and then should provide safe methods that separate the vLFRS and hLFRS for the vast majority of buildings.

N5. Today, code-based design (ELF, RSA, RHA) considers only the vertical system in establishing R, C_d, Ω_o for buildings. What benefit (greater accuracy, greater flexibility in building configuration, reduced demands, consideration of diaphragm effects, etc.) would potentially be great enough to shift design to considering the combined vertical and horizontal systems?

The only compelling reason for the engineer to complicate their design and consider both the hLFRS and the vLFRS directly in design as a combined system is if in doing so there are substantial cost savings to the building. Other benefits: design flexibility, reliability, repairability, accuracy, performance are also recognized, but cost is paramount.

N6. If seismic diaphragm demands could be directly predicted from a building model, would that be attractive? If the following were required by codes, how would each affect your decision to use a more *analysis/model-based design*: 3D models, semi-rigid diaphragm modeling, response-history analysis, nonlinear analysis?

The notion that diaphragm demands could come directly from a building model was universally supported by mid/high-rise engineers that presumably already have such models; only about 1/2 of engineers specializing in low-rise and/or industrial structures supported this design paradigm. In general there is concern that creating the model be a billable effort with a useful result. The more complex the model (3D) or analysis (nonlinear time history) the less the interest from the engineers.

N7. In considering innovations to support new technologies – how important do you think the principles of modular construction will be in the future? From your perspective, what innovations are needed to make modular systems have an effective diaphragm?

Engineers do perceive further increases in modular construction, but believe the connections at the modules (for diaphragm as well as gravity and other loads) need deeper thinking. For the diaphragm, continuity and stability bracing for the columns are specific concerns for modular construction.

N8. In considering innovations to support future performance of buildings – how important do you think the principles of "design for deconstruction" will be in the future? What opportunities for innovation do you perceive in floor and roof systems that are designed for deconstruction?

The engineers noted that DfD is a benefit that is not aligned with the decision-makers and only if regulations change do they perceive a large change in this arena. That said a focus on design life in the big picture, and connections as a primary detail of concern are most important for DfD.

N9. Please make any additional comments you would like with respect to innovation as related to diaphragms for steel buildings (incorporating high strength steel rebar; or high performance steel for members, deck, studs, etc.; dry floor systems with concrete board; 2-way steel systems, etc.):

Engineers were challenged to point out or advocate for specific innovations, but did note that an emphasis on constructability can lead to innovative ideas and provided the example of steel deck with concrete panels as a system worthy of further study.

General Feedback

The questions are great, but what you really need to know is:

Overall the engineers emphasized that any changes must be relatively simple, or they cannot be used. Many of the challenges mentioned in earlier questions were echoed in the summary here. However a few new thought emerged: the role of diaphragms in bracing the gravity columns, not just in providing in-plane shear needs to be considered; how to handle wind vs. seismic in diaphragm design seems unclear; post EQ inspect-ability should be considered with these systems. Keep it as simple as you can.

The detailed lists in Appendix 3 provide even more depth to these summary answers. The summary response to question N3 bears repeating:

> "Expected seismic behavior of steel diaphragms lacks clear objectives and engineers are not operating under a consistent set of assumptions. Some engineers believe the code explicitly, others not at all."

What this questionnaire unequivocally demonstrates is that fundamental issues remain for this system – while this creates a challenge, it also provides an opportunity.

8 Conclusions

Steel decks, both bare and concrete-filled, are commonly used as roof and/or floor diaphragms in buildings subjected to seismic demands. Today neither engineers, nor the prevailing standards, provide a clear set of objectives for the seismic performance of these steel deck diaphragms. SDII research is investigating the conditions required for steel deck diaphragms to have ductility and deformation capacity and examining the impact of diaphragm performance on whole building seismic performance. The research encompasses experiments, modeling, innovation, and practice/codes and standards. The Steel Diaphragm Innovation Initiative (SDII) effort is envisioned as a 5 year project and at the time of the workshop described herein year 3 was just completed. A full report on progress was provided and may be found in Appendix 1 and 2.

Workshop participants provided detailed feedback on challenges and potential innovation for both bare and concrete-filled steel deck diaphragms. Through a detailed questionnaire and facilitated small group participation at the workshop a number of key issues were identified. Major challenges were detailed with respect to: stiffness, strength, demands, (plan) irregularities, design workflow, training, and best practices. As diaphragm demands become large, whether due to irregularities, transfers, or fundamental layout choices (span, aspect ratio etc.) engineers are faced with increasing challenges and opinions diverge on best practices: with one group advocating to use more reinforcement in the concrete fill above the steel deck – including to handle chord and collector demands; versus another group with concerns that the thin slabs above deck have sufficient confinement to achieve currently calculated capacities, who instead advocate for using large discrete steel components as chords and collectors. A number of other detailed challenges are provided in this report and are fully detailed in Appendix 3.

Discussion surrounding the potential for innovation in steel deck diaphragms was hampered in part by a series of constraints: (1) fundamental: inelastic interaction of the vertical (walls) and horizontal (floors/roof) lateral force resisting system is not fully understood, (2) financial: engineers do not have additional design time to spend on more sophisticated diaphragm or building analysis. Thus, the participants understood that the actual seismic building performance is complex, and if the diaphragm is fully considered, decidedly more complex that current design, but at the same time, for all but the most rare of buildings, the design methods must remain simple and straightforward given current fees. These tensions speak to the need for innovation in the design methods, workflow, and training related to steel deck diaphragms. Specific technological innovations for steel deck diaphragms: structural fuses, changes for modular construction, etc. were also explored and the participants provided advantages and challenges for a number of possible innovations.

SDII has significant efforts underway to improve understanding of steel deck diaphragm performance, improve understanding of the role of diaphragms in the seismic performance of buildings and improve the design solutions and methods available to engineers employing bare and concrete-filled steel deck diaphragms. While the existing work plan is comprehensive, the workshop highlighted key challenges and opportunities for the effort going forward.

9 Acknowledgments and Disclaimer

The SDII research team would like to acknowledge all of the volunteer participants who gave a full day of their time to participate in the workshop. The assembled group was a who's who of steel building engineers and the SDII team is honored that they donated their time to this effort. In addition to the overall acknowledgments for the SDII effort as provided on page 2 of this report the author would like to specifically acknowledge Tom Sputo (SDI), Bonnie Manley (AISI), and Pat Bodwell (Verco) for their assistance in bringing the workshop together. Bonnie Manley (AISI) and JP Cardin (AISI) provided notes that were used, in part, to complete the workshop summary provided in this report. Further, SDI and AISI provided funding above their SDII contributions to make the workshop a reality. All of this support is deeply appreciated. Any opinions, findings, and conclusions or recommendations expressed in this publication are those of the author(s) and do not necessarily reflect the views of the sponsors.

Appendix 1: SDII Research Summary Slides

Steel Diaphragm Innovation Initiative
Workshop

Welcome and Introductions
Tom Sputo

10 January 2019

COLD-FORMED STEEL RESEARCH CONSORTIUM

SDII Industry Sponsors

- American Iron and Steel Institute
- American Institute of Steel Construction
- Steel Deck Institute
- Steel Joist Institute
- Metal Building Manufacturers Association

CFSRC

Schedule for the Morning

8:00 – 8:05	Introduction (Sputo)
8:05 – 10:00	Overview of SDII (Schafer)
	Compiling and analyzing existing data (Eatherton)
	New cyclic testing to characterize performance across scales
	Connector (fastener shear) (Schafer)
	Interface (pushout) (Hajjar)
	Diaphragm (cantilever) (Easterling)
	Planned large scale testing (Hajjar)
	Leveraging Simulation
	Vertical vs. horizontal LFRS (Schafer)
	Building scale archetype simulations (Eatherton)
	Bringing fracture into models (Hajjar)
	Optimization (Schafer)
	Conclusions (Schafer)
10:00 – 10:30	**Break**
10:30 – 11:30	SDII Codes and Standards - Proposals and Future Pathways
11:30 – 12:00	Questionnaire

SDII

CFSRC

Steel Diaphragm Innovation Initiative

Workshop

S. Easterling, M. Eatherton, J. Hajjar, R. Sabelli, B. Schafer

10 January 2019

COLD-FORMED STEEL RESEARCH CONSORTIUM

“Diaphragm” Failure

(1994 Northridge Earthquake, CSU Parking Garage Collapse)


USGS


USGS

The 1994 Northridge earthquake was a huge event in the steel industry putting in motion a large effort on moment connections, with significant federal funding, and resulting in new innovations, new code procedures, and improved buildings. The same event also triggered significant work in concrete, a sizeable portion of which was on diaphragms.

Research Lead to New Code Methods


Restrepo et al. UCSD

STRUCTURAL PERFORMANCE

performance issues relative to extreme events

Alternative Diaphragm Seismic Design Force Level of ASCE 7-16

By S. K. Ghosh, Ph.D.

S. K. Ghosh is President at S. K. Ghosh Associates Inc., Palatine, IL and Aliso Viejo, CA. He chaired Issue Team 6 on Diaphragms of the Building Seismic Safety Council Provisions Update Committee for the 2015 NEHRP Provisions. He can be reached at ***skghoshinc@gmail.com.***

The next edition of ASCE 7 *Minimum Design Loads for Buildings and Other Structures*, ASCE 7-16 (ASCE, 2016), is expected to be published in September 2016, in time for adoption into the 2018 *International Building Code* (IBC) (ICC, 2018). For that edition, ASCE 7-10 (ASCE, 2010) has been modified to include a new Section 12.10.3, Alternative Design Provisions for Diaphragms including Chords and Collectors, within Section 12.10, Diaphragm Chords and Collectors. The new section provides for an alternative determination of diaphragm design force level, which is mandatory for precast concrete diaphragms in buildings assigned to SDC C, D, E, or F. The alternative is permitted to be used for other precast concrete diaphragms, cast-in-place concrete diaphragms, and wood sheathed diaphragms on wood framing. Section 12.10.3 does not apply to steel deck diaphragms. ASCE 7-10 has also been modified to add a Section 14.2.4, containing detailed seismic design provisions for precast concrete diaphragms including a connector qualification protocol. Chapter 14 of ASCE 7-10 is currently not adopted by the 2012 or 2015 IBC (ICC, 2012, 2015). Steps directed towards the inclusion of ASCE 7-16 Section 14.2.4 in the 2018 IBC are now being taken.

Both changes originated in the 2015 *NEHRP Recommended Provisions for Buildings and Other Structures* (FEMA, 2015). This article is devoted to a discussion of ASCE 7-16 Section 12.10.3, Alternative Design Provisions for Diaphragms including Chords and Collectors.

Basis and Overview

The seismic design of structures has long been based on an approximation of the inelastic response of the seismic force-resisting system. The approximation reduces the results of an elastic analysis in consideration of the reserve strength, ductility, and energy dissipation inherent in the vertical elements of the seismic force-resisting system. In 1978, ATC-3 (ATC, 1978) provided design force reduction factors based on consideration of inelastic behavior of the vertical elements of the seismic force-resisting system and the performance of structures in past earthquakes. The primary assumption leading to these factors is that yielding in the vertical elements of the seismic force-resisting system is the primary mechanism for inelastic behavior and energy dissipation. Starting with the 1997 *Uniform Building Code* (UBC) (ICBO, 1997), the actual forces and displacements that might occur in the vertical elements in a design-level seismic event were recognized with the introduction of a seismic force amplification factors, Ω_0, and deflection amplification factor, C_d, respectively. In contrast, the design requirements for the horizontal elements of the lateral force-resisting system (the diaphragms) have been established by empirical considerations related to anticipated behavior of the vertical elements, rather than explicitly considering behavior of the diaphragms. For established diaphragm construction types, this empirical approach has been generally satisfactory. Satisfactory system performance, however, requires that the diaphragms have sufficient strength and ductility to mobilize the inelastic behavior of the vertical elements.

In order to help achieve the intended seismic performance of structures, the designs of horizontal and vertical elements of the seismic force-resisting system need to be made more consistent. Analytical results, as well as experimental results from shake-table tests in Japan, Mexico, and the United States, have shown that diaphragm forces over much of the height of the structure actually experienced in the design-level earthquake may at times be significantly greater than code-level diaphragm design forces, particularly where diaphragm response is near-elastic. Overstrength and ductility of the diaphragm, however, may account for satisfactory diaphragm performance. ASCE 7-16 Section 12.10.3 ties the design of diaphragms to levels of force and deformation capacity that represent actual anticipated behavior.

ASCE 7-16 Section 12.10.3 presents a near-elastic diaphragm force as the statistical sum of first mode effect and higher mode effects (Rodriguez et al., 2002). The first mode effect is reduced by the *R*-factor of the seismic force-resisting system, but then amplified by the overstrength factor, Ω_0, because vertical element overstrength will generate higher first mode forces in the diaphragm. The effect caused by higher mode response is not reduced. In recognition of the deformation capacity and overstrength of the diaphragm, the elastic diaphragm force from the first and higher modes of response is then reduced by a diaphragm force reduction factor, R_s.

With the modification by R_s, the proposed design force level may not be significantly different from the diaphragm design force level of ASCE 7-16 Sections 12.10.1 and 12.10.2 for many practical cases. For some types of diaphragms and for some locations within structures, the proposed diaphragm design forces will change significantly, resulting in noticeable changes to resulting construction. Based on data from testing and analysis, and on building performance observations, it is believed that these changes are warranted.

The alternative design force level of Section 12.10.3 is based on work by Rodriguez, Restrepo, and Carr (Rodriguez et al., 2002), verified by more recent work by Fleischman et al. (Pankow, 2014).

18 March 2016

Research Lead to New Code Methods

- R_s reduces floor / diaphragm demand

ASCE 7-16

Diaphragm System	Shear-Controlled	Flexure-Controlled
Cast-in-place concrete	1.5	2.0
Precast concrete	0.7-1.4	0.7-1.4
Wood sheathed	3.0	NA

No provisions for steel diaphragm systems are included in the method.

STRUCTURAL PERFORMANCE

performance issues relative to extreme events

Alternative Diaphragm Seismic Design Force Level of ASCE 7-16

By S. K. Ghosh, Ph.D.

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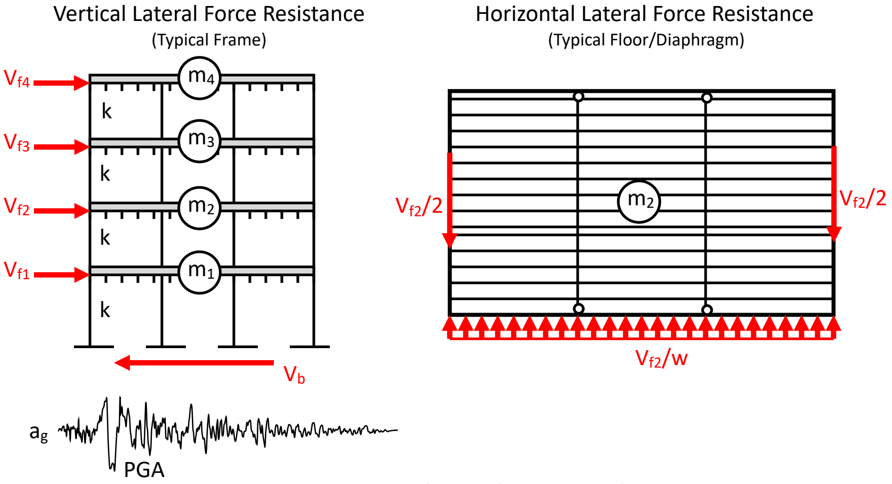
18 March 2016

Traditional Diaphragm EQ Eng.



$$V_b = M \times S_a(T, PGA)/R$$

Almost always controls

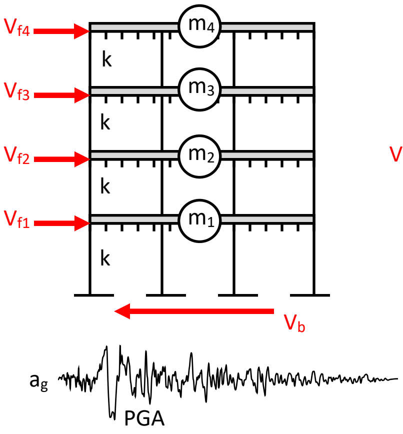
$$\tfrac{1}{2} m_2 \times PGA < V_{f2} < m_2 \times PGA$$

$$0.2 m_2 S_{DS} < V_{f2} < 0.4 m_2 S_{DS}$$

New/Alternative Diaphragm EQ Eng.

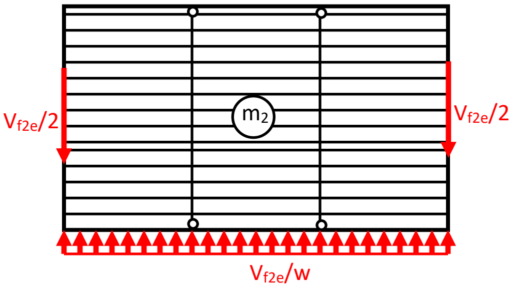
Vertical Lateral Force Resistance

(Typical Frame)



Horizontal Lateral Force Resistance

(Typical Floor/Diaphragm)



$V_b = M \times S_a(T,PGA)/R$

$\frac{1}{2}m_2 \times PGA < V_{f2e} = \text{“}m_2 \times PGA\text{”}/R_s$

$R_s > 2$ or floor forces likely go up (can more than double)

Steel Diaphragm Innovation Initiative (SDII)

- *Origin*: SDII was born, in part, out of the limitations in knowledge that came to light in developing alternative diaphragm design provisions (R_s) for steel deck diaphragms in the last seismic code cycle
- *Objective*: Advance the seismic performance of steel floor and roof diaphragms utilized in steel buildings through:
  - better understanding of diaphragm-structure interaction,
  - new design approaches, and
  - new three-dimensional modeling tools that provided enhanced capabilities to designers utilizing steel diaphragms in their building systems.
- *Scope*: SDII primarily focuses on the seismic design of diaphragms commonly used in steel mid-rise buildings, but considers innovation for all systems employing steel floor and roof diaphragms.

SDII Team and Partners:

- Management:
  COLD-FORMED STEEL RESEARCH CONSORTIUM
- Industry Sponsors:
  American Iron and Steel Institute
- Government Sponsors:
- Researchers:
  JOHNS HOPKINS WHITING SCHOOL of ENGINEERING
  Virginia Tech
  Northeastern
  WALTER P MOORE

SDII Case and Research Plan

- Case Statement
- Research Overview
- Detailed Tasks
  - Innovation and Practice
  - Experiments
  - Modeling
- Funding Plan
- Glossary of terms

Appendix C: Steel Diaphragm Innovation Initiative Case and Plan

SDII
(Steel Diaphragm Innovation Initiative)

Case and Research Plan
version 3.0
(SDII year 2 update)
15 June 2016

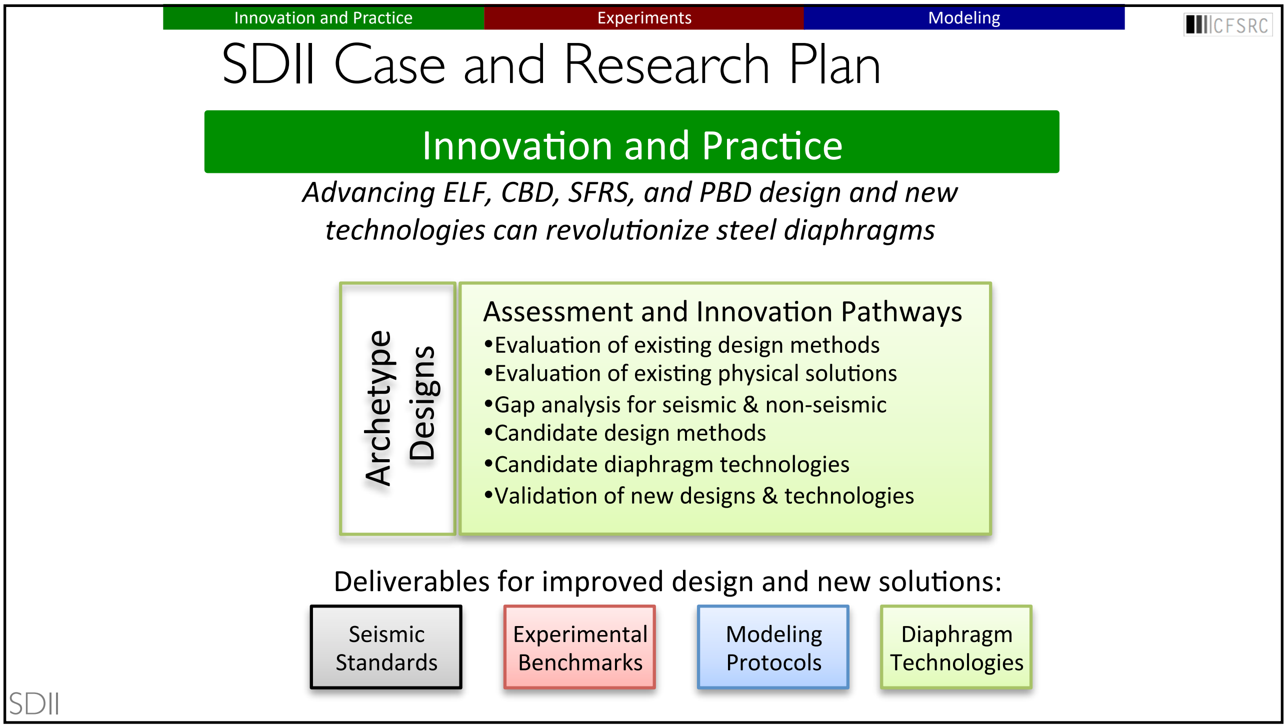
Innovation and Practice

Experiments

Modeling

COLD-FORMED STEEL RESEARCH CONSORTIUM

*Currently (January 2019) Just began **year 4** of the 5 year plan*


Innovation and Practice | Experiments | Modeling

CFSRC

SDII Case and Research Plan

Innovation and Practice

Advancing ELF, CBD, SFRS, and PBD design and new technologies can revolutionize steel diaphragms

Archetype Designs

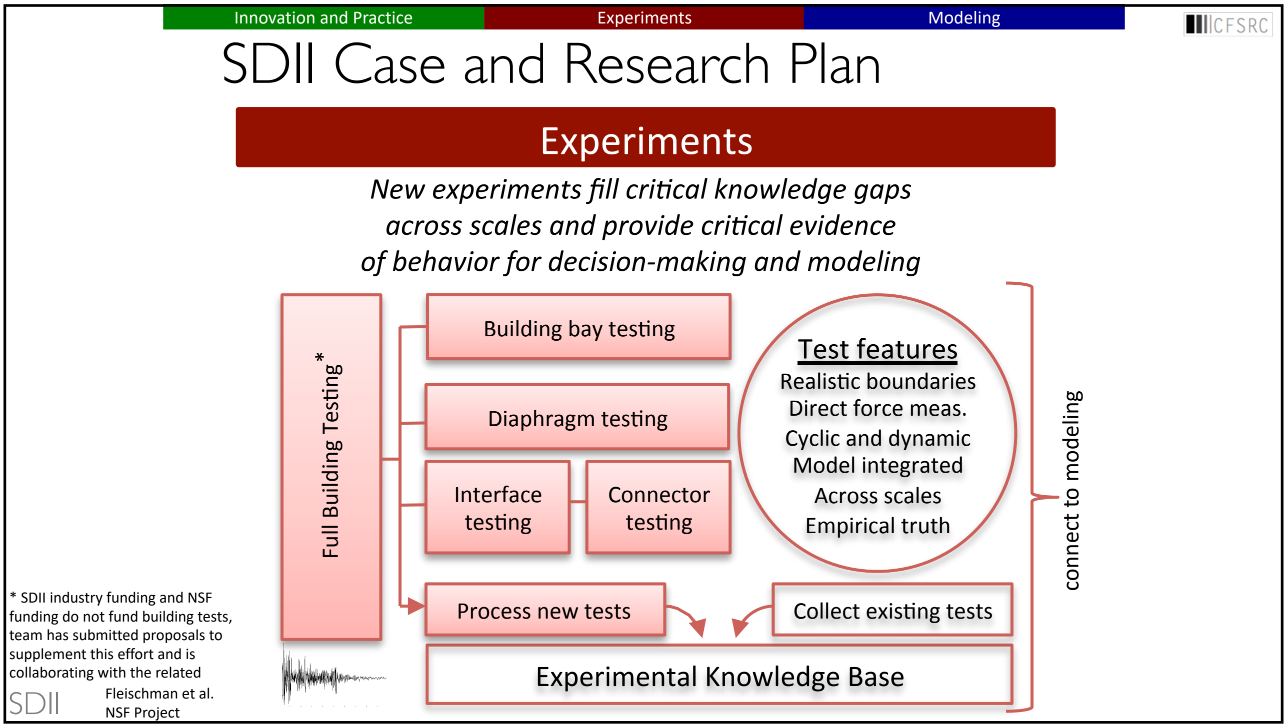
Assessment and Innovation Pathways

- Evaluation of existing design methods
- Evaluation of existing physical solutions
- Gap analysis for seismic & non-seismic
- Candidate design methods
- Candidate diaphragm technologies
- Validation of new designs & technologies

Deliverables for improved design and new solutions:

Seismic Standards | Experimental Benchmarks | Modeling Protocols | Diaphragm Technologies

SDII


Innovation and Practice | Experiments | Modeling

CFSRC

SDII Case and Research Plan

Experiments

New experiments fill critical knowledge gaps across scales and provide critical evidence of behavior for decision-making and modeling

Full Building Testing*

Building bay testing

Diaphragm testing

Interface testing

Connector testing

Process new tests

Collect existing tests

Experimental Knowledge Base

Test features

Realistic boundaries
Direct force meas.
Cyclic and dynamic
Model integrated
Across scales
Empirical truth

connect to modeling

* SDII industry funding and NSF funding do not fund building tests, team has submitted proposals to supplement this effort and is collaborating with the related Fleischman et al. NSF Project

SDII

Innovation and Practice | Experiments | Modeling

CFSRC

SDII Case and Research Plan

Modeling

Novel models across scale and fidelity provide the platform for exploring behavior, developing enabling tools, and establishing reliable designs

connect to experiments

Experimental Integration
- Develop Tests
- Support Tests
- Hybrid Testing
- Validation
- Parametric Studies
- Virtual Tests

Evolve Todays Design Models
- 3D Fundamentals
- Diaphragm Forces, Response, Detailing, Sizing
- 3D ELF++

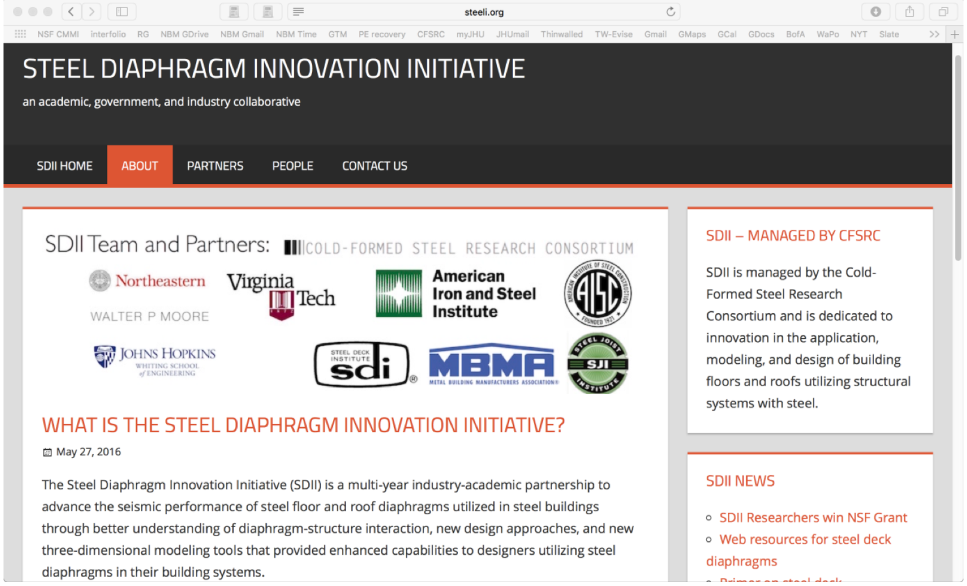
Whole Building Models
- Efficiency
- Uncertainty
- Optimality
- Nonstructural inc.
- Diaphragm role

Next generation performance prediction models
- 3D
- High Fidelity
- Explicit Fracture
- Accurate Collapse

SDII

CFSRC

Keep up to date on the whole effort: steeli.org



STEEL DIAPHRAGM INNOVATION INITIATIVE

an academic, government, and industry collaborative

SDII Team and Partners:

WHAT IS THE STEEL DIAPHRAGM INNOVATION INITIATIVE?

May 27, 2016

The Steel Diaphragm Innovation Initiative (SDII) is a multi-year industry-academic partnership to advance the seismic performance of steel floor and roof diaphragms utilized in steel buildings through better understanding of diaphragm-structure interaction, new design approaches, and new three-dimensional modeling tools that provided enhanced capabilities to designers utilizing steel diaphragms in their building systems.

SDII – MANAGED BY CFSRC

SDII is managed by the Cold-Formed Steel Research Consortium and is dedicated to innovation in the application, modeling, and design of building floors and roofs utilizing structural systems with steel.

SDII NEWS

- SDII Researchers win NSF Grant
- Web resources for steel deck diaphragms

SDII

Compiling and analyzing existing data: diaphragm database

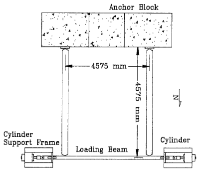
Analyzing ductility and diaphragm seismic factor, Rs

Cantilever Diaphragm Test Database

Overview

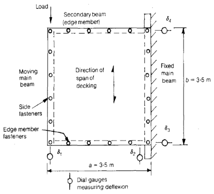
Testing Program	# of Specimens
Cornell University, 1950s-1960s	40
S. B. Barnes and Associates, 1950s -1960s	38
West Virginia University, 1960s-70s	246
Development Lab of Inland Ryserson Co.	1
University of Salford, Manchester 1970s-80s	5
ABK, a Joint Venture, California 1980s	3
Iowa State University, 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

Types of Experimental Studies Included

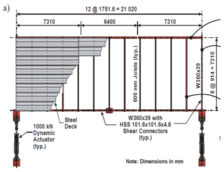


Group from Iowa State in 1980's and 1990's

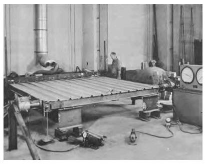
Diaphragm Tests by Industry (e.g. Hilti)



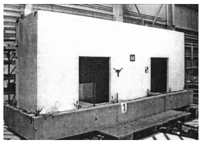
Research from Europe (e.g. Davies and Fisher 1979)



Work by Tremblay and Rogers in Canada



Larry Luttrell's group at West Virginia



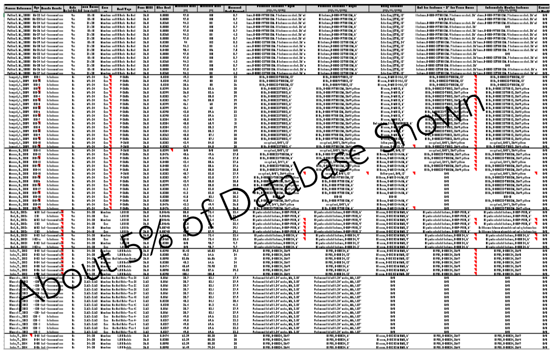
Building Tests (e.g. Cohen et al. 2004)

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 (medium and long period), R_s
	Predicted diaphragm stiffness	

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.



About 5% of Database Shown

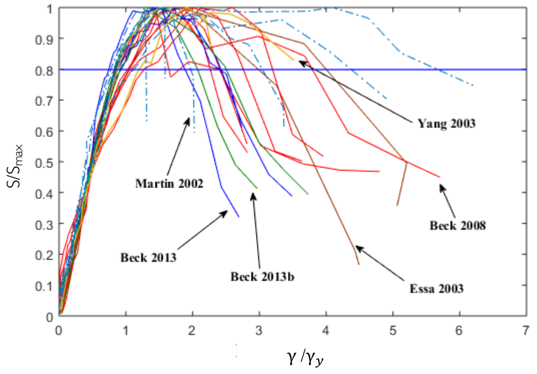
Data provides subassembly ductility

Grouping (Monotonic and Cyclic Groups for Each):

1. Bare Deck – PAF support, Screw sidelap
2. Bare Deck – Weld support, Button Punch sidelap
3. Bare Deck – Weld support, Screw sidelap
4. Bare Deck – Weld support, welded sidelap

---

5. Concrete filled – arc spot welds to supports (Cyclic only)
6. Concrete filled – welds and studs (Cyclic only)



Group with PAF Support Fasteners and Screwed Sidelap Fasteners (Cyclic Testing)

Ductility defined as ratio of displacement at 80% strength degradation to yield displacement

$$\gamma_{ult} = \gamma \ at \ 0.8\, P_{max}$$

$$\gamma_y = \frac{S_{max}}{G'}$$

$$\mu_{sub} = \frac{\gamma_{ult}}{\gamma_y}$$

PAF/Screw (21 Specimens)	
Average μ_{sub}	Std. Dev.
2.76	1.02

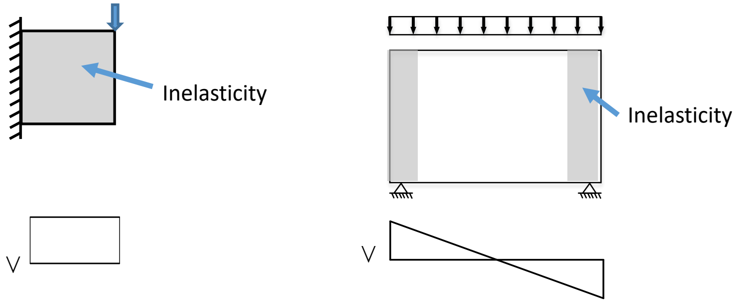
Data provides insight on ASCE 41 “m” & R_s

Fastener Configuration	Total Specimens	G'$_{avg}$ (kip/in)	S$_{max\_avg}$ (kips/ft)	μ_{sub} Avg.	μ_{sub} Std.Dev.	
PAF/Screw	22	47.9	2.03	4.53	3.62	Monotonic No Conc. Fill
Weld/BP	8	20.3	1.27	2.58	0.36	
Weld/Screw	11	49.2	2.05	3.29	1.20	
Weld/Weld	14	68.5	3.00	3.34	1.17	
PAF/Screw	21	45.3	2.52	2.76	1.02	Cyclic No Conc. Fill
Weld/BP	6	12.3	0.66	1.53	0.39	
Weld/Screw	2	17.2	1.09	1.93	0.07	
Weld/Weld	4	21.2	1.55	2.06	0.44	
Welds	14	1490	10.3	5.53	3.08	Cyclic Conc. Fill
Welds and Studs	6	1670	8.09	3.82	0.62	

Diaphragm system ductility – Not equal to cantilever test (subassembly) ductility

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

Cantilevered diaphragm test　　Simply supported diaphragm



Shear distribution: Uniform shear　　Shear distribution: linear variation

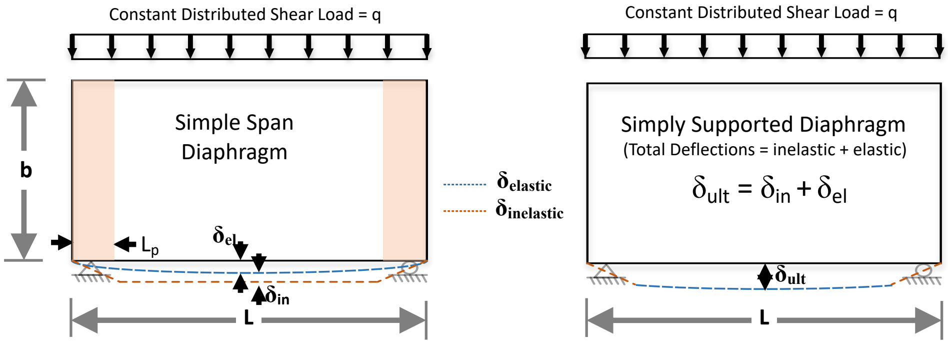
Conclusion: $\mu_{subassembly} > \mu_{system}$

Resolution: estimate elastic and inelastic δ

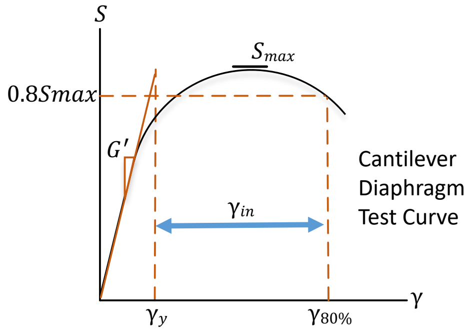
Deflections and ductility will differ from subassembly to system → $\mu_{\text{subassembly}} \neq \mu_{\text{system}}$

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

Find δ_{in} and δ_{el}



Resulting Equation for Ductility and R_S



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

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

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

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

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

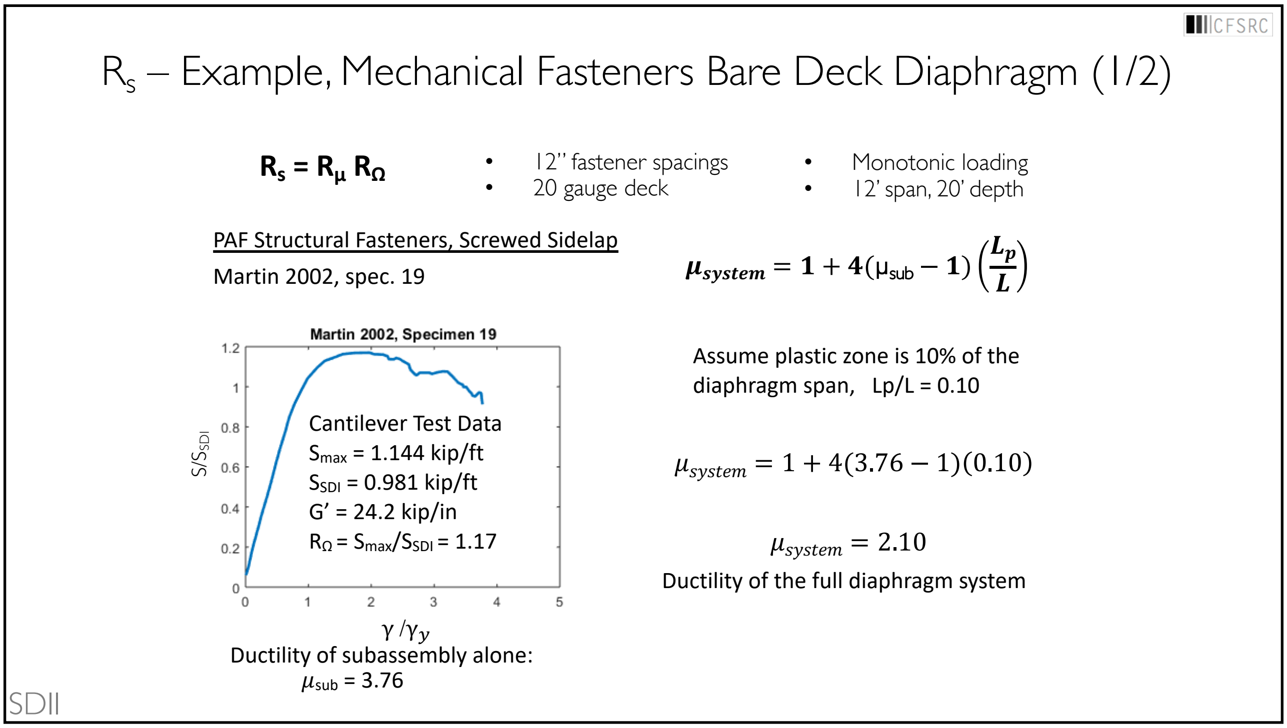
- System ductility depends on L_p/L, not L
- Will need to assume a plastic zone length L_p/L

Method for Estimating Rs (Based on ATC 19)

$R_s = R_\mu R_\Omega$

$R_\mu = \sqrt{2\mu_{system} - 1}$ or μ_{system} (depending on period)

$R_\Omega = S_{max} / S_{SDI}$ (from test)



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

PAF Structural Fasteners, Screwed Sidelap

Martin 2002, spec. 19

Cantilever Test Data

$S_{max} = 1.144$ kip/ft

$S_{SDI} = 0.981$ kip/ft

G' = 24.2 kip/in

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

Ductility of subassembly alone:

$\mu_{sub} = 3.76$

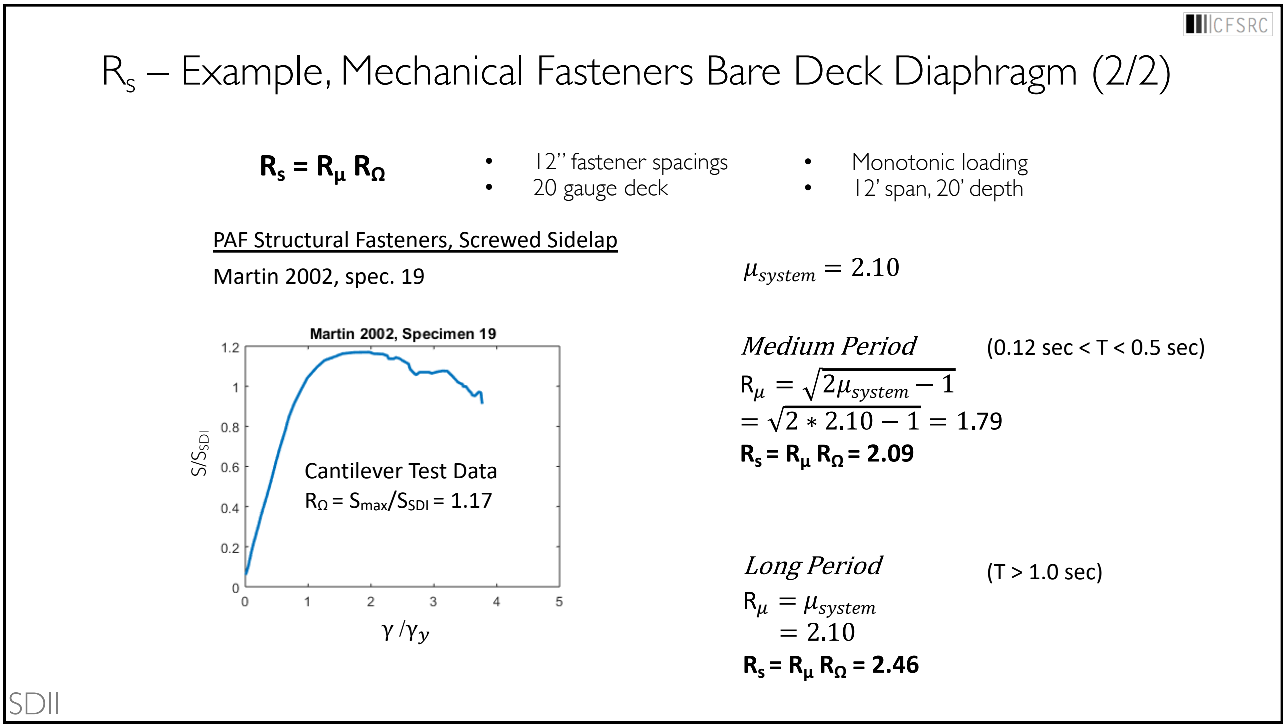
$$\boldsymbol{\mu_{system} = 1 + 4(\mu_{sub} - 1)\left(\frac{L_p}{L}\right)}$$

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

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

$$\mu_{system} = 2.10$$

Ductility of the full diaphragm system



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

$R_s = R_\mu R_\Omega$

- 12" fastener spacings
- 20 gauge deck

- Monotonic loading
- 12' span, 20' depth

PAF Structural Fasteners, Screwed Sidelap

Martin 2002, spec. 19

Cantilever Test Data

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

$$\mu_{system} = 2.10$$

Medium Period (0.12 sec < T < 0.5 sec)

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

$$= \sqrt{2 * 2.10 - 1} = 1.79$$

$R_s = R_\mu R_\Omega = 2.09$

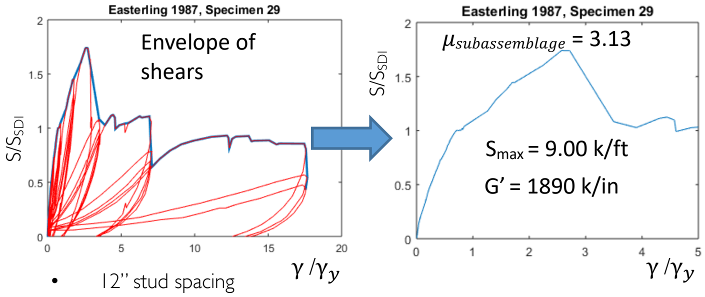
Long Period (T > 1.0 sec)

$$R_\mu = \mu_{system}$$

$$= 2.10$$

$R_s = R_\mu R_\Omega = 2.46$

R_s – Example, Concrete Fill (1/2)



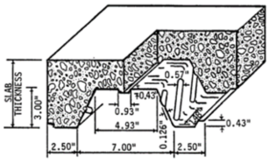
$\mu_{subassemblage} = 3.13$

$S_{max} = 9.00$ k/ft

$G' = 1890$ k/in

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

- 12" stud spacing
- 20 gauge deck
- Cyclic loading
- 12' span, 15' depth
- 5.5" total slab depth
- f'c = 2800 psi



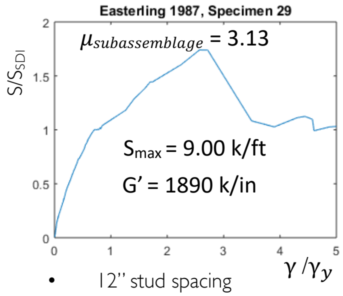
$$\boldsymbol{\mu_{system}} = \mathbf{1} + \mathbf{4}(\mu_{sub} - \mathbf{1})\left(\frac{\boldsymbol{L_p}}{\boldsymbol{L}}\right)$$

$$\mu_{system} = 1 + 4(3.13 - 1)(0.10)$$

$$\mu_{system} = 1.85$$

Ductility of the full diaphragm system

R_s – Example, Concrete Fill (2/2)

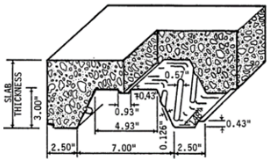


$\mu_{subassemblage} = 3.13$

$S_{max} = 9.00$ k/ft

$G' = 1890$ k/in

- 12" stud spacing
- 20 gauge deck
- Cyclic loading
- 12' span, 15' depth
- 5.5" total slab depth
- f'c = 2800 psi



$R_\Omega = S_{max}/S_{SDI} = 1.74$

$\mu_{system} = 1.85$

Medium Period (0.12 sec < T < 0.5 sec)

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

$= \sqrt{2 * 1.85 - 1} = 1.64$

$R_s = R_\mu R_\Omega = 2.29$

Long Period (T > 1.0 sec)

$R_\mu = \mu_{system}$

$= 1.85$

$R_s = R_\mu R_\Omega = 2.58$

Ongoing work / Summary Thoughts

- Comparing R_s from test data vs. collapse analysis (P695).
- Underlies current proposals for R_s being considered by BSSC and its issue teams.
- Provides evidence for improved strength prediction equations in AISI S310 for composite slab diaphragms.
- Formed the basis for improved provisions for ASCE 41/AISC 342
- Impacts details of current proposals for steel Rigid Wall Flexible Diaphragm buildings.
- Used for nonlinear floor/roof diaphragm models in 3D building models.
- Identified gaps for testing needs

New cyclic testing to characterize behavior across scales

Connector (fastener in shear) – B. Schafer
Interface (push-out tests) – J. Hajjar
Diaphragm (cantilever filled deck) – S. Easterling

New cyclic testing to characterize behavior across scales

Connector (fastener in shear)	**– B. Schafer**
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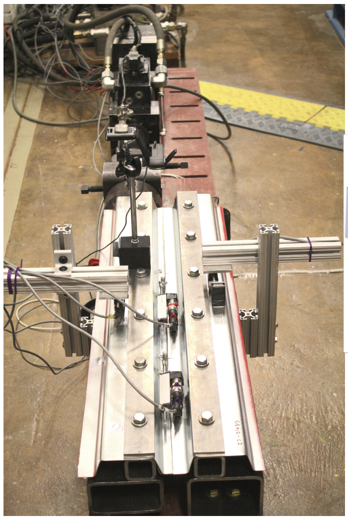
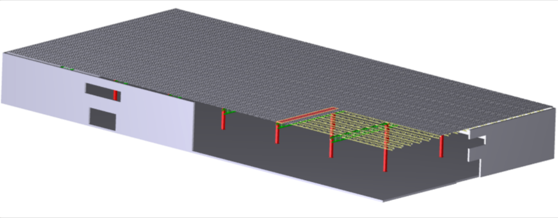
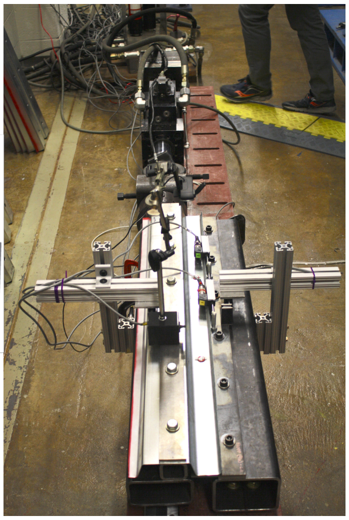
Bare deck fastener testing

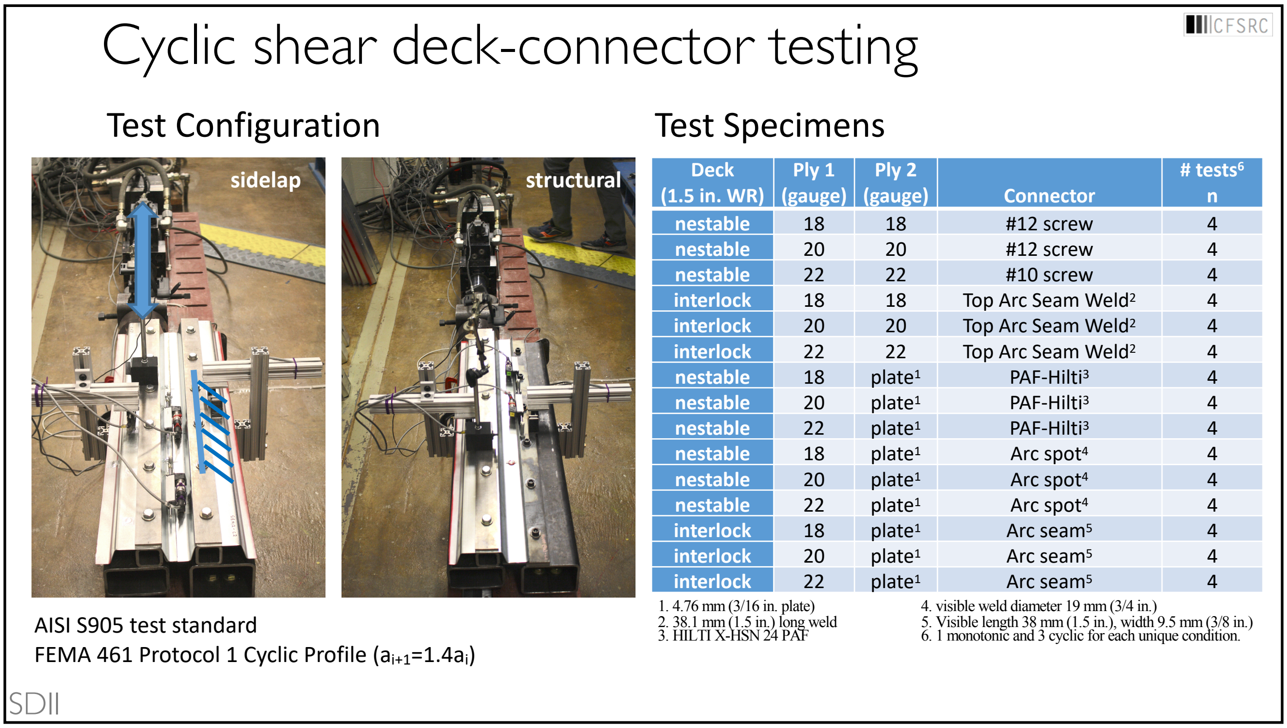


Deck-to-deck
“sidelap” connector



Deck-to-joist/chord
“structural” connector





Cyclic shear deck-connector testing

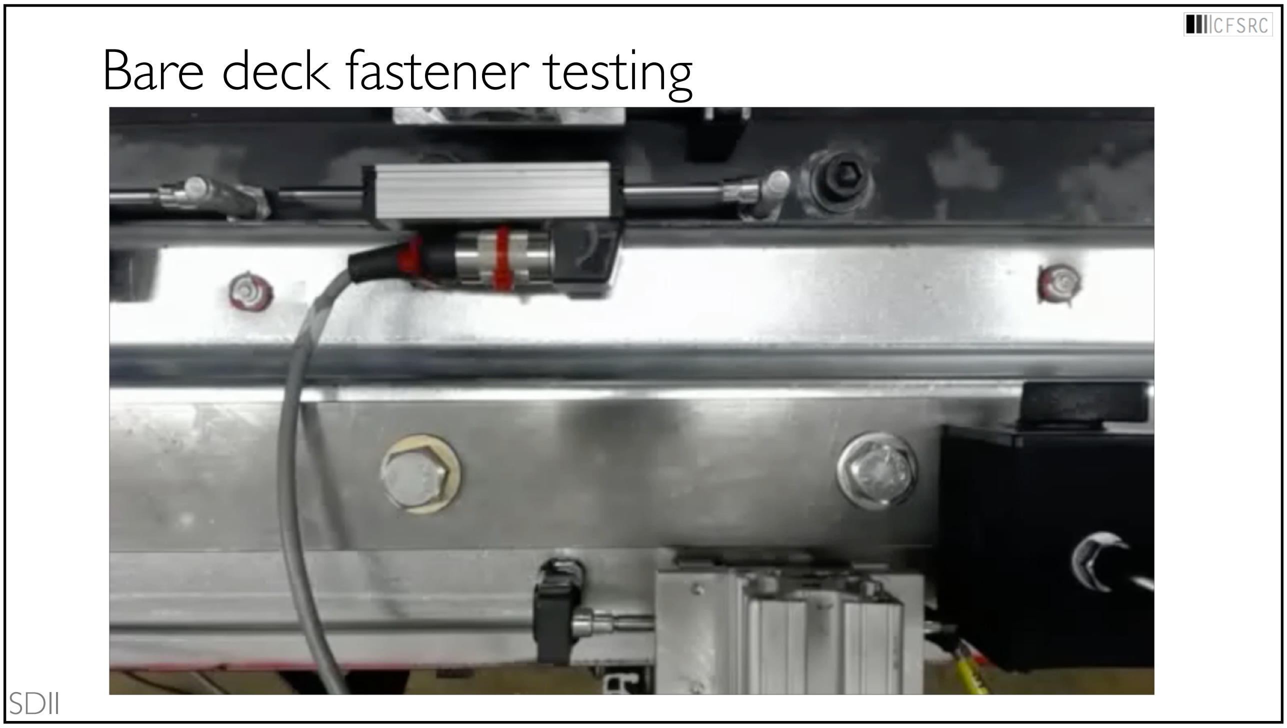
Test Configuration

AISI S905 test standard
FEMA 461 Protocol 1 Cyclic Profile ($a_{i+1}=1.4a_i$)

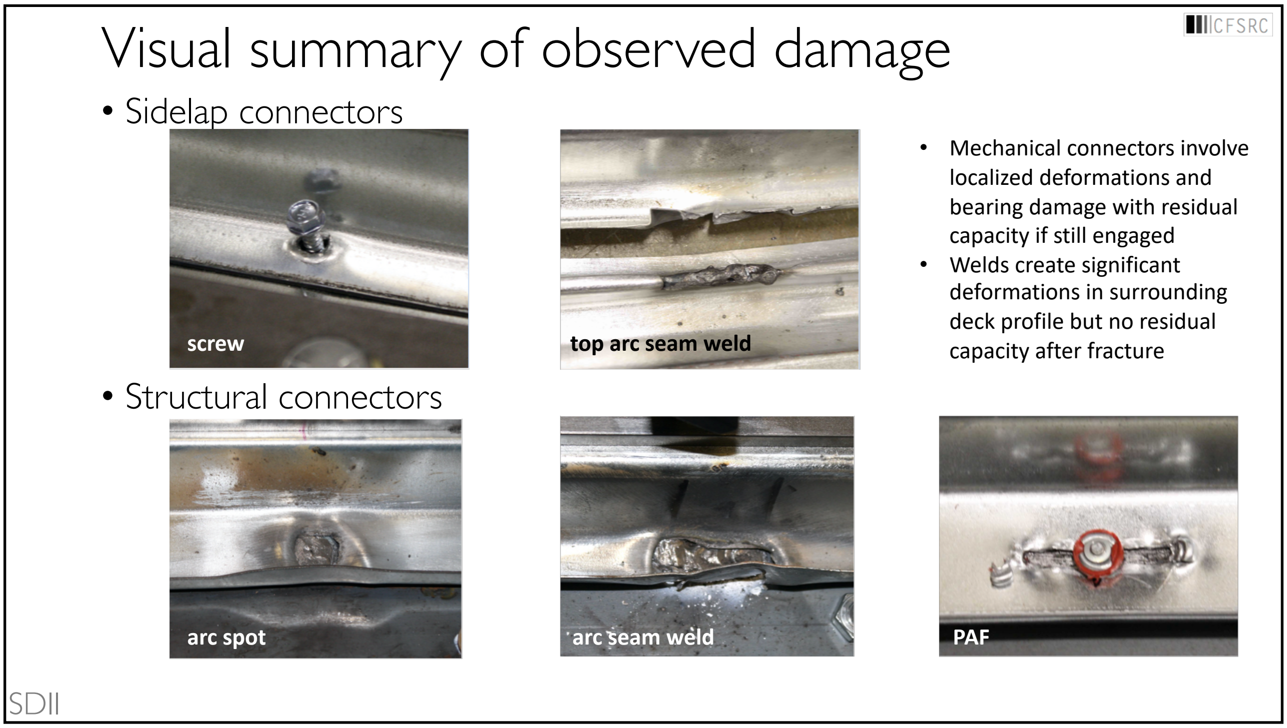
Test Specimens

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.



Bare deck fastener testing

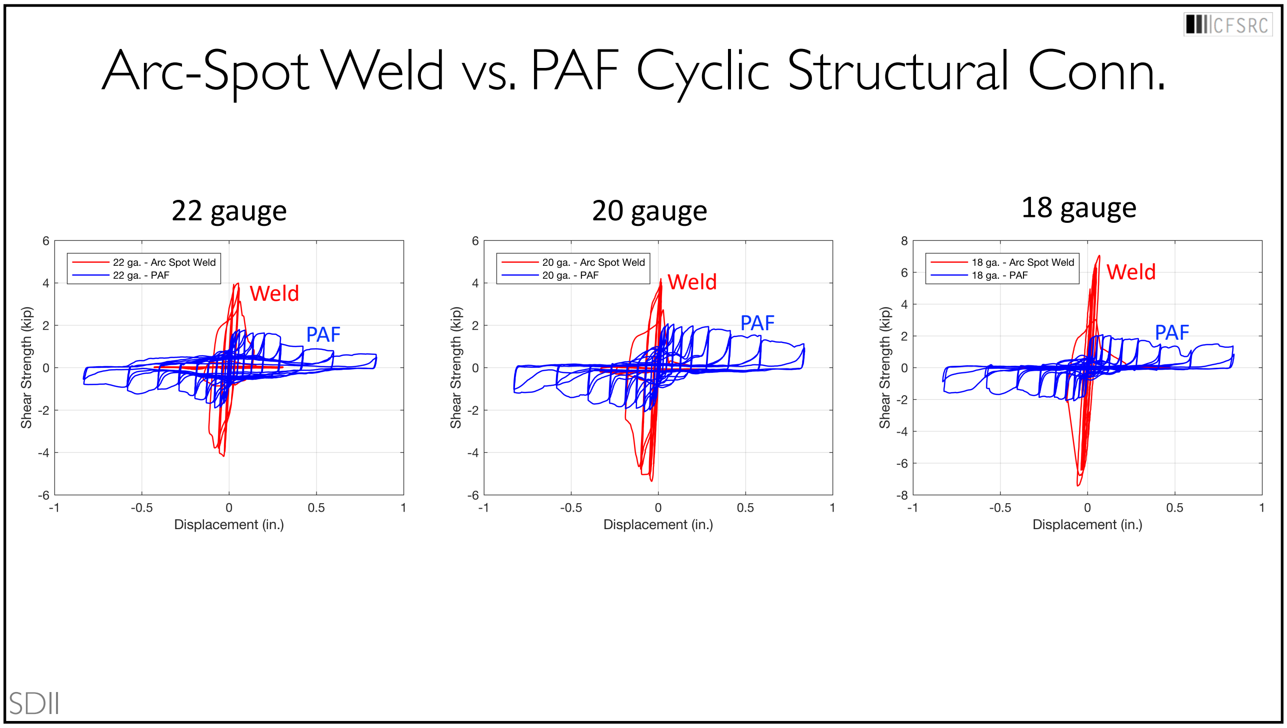


Visual summary of observed damage

- 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



Arc-Spot Weld vs. PAF Cyclic Structural Conn.

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

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New cyclic testing to characterize behavior across scales

Connector (fastener in shear)	– B. Schafer
Interface (push-out tests)	– J. Hajjar
Diaphragm (cantilever filled deck)	– S. Easterling

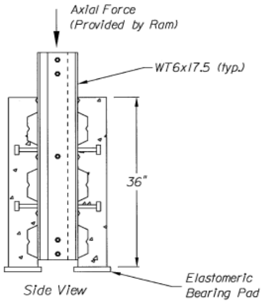
Push-out Test Database Assembled

Essentially no cyclic tests

Overview

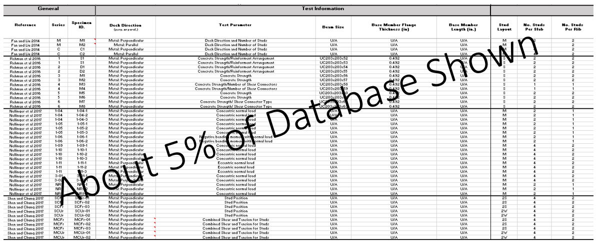
- 556 push-out tests of steel deck diaphragm with concrete fill done in 18 research programs
- Database fields (44 fields) include:
  - Test configurations (e.g. test parameter, deck orientation, loading protocol, magnitude of normal force if used, etc.)
  - Geometric properties of studs, base member, and deck (e.g. stud layout, diameter, height, rib width, metal deck type, height, gage, slab thickness, etc.)
  - Material properties (nominal and measured) of concrete, deck, and studs
  - Test results (e.g. peak force per fastener, failure mode, digitized load-slip curves, etc.)
  - Calculated fields (e.g. initial stiffness, ductility, etc.) – based on 123 digitized load-slip curves

Typical push-out test setup



Tested parameters:

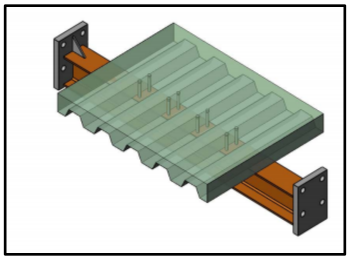
- Stud layout (strong/weak position)
- Stud number and spacing
- Stud properties
- Deck geometry
- Mesh reinforcement
- Concrete properties
- Base member flange thickness
- Normal load



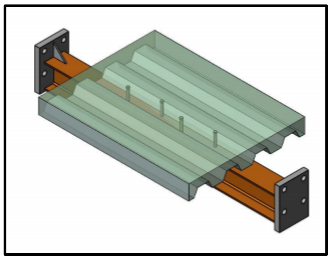
About 5% of Database Shown

Test Matrix for Cyclic Pushout Tests

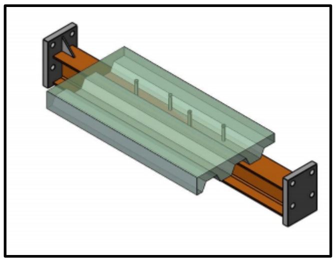
Test #	steel section	deck	section	slab thickness (")	Concrete Weight	Edge or deck Center?	Studs	Strong or weak stud location	stud diameter (in)		length (ft)	width (ft)	stud length (in.)	deck Gauge
1	parallel	perpendicular	w10x39	3.25 + 3	Lightweight	center	One @ 12" O.C.	All strong	0.75	Monotonic	5	4	4.5	20
2	parallel	perpendicular	w10x39	3.25 + 3	Lightweight	center	One @ 12" O.C.	All weak	0.75	Monotonic	5	4	4.5	20
3	parallel	perpendicular	w10x39	3.25 + 3	Lightweight	center	One @ 12" O.C.	All Weak	0.75	cyclic	5	4	4.5	20
4	parallel	perpendicular	w10x39	3.25 + 3	Lightweight	center	One @ 12" O.C.	50-50	0.75	Monotonic	5	4	4.5	20
5	parallel	perpendicular	w10x39	3.25 + 3	Lightweight	center	One @ 12" O.C.	50-50	0.75	cyclic	5	4	4.5	20
6	parallel	perpendicular	w10x39	3.25 + 3	Lightweight	center	Two @ 12" O.C.	All strong	0.75	Monotonic	5	4	4.5	20
7	parallel	perpendicular	w10x39	3.25 + 3	Lightweight	center	Two @ 12" O.C.	All weak	0.75	Monotonic	5	4	4.5	20
8	parallel	perpendicular	w10x39	3.25 + 3	Lightweight	center	Two @ 12" O.C.	All Weak	0.75	cyclic	5	4	4.5	20
9	parallel	perpendicular	w10x39	3.25 + 3	Lightweight	center	Two @ 12" O.C.	50-50	0.75	Monotonic	5	4	4.5	20
10	parallel	perpendicular	w10x39	3.25 + 3	Lightweight	center	Two @ 12" O.C.	50-50	0.75	cyclic	5	4	4.5	20
11	parallel	parallel	w10x39	3.25 + 3	Lightweight	center	One @ 12" O.C.	Alternate sides	0.75	Monotonic	5	4	4.5	20
12	parallel	parallel	w10x39	3.25 + 3	Lightweight	center	One @ 12" O.C.	Alternate sides	0.75	cyclic	5	4	4.5	20
13	parallel	parallel	w10x39	4.5 + 3	Normal	center	One @ 12" O.C.*	Alternate sides	0.75	Monotonic	5	4	4.5	20
14	parallel	parallel	w10x39	4.5 + 3	Normal	center	One @ 12" O.C.*	Alternate sides	0.75	cyclic	5	4	4.5	20
15	parallel	parallel	w10x39	3.25 + 3	Lightweight	edge	One @ 12" O.C.	Alternate sides	0.75	Monotonic	5	2'-8"	4.5	20
16	parallel	parallel	w10x39	3.25 + 3	Lightweight	edge	One @ 12" O.C.	Alternate sides	0.75	cyclic	5	2'-8"	4.5	20



Beam Perpendicular

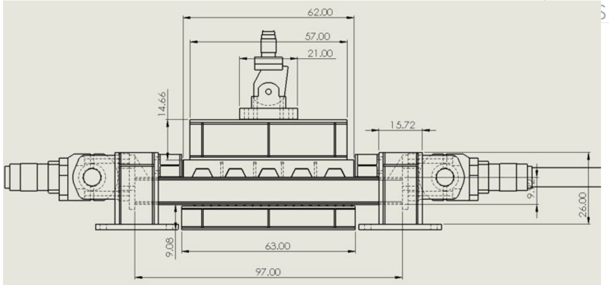


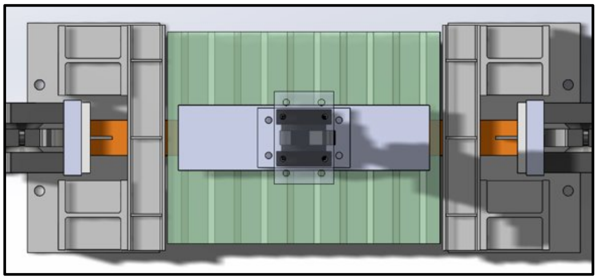
Beam Parallel

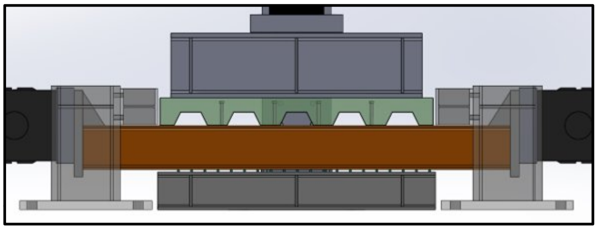


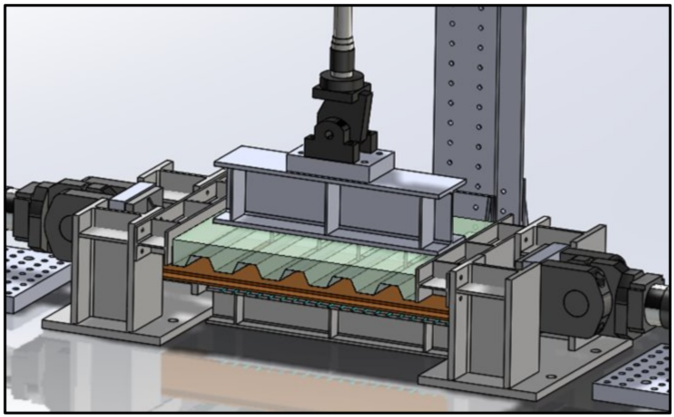
Parallel with Edge

Push-out Test Setup

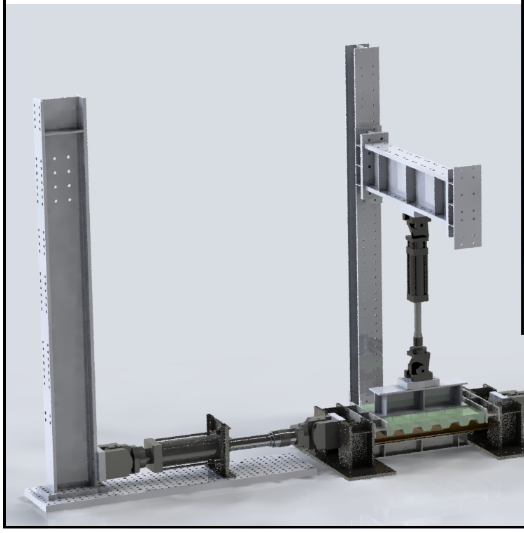








Setup and Status



STReSS LAB
Laboratory for Structural Testing of Resilient and Sustainable Systems

New cyclic testing to characterize behavior across scales

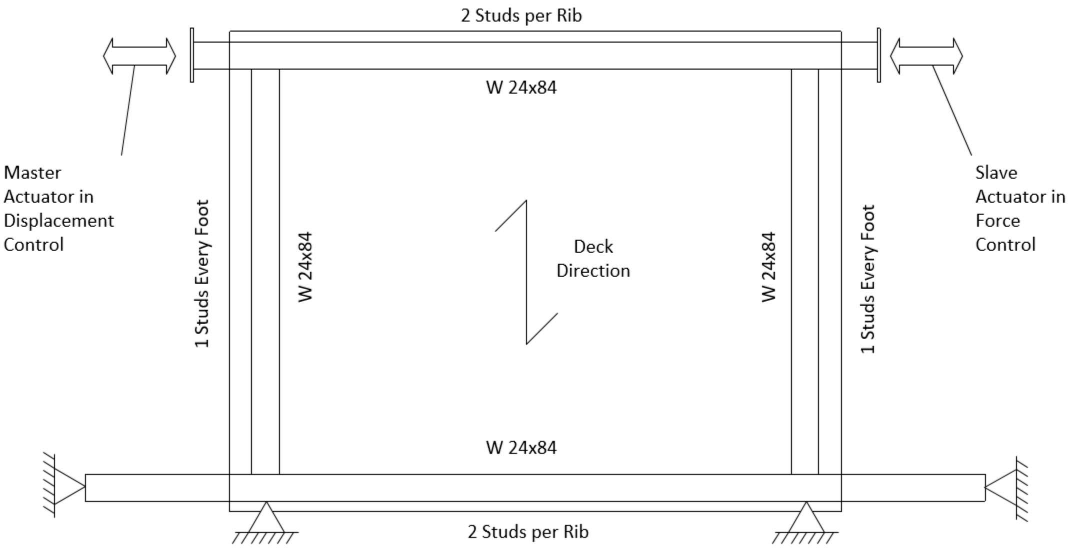
Connector (fastener in shear) – B. Schafer
Interface (push-out tests) – J. Hajjar
Diaphragm (cantilever filled deck) – S. Easterling

Cantilever Diaphragm Testing Program on Composite Slabs

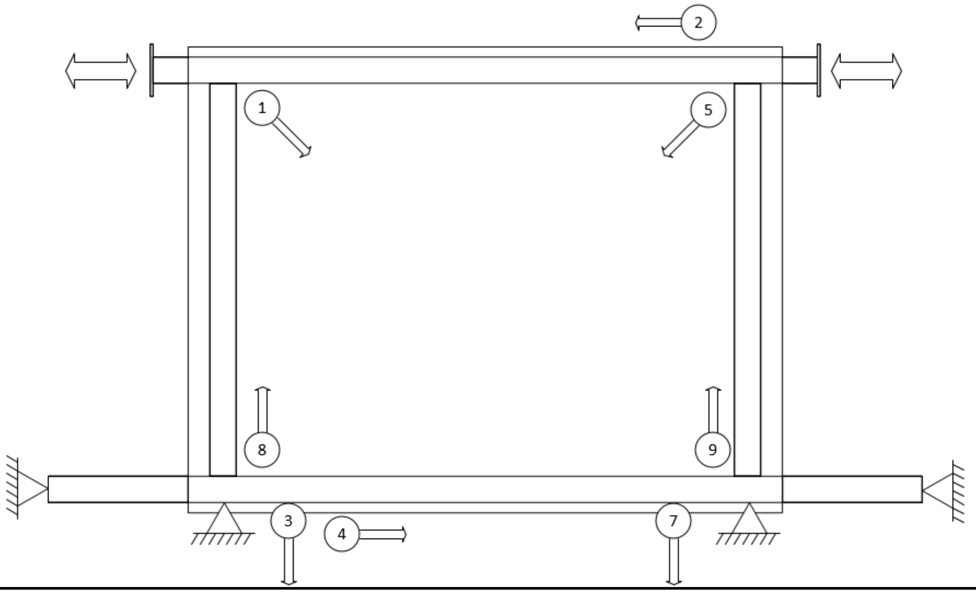
- 6.25" thick slab
  - 20 gage Verco W3 Steel Deck with 3.25" concrete cover
- 4000 psi lightweight concrete mix
- Goal for specimen: Typical 2 hour fire rating for LW
- 2 studs per rib (every 12") perpendicular to deck ribs
- 1 stud every 12" parallel to deck ribs

Test Specimen	Steel Deck	Concrete Type	Total Thickness (in)	Proposed Shear Strength (kip)	Objective
3/6.25-4-L-NF-DT	3	Lightweight	6.25	136	Typical 2 Hr Fire Rating for LW
3/7.5-4-N-NF-DT	3	Normalweight	7.5	219	Typical 2 Hr Fire Rating for NW
2/4-4-N-NF-DT	2	Normalweight	4	109	Thin assembly using NW
2/4-4-L-NF-DT	2	Lightweight	4	82	Thin assembly using LW
3/6.25-4-L-NF-P	3	Lightweight	6.25	98	Fail Studs with LW
3/7.5-4-N-NF-P	3	Normalweight	7.5	99	Fail Studs with NW

Test Setup



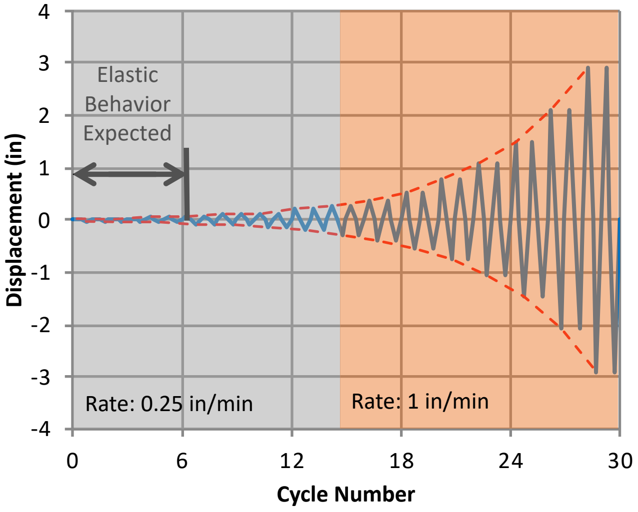
Instrumentation Setup

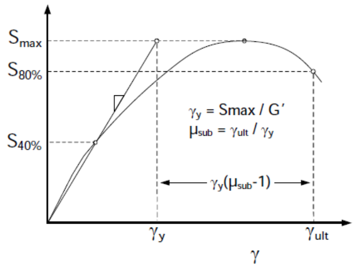


Loading Protocol

- Based on FEMA 461
- 40% increase in amplitude between displacement steps
- 6 Cycles before reaching elastic limit
- Elastic limit predicted using predicted stiffness and ultimate strength
  - γ_y of 0.072”

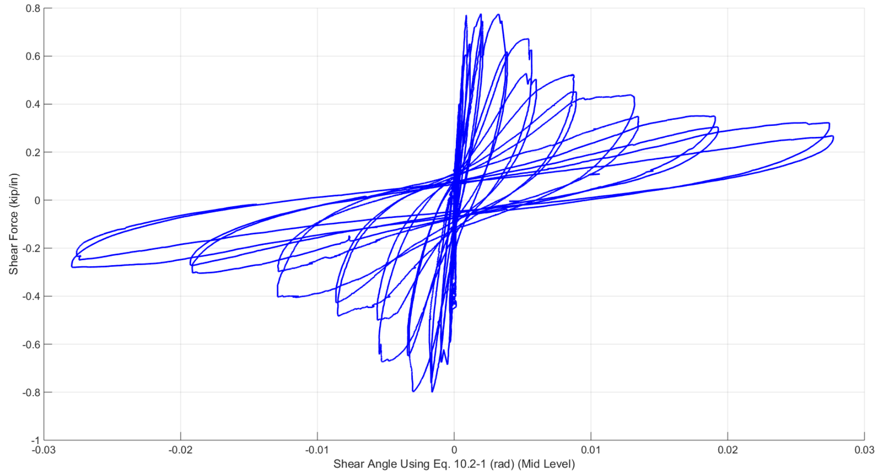
Disp. Step	Δ (in)
1	0.0262
2	0.0367
3	0.0514
4	0.0720
5	0.1008
6	0.1411
7	0.1976
8	0.2766
9	0.3872
10	0.5421
11	0.7590
12	1.0626
13	1.4876
14	2.0826
15	2.9157
16	4.0820

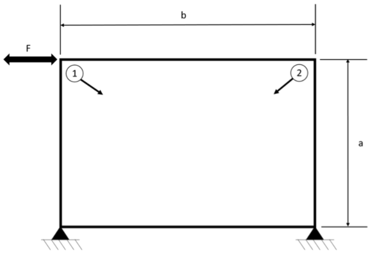




O’Brien, P.O., Eatherton, M.R., Easterling, W.S. (2016) “Characterizing the Load-Deformation Behavior of Steel Deck Diaphragms Using Past Test Data”

Cantilever Diaphragm Testing Program on Composite Slabs



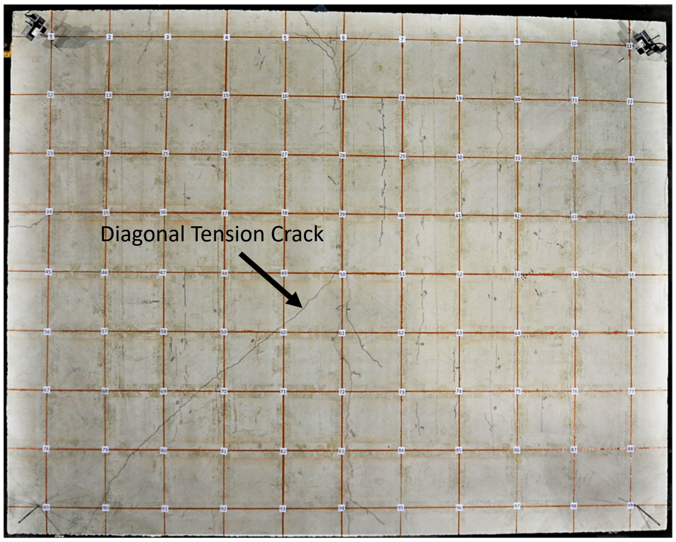


$$\Delta_i = \left(|\Delta_1| + |\Delta_2|\right)\frac{\sqrt{a^2+b^2}}{2b} \qquad (Eq.\ 10.2\text{-}1)$$

AISI S907-13 – Test Standard for Cantilever Test Method for Cold-Formed Steel Diaphragms

Diagonal Tension Cracking

- First Diagonal tension crack developed at shear angle of 0.009 rad

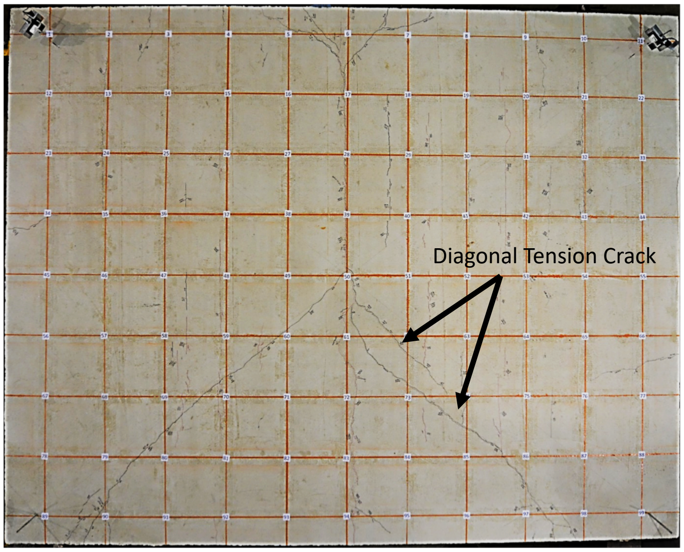


Diagonal Tension Crack

SDII

Diagonal Tension Cracking

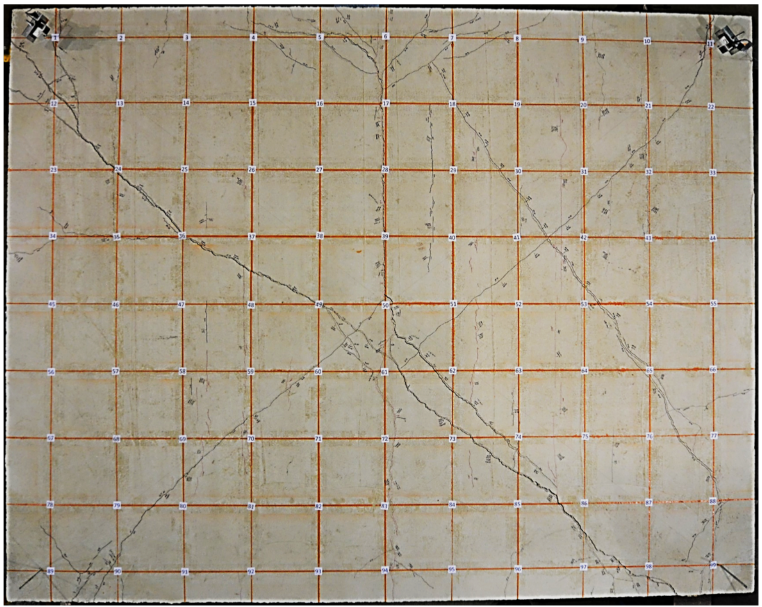
- Second Diagonal tension crack developed at shear angle of 0.0083 rad



Diagonal Tension Crack

SDII

Diagonal Tension Cracking

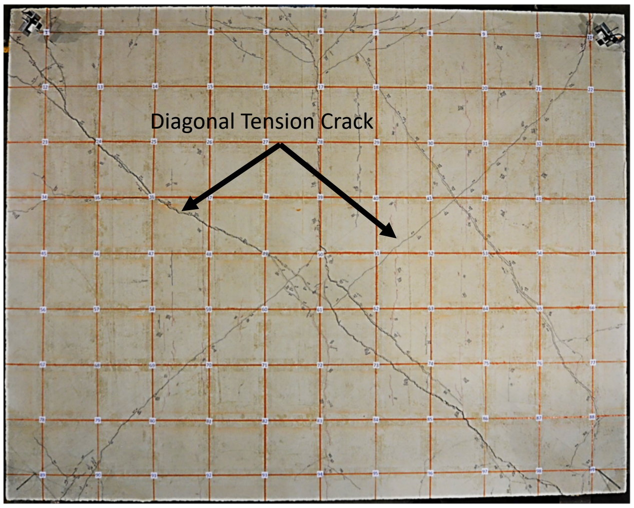


Cantilever Diaphragm Testing Program on Composite Slabs

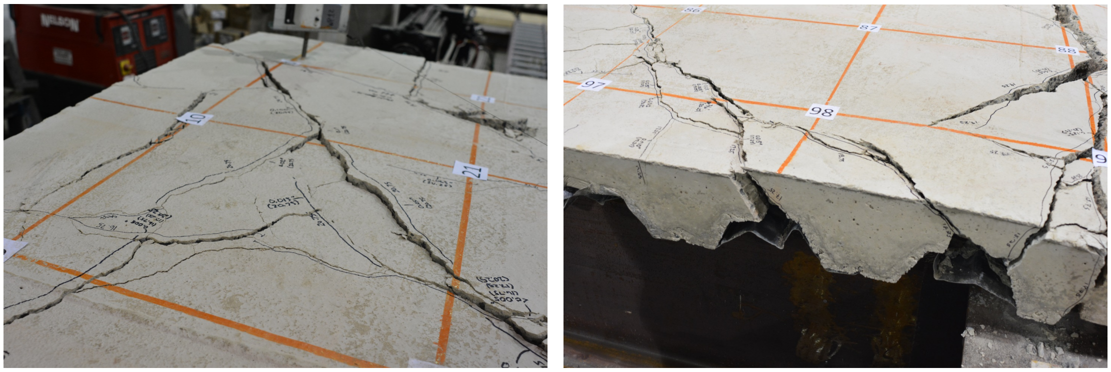
Summary of Results:

- Predicted Stiffness: 1507 kip/in
- Recorded Stiffness: 1248 kip/in

- Predicted Ultimate Strength: 136 kips
- Recorded Ultimate Strength: 139 kips



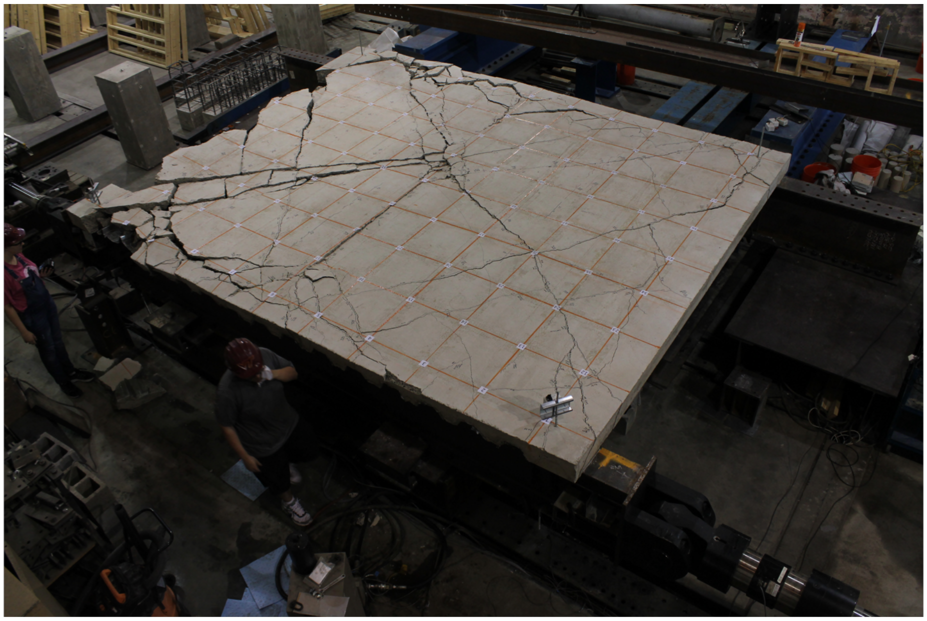
Failure Past Ultimate



Failure Past Ultimate

CFSRC

Final State: Cantilever Diaphragm Test on Composite Slab



CFSRC

Companion Monotonic Push-Out Tests Ongoing

Status	Test Specimen	Position of Stud within Rib	Stud Tensile Strength (ksi)	Steel Reinf.	Concrete Type	Total Slab Thickness (in)
Complete	W1-3/6.25-4-L-NF	Weak	82	N/A	LW	6.25
	W2-3/6.25-4-L-NF	Weak	82	N/A	LW	6.25
	W3-3/6.25-4-L-NF	Weak	82	N/A	LW	6.25
	S1-3/6.25-4-L-NF	Strong	82	N/A	LW	6.25
	S2-3/6.25-4-L-NF	Strong	82	N/A	LW	6.25
	S3-3/6.25-4-L-NF	Strong	82	N/A	LW	6.25
	SR1-3/6.25-4-L-NF	Strong	82	(4) #5 bars	LW	6.25
	SR2-3/6.25-4-L-NF	Strong	82	(4) #5 bars	LW	6.25
	SR3-3/6.25-4-L-NF	Strong	82	(4) #5 bars	LW	6.25
In Progress	W1-3/7.5-4-N-NF	Weak	82	N/A	NW	7.5
	W2-3/7.5-4-N-NF	Weak	82	N/A	NW	7.5
	S1-3/7.5-4-N-NF	Strong	82	N/A	NW	7.5
	S2-3/7.5-4-N-NF	Strong	82	N/A	NW	7.5
	SR1-3/7.5-4-N-NF	Strong	82	(4) #5 bars	NW	7.5
	SR2-3/7.5-4-N-NF	Strong	82	(4) #5 bars	NW	7.5
	SL1-3/7.5-4-N-NF	Strong	72	N/A	NW	7.5
	SL2-3/7.5-4-N-NF	Strong	72	N/A	NW	7.5
	WL1-3/7.5-4-N-NF	Weak	72	N/A	NW	7.5
	WL2-3/7.5-4-N-NF	Weak	72	N/A	NW	7.5
	SSM2-3/7.5-4-N-NF	Strong	82	N/A	NW	7.5
	SSM3-3/7.5-4-N-NF	Strong	82	N/A	NW	7.5

Planned large scale testing

J. Hajjar

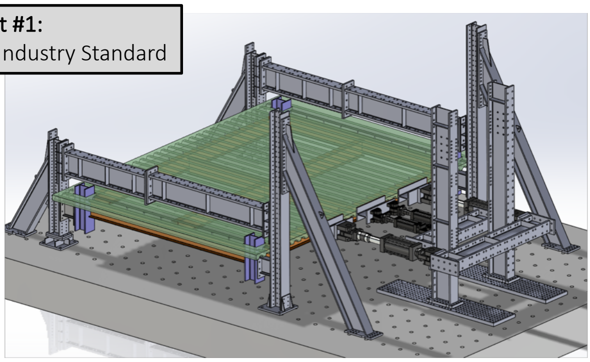
Full-Scale Beam-Style Cyclic Diaphragm Testing

Unlike cantilever tests, here the beams and girders intentionally contribute – this is a full-scale floor diaphragm test…

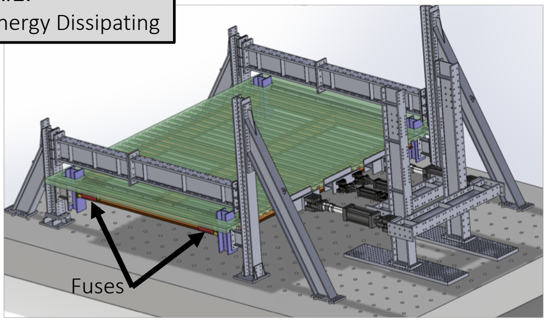
Test #	Test Type	Deck	Slab Thickness (in)	Concrete Weight	Stud Diameter (in)	Loading	Length (ft)	Width (ft)	Stud Length (in.)
1	Industry Standard	Parallel to Chords	3.25 + 3	Light Weight	0.75	Cyclic	28	20	4.5
2	Energy Dissipating	Parallel to Chords	3.25 + 3	Light Weight	0.75	Cyclic	28	20	4.5

Under Consideration: load introduction methodology and loading history

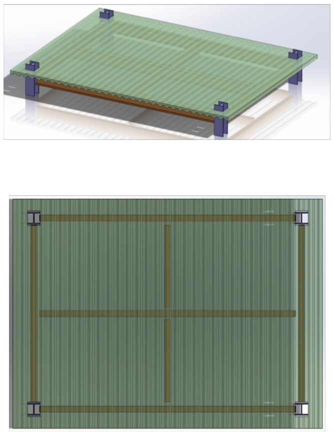
Test #1:
Industry Standard



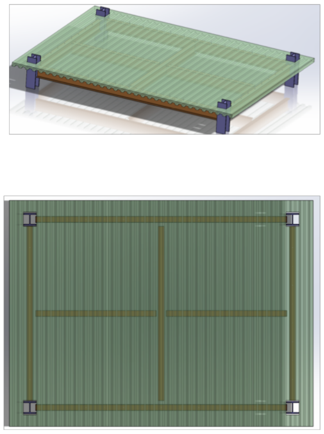
Test #2:
Energy Dissipating



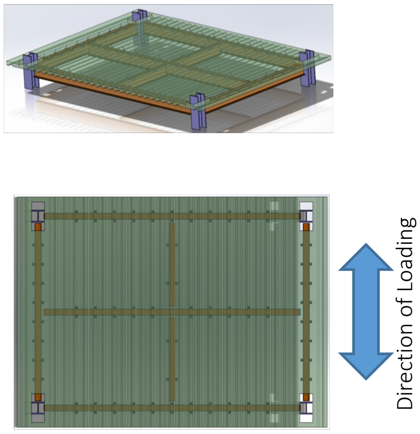
Specimen Variation



Chord Continuous

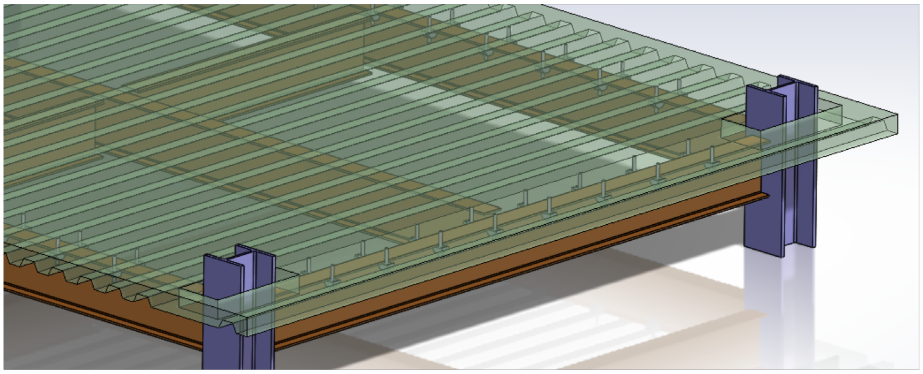


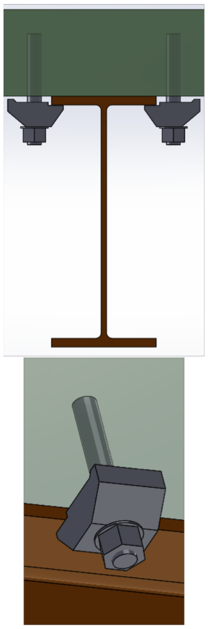
Collector Continuous



Replace studs with clamps

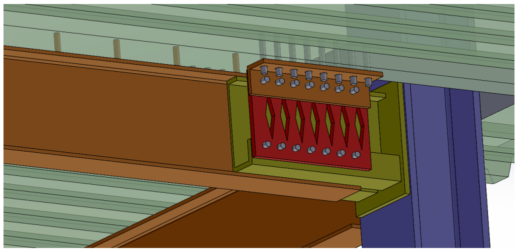
Energy Dissipating Fuse – Clamp From Below

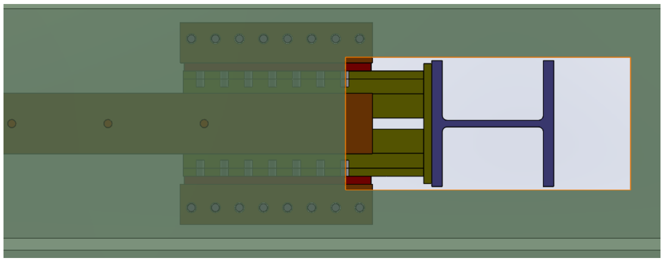


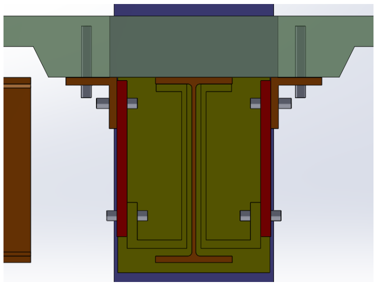


- Clamps consist of threaded studs embedded into the concrete with cast connectors with nut and washer on either side of the top flange.
- Connectors bite into the flange as the nut is tightened
- Lateral force will cause the concrete to move relative to the collector, with a cutout at the columns enabling slip.

Energy-Dissipating Fuse at the LRFS Connection

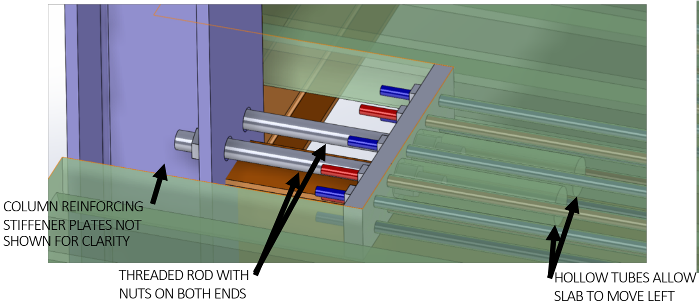


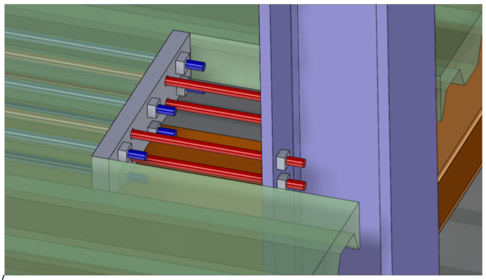


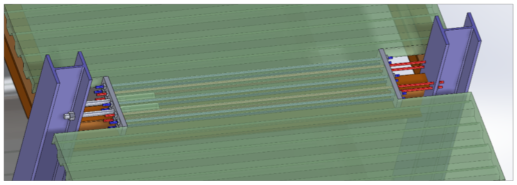


- Lateral force transfers from the diaphragm to the column through a butterfly fuse
- A linear guide keeps collector and fuse aligned
- The fuse can run down much of the length of the collector depending on the amount of force required to transfer
- Lateral force will cause the concrete to move relative to the collector, with a cutout at the columns enabling slip.

Self-Centering Mechanism







- Self-center mechanism may be considered to re-center the diaphragm through opposing post-tensioned rods, similar in concept to a Self-Centering BRB.
- Movement in either direction produces additional tension in either the red or blue rods, while endplates restrict or allow movement of different parts of the system.
- The large silver rods with nuts act as an anchor for the left endplate during movement toward the right.
- The self-centering system is compatible with the fuses on the previous slides.

Leveraging simulation to improve understanding of behavior

Vertical vs. Horizontal LFRS – B. Schafer
Building scale simulations – M. Eatherton
Bringing fracture into models – J. Hajjar
Optimization – B. Schafer

Leveraging simulation to improve understanding of behavior

Vertical vs. Horizontal LFRS – B. Schafer
Building scale simulations – M. Eatherton
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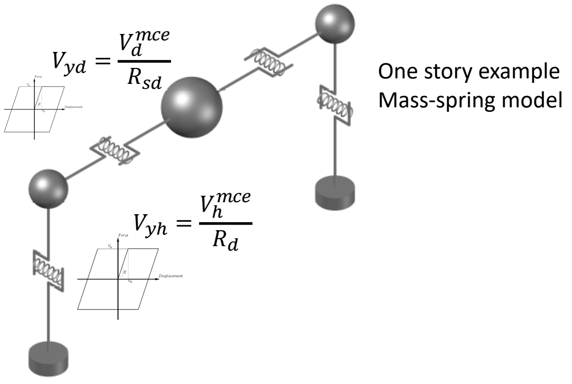
SDII Mass-Spring Models

- Simplified mass-spring models from 1 to 12 stories studied to explore R vs R_s or vLFRS vs hLFRS issues.
- Large parameter variation across *m*, *K*, *T*, yielding of both vertical and horizontal systems
- Inelastic time history analysis across P695 EQ suite
- Allows for broad discussion on the impact of ductility in the walls, floors, or both on the force levels and drift demands expected in the system given R and R_s.



$$V_{yd} = \frac{V_d^{mce}}{R_{sd}}$$

$$V_{yh} = \frac{V_h^{mce}}{R_d}$$

One story example Mass-spring model

R_d

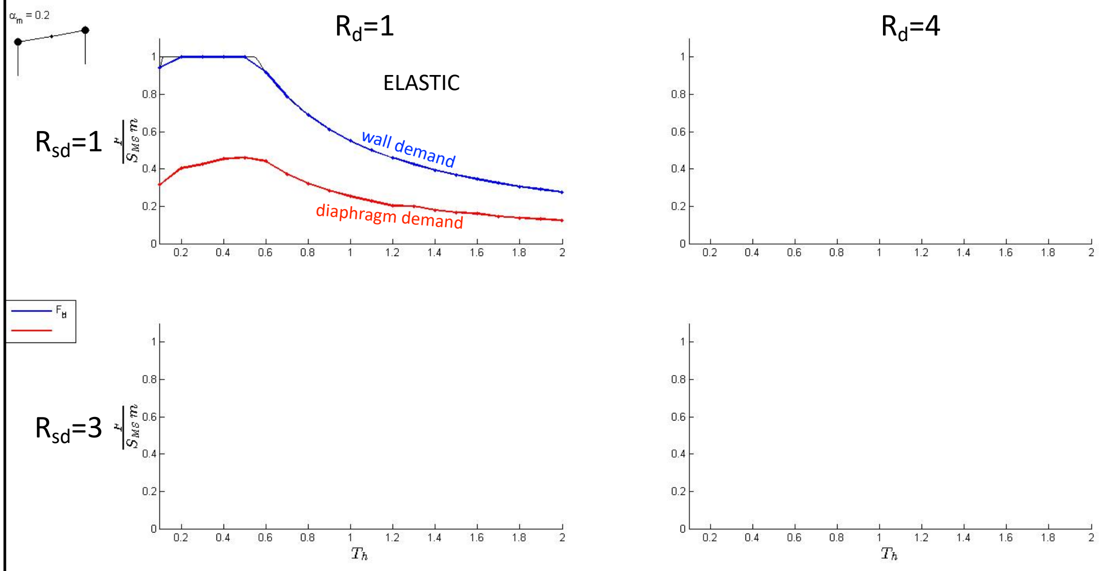
	Elastic	OCBF	SMF & BRB	
R	1	4	3	8
Ω_0	1	2	1	1.6
R_d	**1.0**	**2.0**	**3.0**	**5.0**

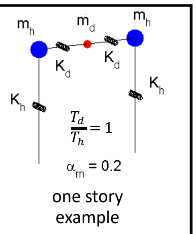
R_{sd}

	Elastic	Precast RDO	"Steel"	Wood
R_s	1	1.4	2.2	3
R_{so}	1	1	1.1	1.2
R_{sd}	**1.0**	**1.4**	**2.0**	**2.5**

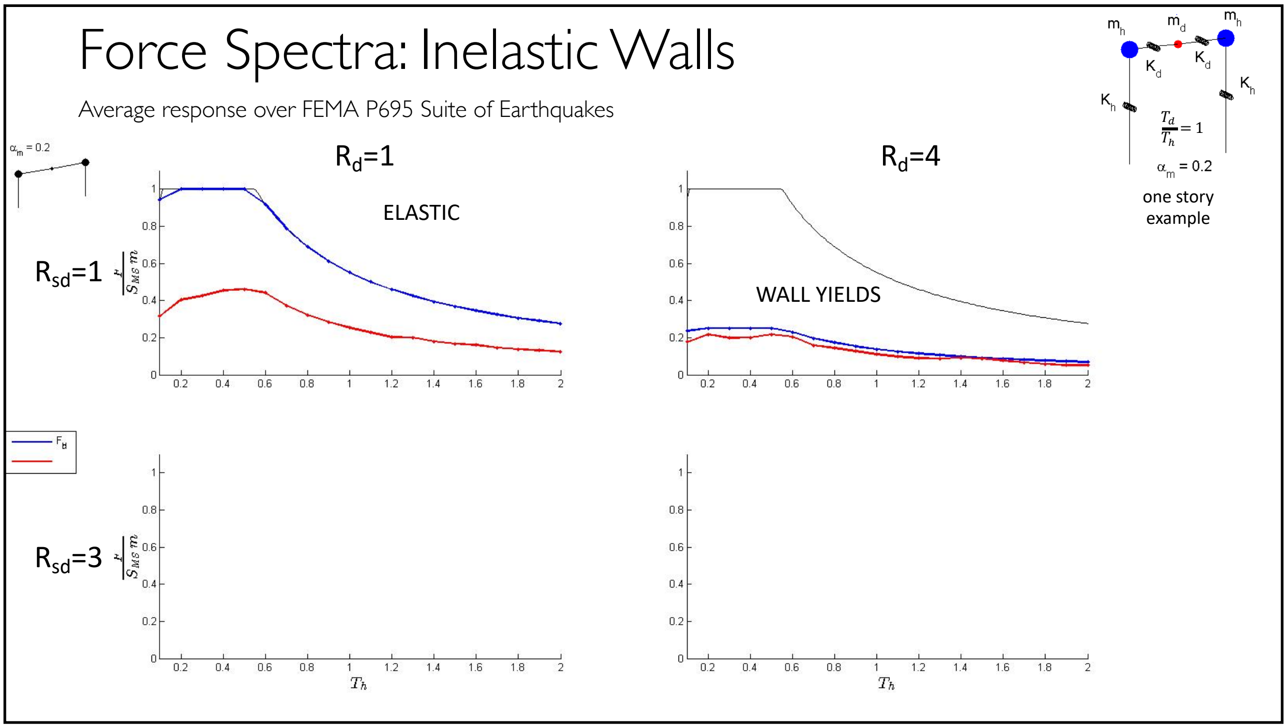
Force Spectra: Elastic

Average response over FEMA P695 Suite of Earthquakes



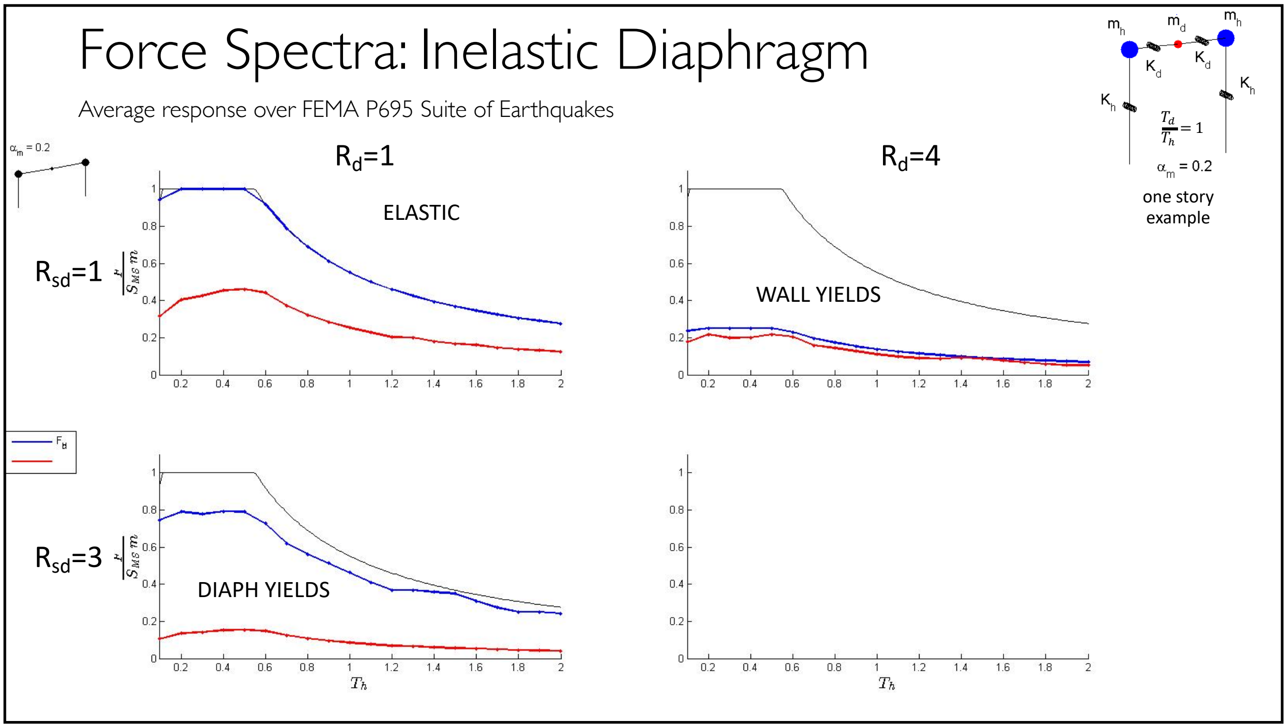


one story example



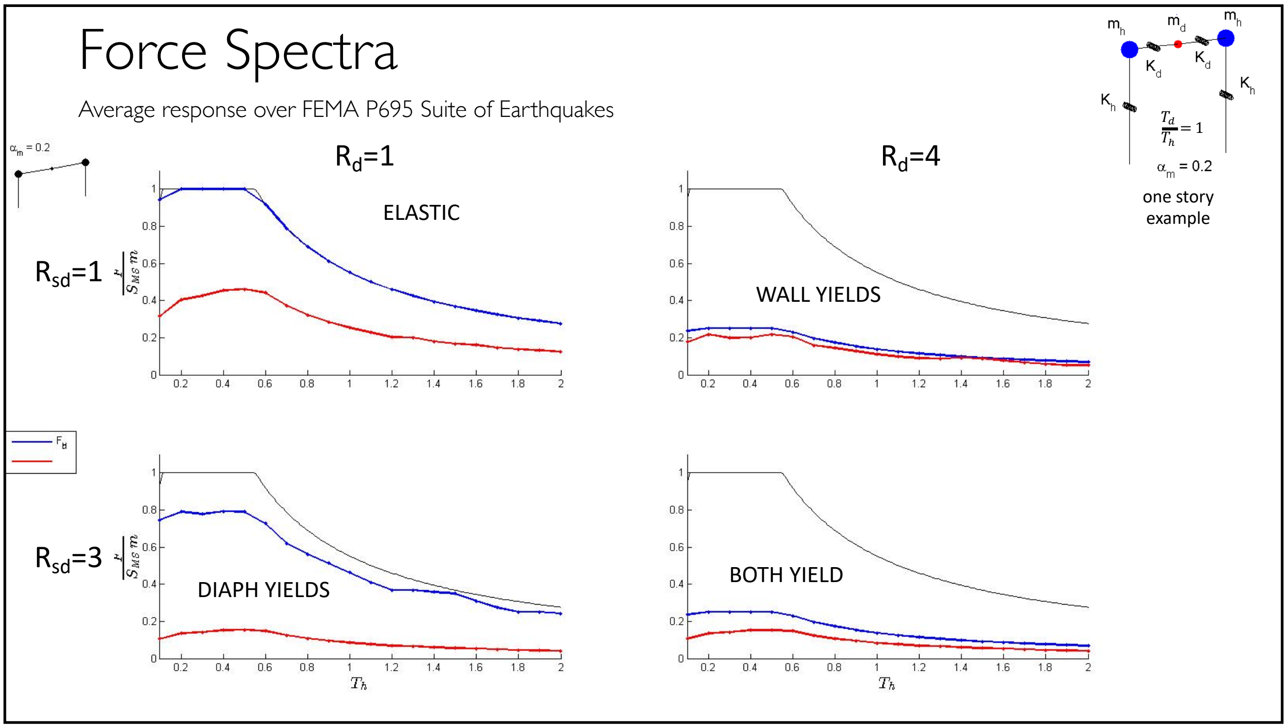
Force Spectra: Inelastic Walls

Average response over FEMA P695 Suite of Earthquakes



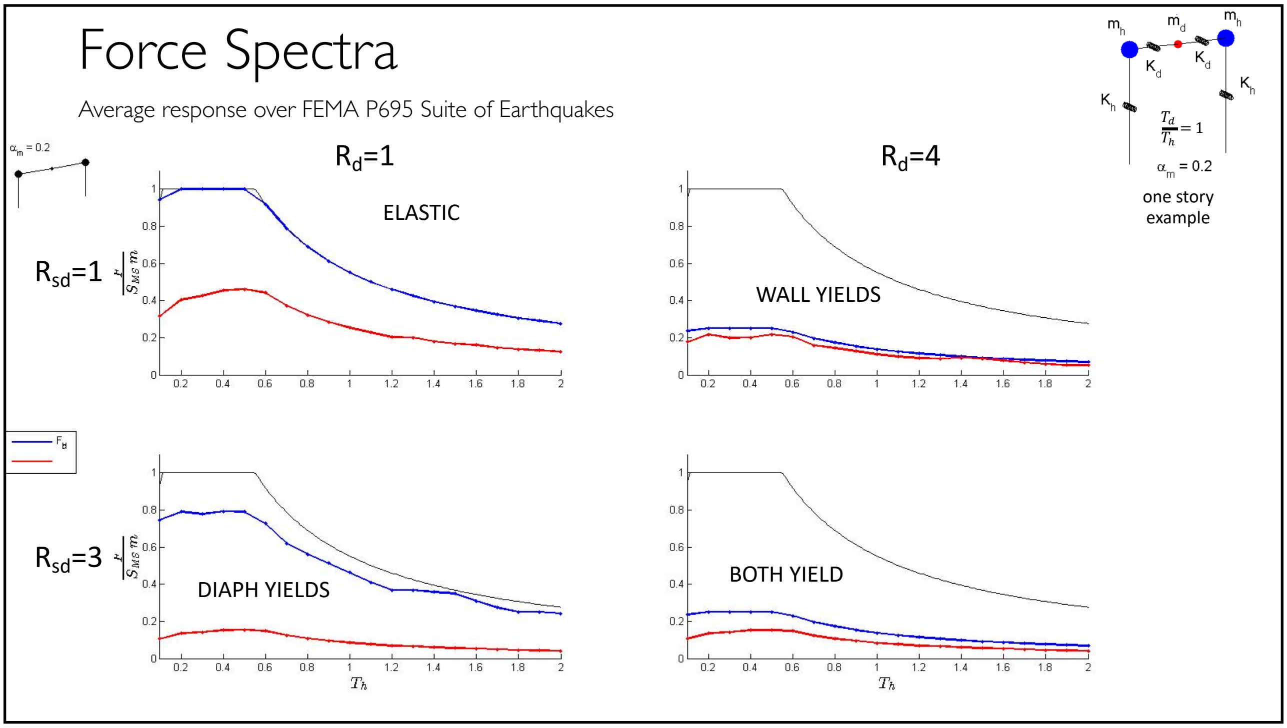
Force Spectra: Inelastic Diaphragm

Average response over FEMA P695 Suite of Earthquakes



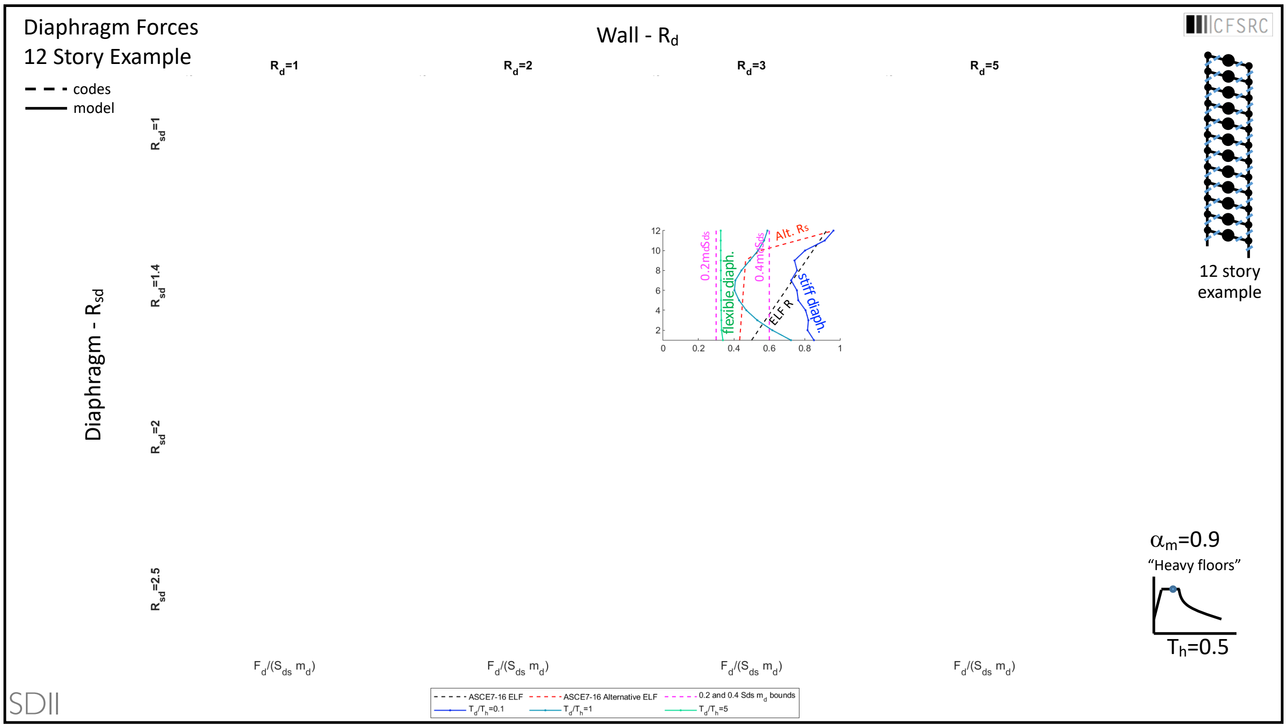
Force Spectra

Average response over FEMA P695 Suite of Earthquakes



Force Spectra

Average response over FEMA P695 Suite of Earthquakes



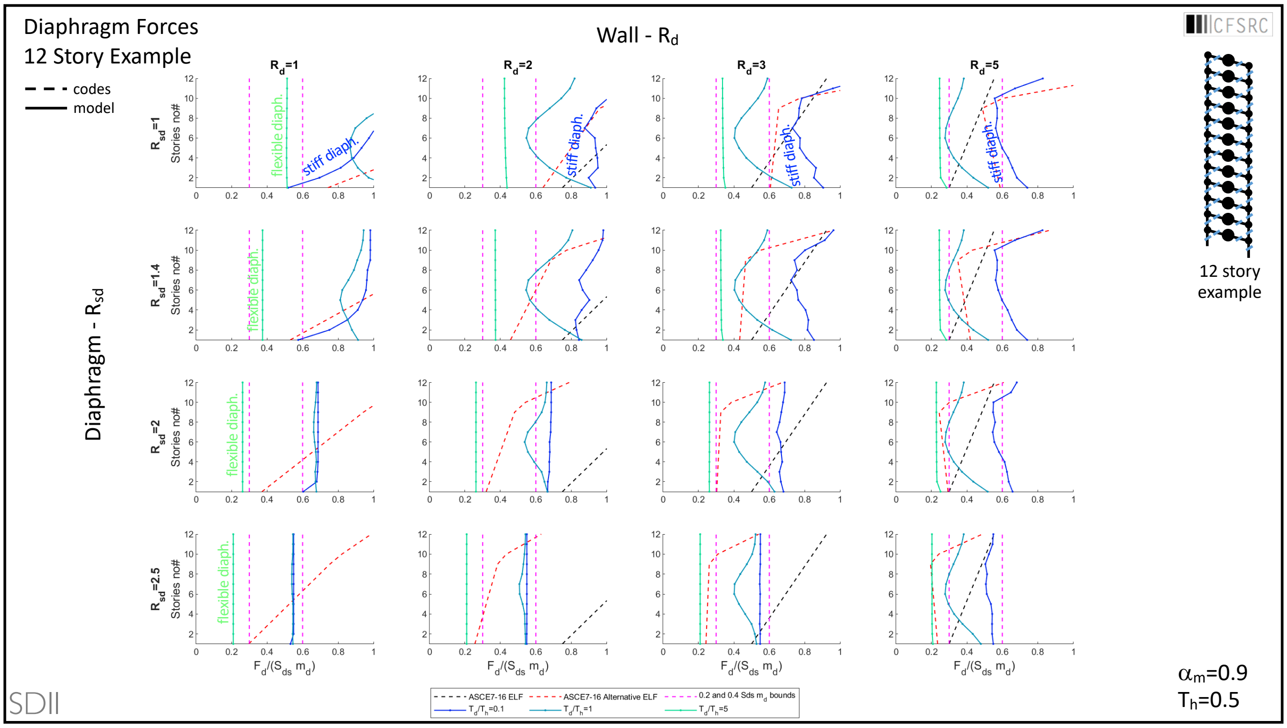
Diaphragm Forces
12 Story Example

Wall - R_d

12 story example

α_m=0.9
"Heavy floors"

T_h=0.5



Diaphragm Forces
12 Story Example

Wall - R_d

12 story example

α_m=0.9
T_h=0.5

Discussion of mass-spring model findings

- Still digesting full implications of these models
- Running real building archetypes to validate/reject findings in specific cases, as discussed in next section
- Ductility demands in addition to force demands explored

- R and R_s applied independently may not result in expected behavior
- A wall's ability to shield a diaphragm may be much greater than a diaphragm's ability to shield a wall (strongly so in one story building)
- Accurate force and ductility prediction, particularly for diaphragm, likely to require system level prediction, not just diaphragm
- All of which create interesting challenges and implications going forward

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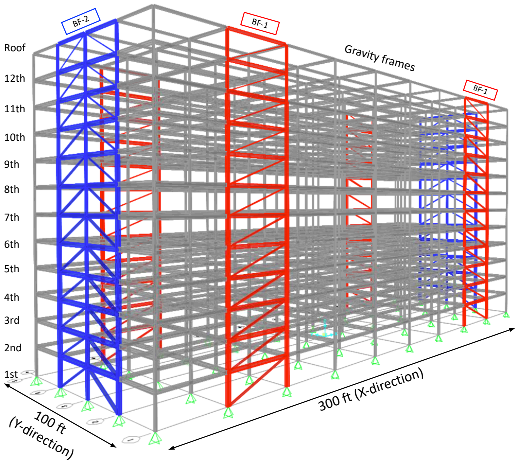
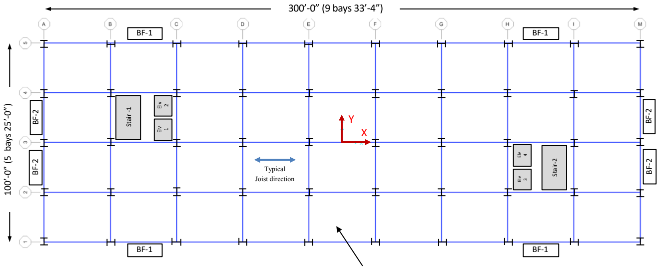
Leveraging simulation to improve understanding of diaphragm and building behavior

Building Scale Archetype Simulations

SDII Building Archetypes

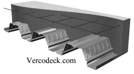


Typical plan

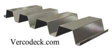


Building Dimensions		
Width (E-W)	100.00	(ft)
Length (N-S)	300.00	(ft)
H (First story)	14.00	(ft)
H (Typical story)	12.50	(ft)
Bay size X	33.33	(ft)
Bay size Y	25.00	(ft)
Parapet	3.00	(ft)
Number of Stories	12	(1, 4, 8,12)
H (total height)	151.5	(ft)

Typical floors:
6.25” total slab deck,
Light weight concrete,
2 Hours fire rating



Roof:
Bare Steel deck



- Series of designs, high seismic, high diaphragm utilization
- 1, 4, 8, and 12 stories; NWC and LWC Floors,
- BRB and OCBF vLFRS (SMRF later)
- Diaphragm design: Traditional, Alt. R_s=1, Alt. R_s=3

Diaphragm demands (ASCE7-16 and ASCE7-16 Alt. R_s=1, 3)

ASCE7-16 Standard

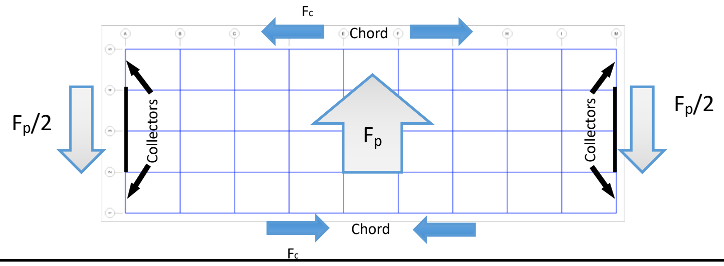
Level	F_i(k)	W_i (k)	F_P (k)	$F_{P\text{-min}}$ (k)	$F_{P\text{-max}}$ (k)	**F_P (k) design**
Roof	145	1271	145	262	524	**262**
12th	252	2545	264	524	1049	**524**
11th	215	2545	245	524	1049	**524**
10th	181	2545	227	524	1049	**524**
9th	149	2545	209	524	1049	**524**
8th	120	2545	193	524	1049	**524**
7th	94	2545	178	524	1049	**524**
6th	70	2545	163	524	1049	**524**
5th	49	2545	150	524	1049	**524**
4th	31	2545	137	524	1049	**524**
3rd	16	2545	126	524	1049	**524**
2nd	6	2545	115	524	1049	**524**

ASCE7 Alt. R_s=3

F_P (k) design
262
524
524
524
524
524
524
524
524
524
524
524

ASCE7 Alt. R_s=1

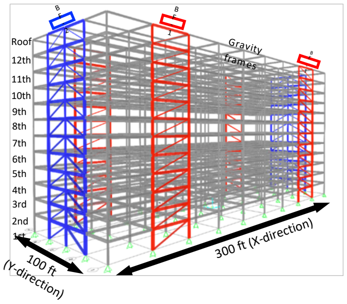
F_P (k) design
419
839
839
851
873
895
916
938
959
981
1003
1024



Evaluate diaphragm design methodologies:

1. Conventional design
2. Alternative with Rs=1
3. Alternative with Rs=3
4. Others

Building Models in OpenSees



Modeling in SAP2000:

- Rigid diaphragm or Elastic Area Elements for Diaphragm
- Typical design practice

Modeling in OpenSees:

- Nonlinear elements for both vertical LFRS and diaphragm
- Predict actual building behavior

Types of Analyses:

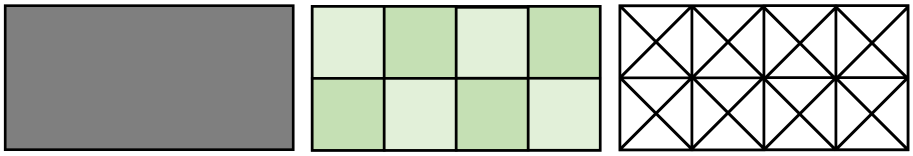
- Elastic using design loads
- Nonlinear static (Push-Over)
- Nonlinear response history

Research Objectives:

- Investigate load path and magnitude of loads in diaphragms
- Study interaction of inelasticity in vertical LFRS and inelasticity in diaphragm
- Compare loads and load path to design values and assumptions
- Test new design approaches
- Advance modeling tools for diaphragms

Diaphragm modeling options

Options for Reduced Order Diaphragm Models in a Building Model:



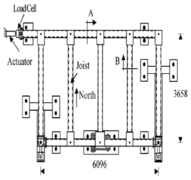
Rigid

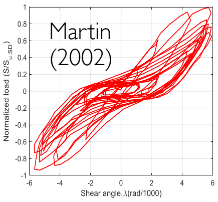
Elastic or Nonlinear Area Elements

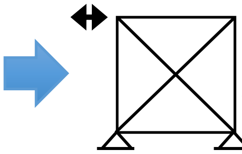
Nonlinear Truss Elements

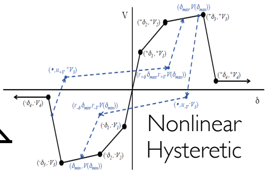
Calibration of the Model:

- Past Diaphragm Cantilever Tests (e.g., Martin (2002), Easterling (1987))
- Nonlinear Hysteretic Model (e.g. Pinching 4 Material in OpenSees)







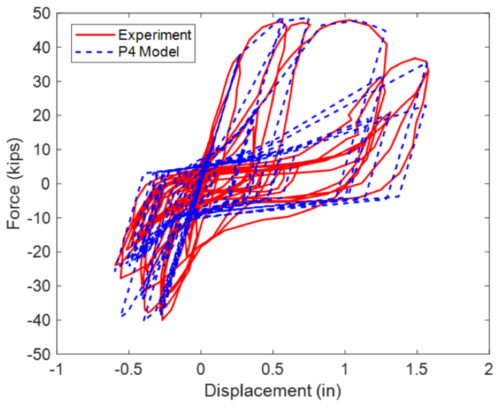


Nonlinear Hysteretic

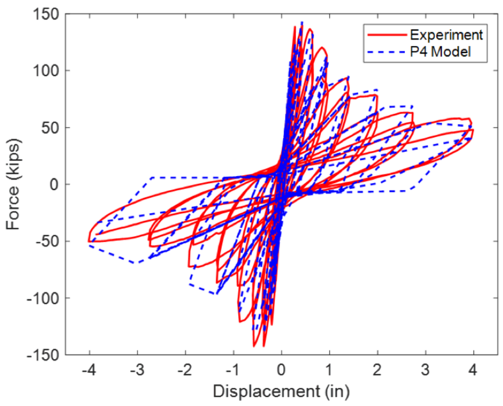
Calibrating diaphragm truss model to tests

Calibrated results to SDII experimental database:

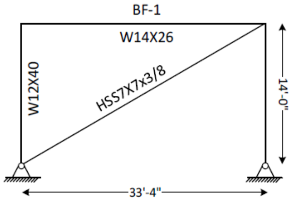
Specimen 33 (Martin 2002)

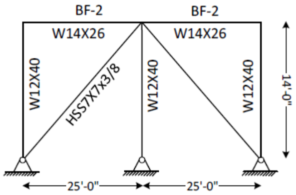


Specimen 4 (Virginia Tech)

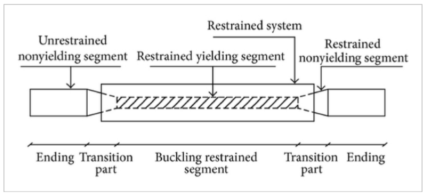


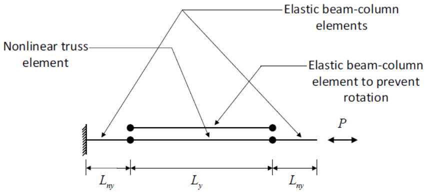
Calibrating BRB truss model to tests

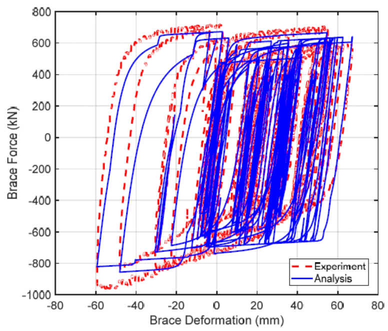




Assuming rigid beam-to-column connections

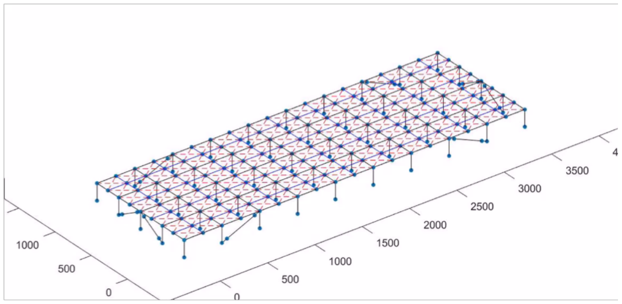




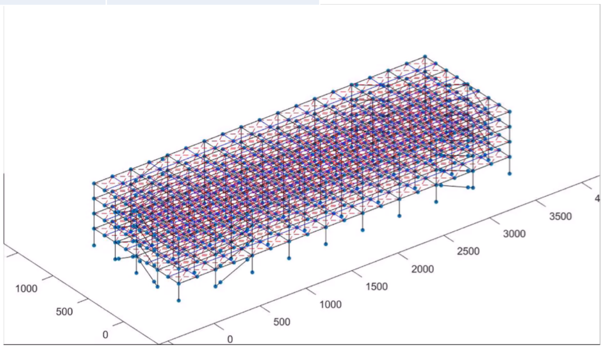


Eigenvalue Analysis Example

	1st period (sec)	2nd period (sec)
1-story archetype	0.98 (Transverse)	0.59 (Longitudinal)
4-story archetype	1.13 (Longitudinal)	1.10 (Transverse)



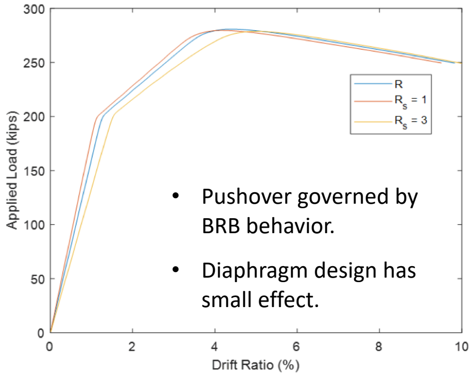
One-Story
1st Mode



Four-Story
2nd Mode

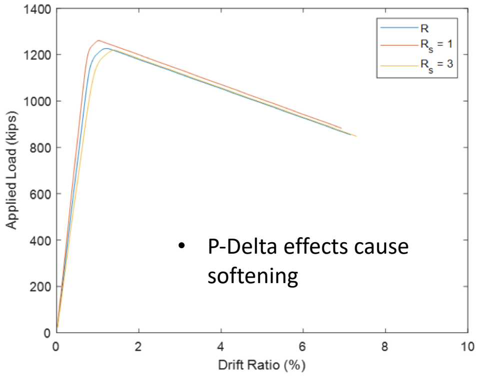
BRB Buildings – Example Pushover Analysis

One-story



- Pushover governed by BRB behavior.
- Diaphragm design has small effect.

Four-story

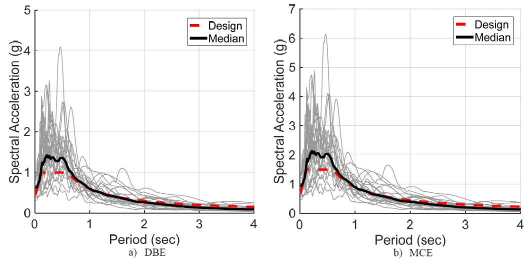


- P-Delta effects cause softening

Pushover curves plotted up to tested BRB strain and diaphragm shear angle (non-simulated collapse limits)

Nonlinear Response History Analysis of Archetype Buildings

- 44 ground motions from P695 suite far-field set. Applied in bi-directional pairs.
- 3 scale levels:
  1. DBE (10% in 50 years), Scale factor =1.29 (1-story), 1.67 (4-story)
  2. MCE (2% in 50 years), Scale factor =1.94 (1-story), 2.50 (4-story)
  3. A third scale level based on an adjusted collapse margin ratio (ACMR) from FEMA P695 10% probability of collapse, Scale factor =2.47 (1-story), 3.07 (4-story)



a) DBE b) MCE

Table 7-1a Spectral Shape Factor (*SSF*) for Archetypes Designed for SDC B, SDC C, or SDC D_{min}

T (sec.)	Period-Based Ductility, μ_T							
	1.0	1.1	1.5	2	3	4	6	≥ 8
≤ 0.5	1.00	1.02	1.04	1.06	1.08	1.09	1.12	1.14
0.6	1.00	1.02	1.05	1.07	1.09	1.11	1.13	1.16
0.7	1.00	1.03	1.06	1.08	1.10	1.12	1.15	1.18
0.8	1.00	1.03	1.06	1.08	1.11	1.14	1.17	1.20
0.9	1.00	1.03	1.07	1.09	1.13	1.15	1.19	1.22
1.0	1.00	1.04	1.08	1.10	1.14	1.17	1.21	1.25
1.1	1.00	1.04	1.08	1.11	1.15	1.18	1.23	1.27
1.2	1.00	1.04	1.09	1.12	1.17	1.20	1.25	1.30
1.3	1.00	1.05	1.10	1.13	1.18	1.22	1.27	1.32
1.4	1.00	1.05	1.10	1.14	1.19	1.23	1.30	1.35
≥ 1.5	1.00	1.05	1.11	1.15	1.21	1.25	1.32	1.37

$$\mu_T = \frac{\delta_u}{\delta_{y,eff}}$$

$$\delta_{y,eff} = C_O \frac{V_{max}}{W}\left[\frac{g}{4\pi^2}\right](\max(T,T_1))^2$$

$$C_0 = \phi_{1,r}\frac{\sum_1^N m_x \phi_{1,x}}{\sum_1^N m_x \phi_{1,x}^2}$$

Table 7-3 Acceptable Values of Adjusted Collapse Margin Ratio ($ACMR_{10\%}$ and $ACMR_{20\%}$)

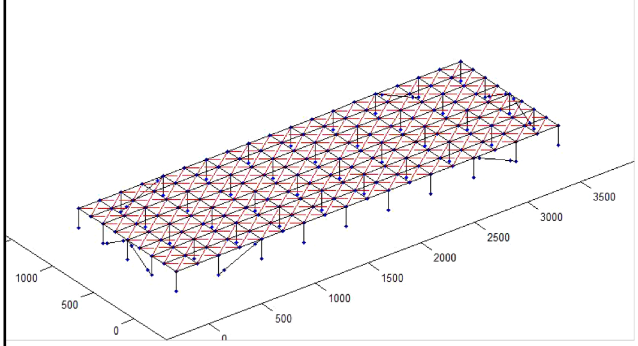
Total System Collapse Uncertainty	Collapse Probability				
	5%	10% ($ACMR_{10\%}$)	15%	20% ($ACMR_{20\%}$)	25%
0.275	1.57	**1.42**	1.33	**1.26**	1.20
0.300	1.64	**1.47**	1.36	**1.29**	1.22
0.325	1.71	**1.52**	1.40	**1.31**	1.25
0.350	1.78	**1.57**	1.44	**1.34**	1.27
0.375	1.85	**1.62**	1.48	**1.37**	1.29
0.400	1.93	**1.67**	1.51	**1.40**	1.31
0.425	2.01	**1.72**	1.55	**1.43**	1.33
0.450	2.10	**1.78**	1.59	**1.46**	1.35
0.475	2.18	**1.84**	1.64	**1.49**	1.38
0.500	2.28	**1.90**	1.68	**1.52**	1.40
0.525	2.37	1.96	1.72	1.56	1.42
0.550	2.47	**2.02**	1.77	**1.59**	1.45

$$ACMR_i = SSF_i \times CMR_i$$

$$CMR = \frac{\hat{S}_{CT}}{S_{MT}}$$

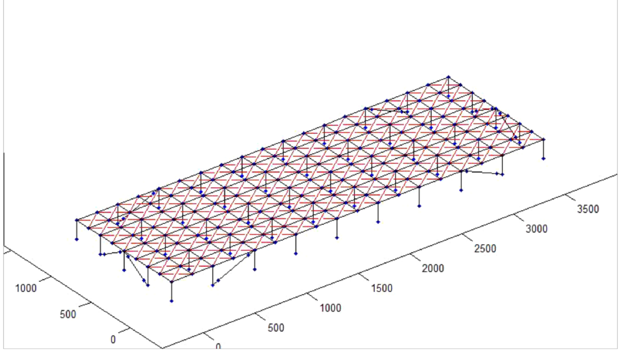
Nonlinear Response History Analysis of Archetype Buildings

Animated Response of One-Story Building



DBE Level

BRB Inelastic, Diaphragm Mostly Elastic



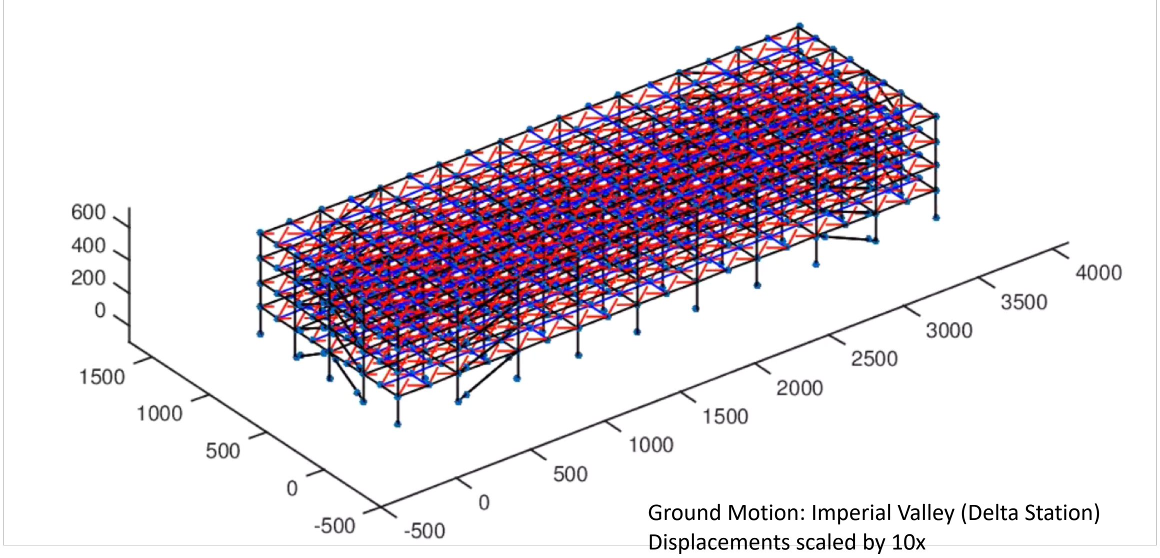
MCE Level

Diaphragm can participate in collapse

Ground Motion: Imperial Valley (Delta Station)
Displacements scaled by 10x

Nonlinear Response History Analysis of Archetype Buildings

Animated Response of Four-Story Building – DBE Level

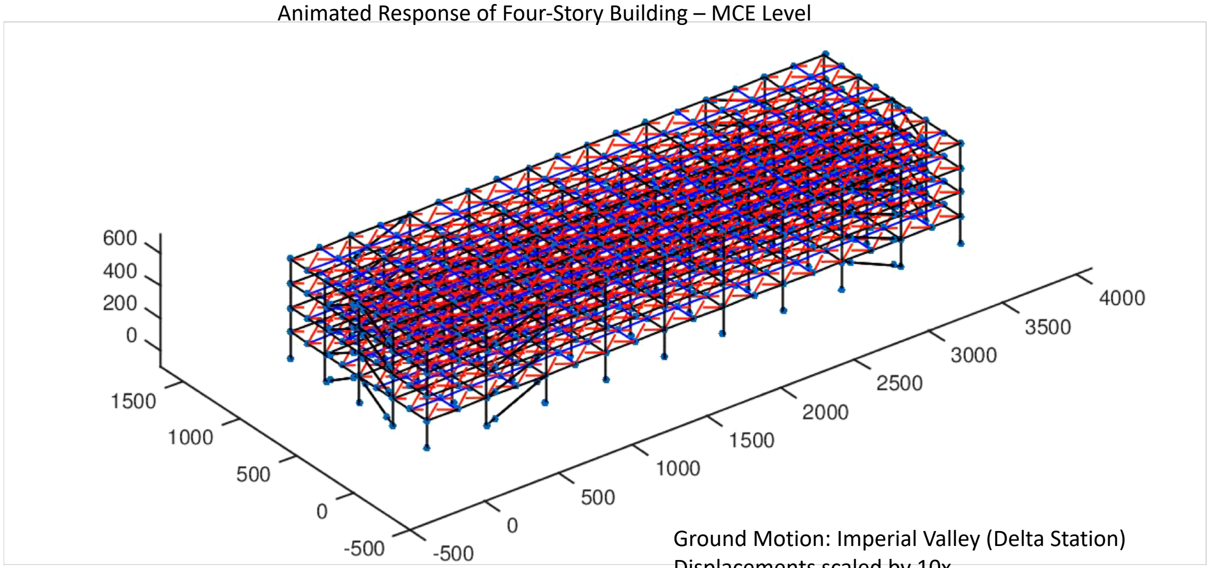


Ground Motion: Imperial Valley (Delta Station)
Displacements scaled by 10x

SDII

Nonlinear Response History Analysis of Archetype Buildings

Animated Response of Four-Story Building – MCE Level



Ground Motion: Imperial Valley (Delta Station)
Displacements scaled by 10x

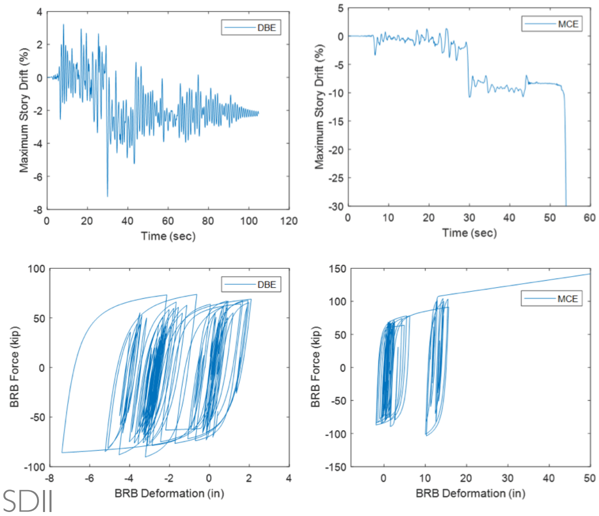
SDII

Nonlinear Response History Analysis of Archetype Buildings

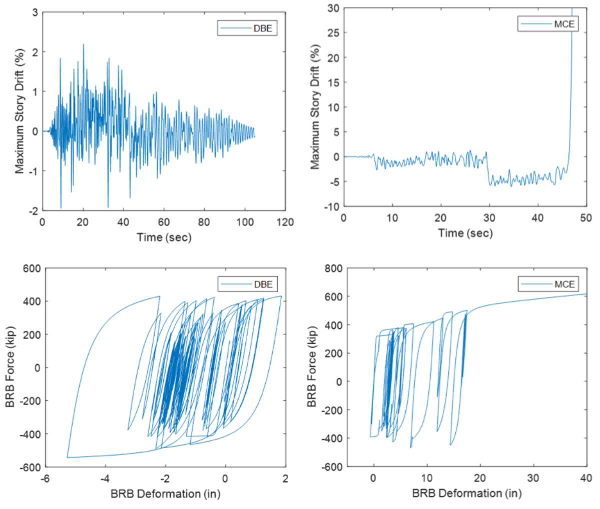
Time history and hysteretic response **examples**

Ground Motion: Imperial Valley (Delta Station)

One-story



Four-story



Collapse Criteria

Four criteria considered for defining collapse:

1. Maximum story drift ratio exceeds a specified limit (10% story drift used here).
2. BRB strain exceeds maximum from tests (3.4%).
3. Diaphragm shear angle exceeds maximum from tests (17% bare, 2.8% filled).
4. Analysis fails to converge (rare occurrence before previous limits).

Preliminary Results from Nonlinear Response History Analysis

Preliminary results of NRHA with 44 ground motion pairs at 3 scale levels

Percent of ground motions that cause collapse

One-Story Archetype Building

Story Drift Ratio Limit	Conventional	Rs=1	Rs=3
DBE	0.0%	0.0%	In progress
MCE	9.1%	6.8%	In progress
$ACMR_{10\%}$	36.4%	31.8%	In progress

Four-Story Archetype Building

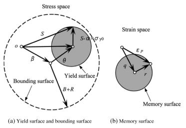
Story Drift Ratio Limit	Conventional	Rs=1	Rs=3
DBE	0.0%	0.0%	0.0%
MCE	13.6%	4.5%	13.6%
$ACMR_{10\%}$	25.0%	20.5%	29.5%

Leveraging simulation to improve understanding of behavior

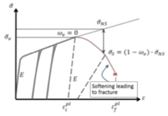
Vertical vs. Horizontal LFRS – B. Schafer
Building scale simulations – M. Eatherton
Bringing fracture into models – J. Hajjar
Optimization – B. Schafer

Cyclic Fracture Framework for Steel and Concrete

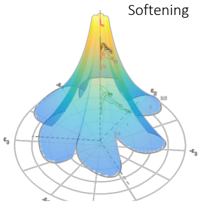
Steel Plasticity + Fracture



Three-surface plasticity



Effective Stress Softening



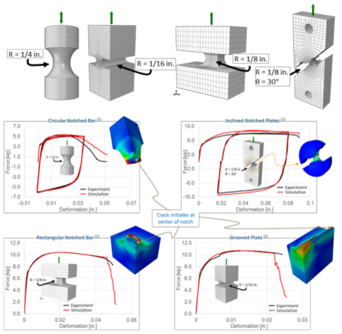
Fracture Initiation Surface

Plastic behavior is modeled using a Hybrid Approach combining a cyclic plasticity model with two damage formulations for fracture initiation and propagation that can be used in conventional High Fidelity Finite Element Models

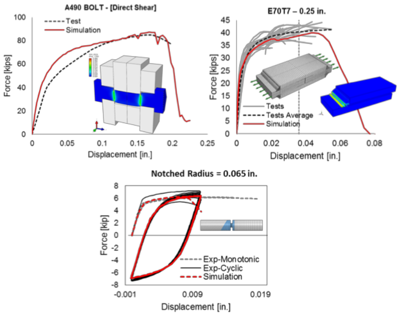
Calibration of Steel Material Parameters

Material parameters were calibrated for structural steels A572 and A992 both Grade 50; weldments E70T6 and E71T8-K6; and A490 bolts using experimental results of circular notched bars, rectangular notched bars, grooved plates, and inclined notched plates

Calibration to a Set of Material Tests



Calibration for Bolts and Weldments

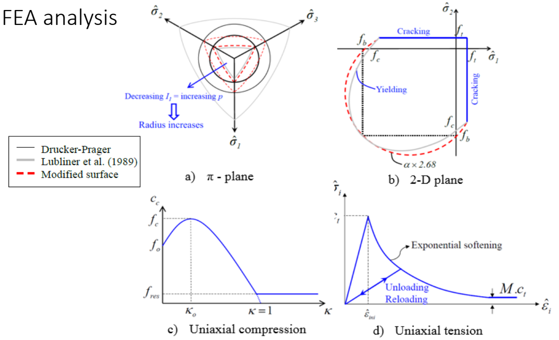


Cyclic Fracture Framework for Steel and Concrete

Triaxial Constitutive Model for Concrete

A combination of elastoplastic formulation with a non-associative flow rule that captures the compression dominated behavior, and a rotating smeared crack model that captures the tension dominated behavior developed by Mohammadreza and I. Koutromanos .

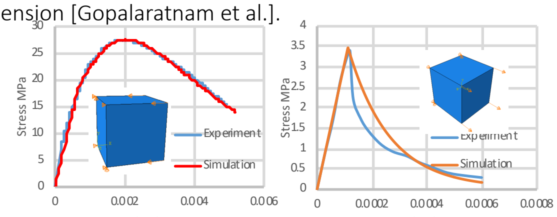
The model also uses a damage variable, κ that controls the softening of the material and subsequent element deletion in FEA analysis



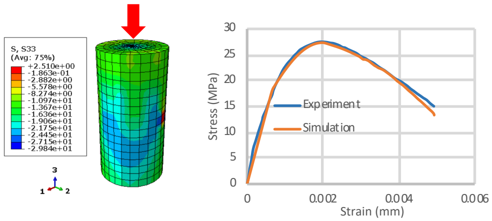
Cyclic fracture model of concrete (Mohammadreza and Koutromanos, 2016)

Calibration of Concrete Material Parameters

The material parameters of the model have been calibrated for normal weight concrete by performing single element analysis of cylinder tests in compression [Karsan et al.)] and tension [Gopalaratnam et al.].



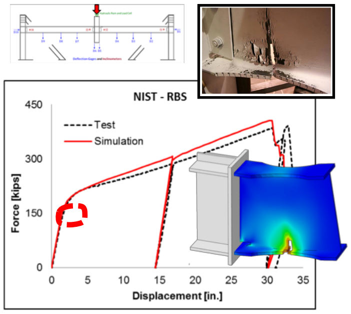
Single element analysis



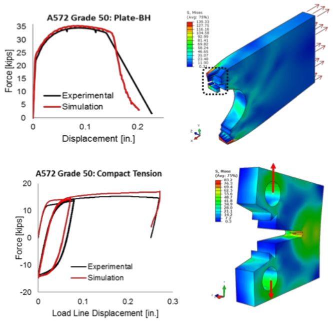
Validation against cylinder compression tests

Cyclic Fracture Framework for Steel and Concrete

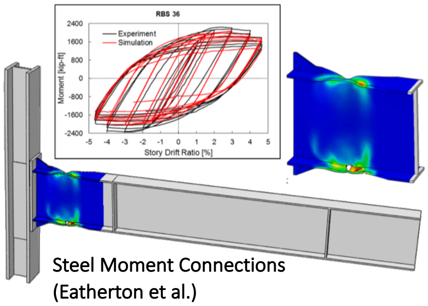
Validation Against Test on Steel Structural Elements



NIST Steel Moment Frames with Column Removal (Sadek et al.)

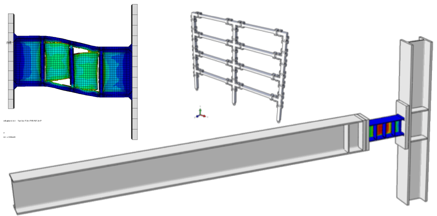


A572 material test (Kanvinde et al.)



Steel Moment Connections (Eatherton et al.)

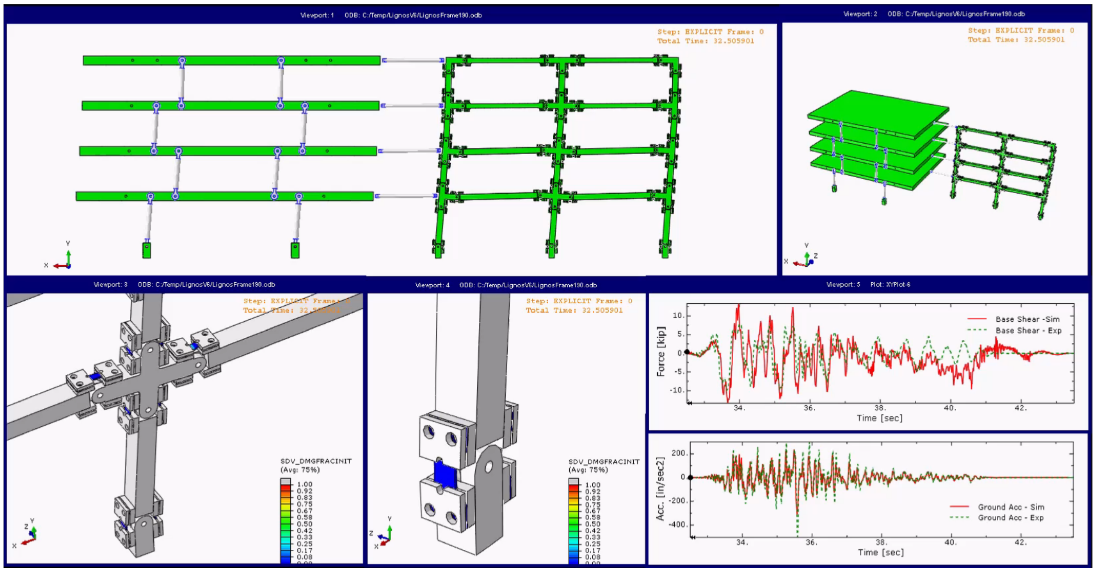
The fracture model was validated against ancillary material tests and tests on steel frames under monotonic and cyclic loading.



Shear Link in Steel Eccentric Braced Frame (Galvez P.)

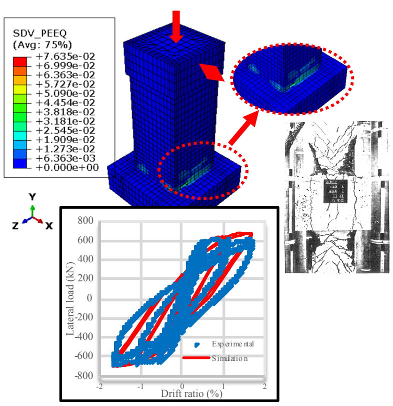
Validation Against Steel Frames (Cont'd.)

Simulation of the Lignos Frame for the Collapse Level Earthquake (i.e., 190% of the Canoga Park Earthquake)

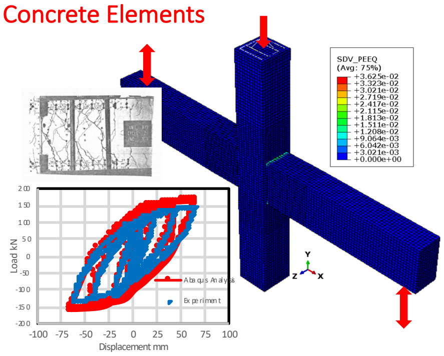


Cyclic Fracture Framework for Steel and Concrete (contd.)

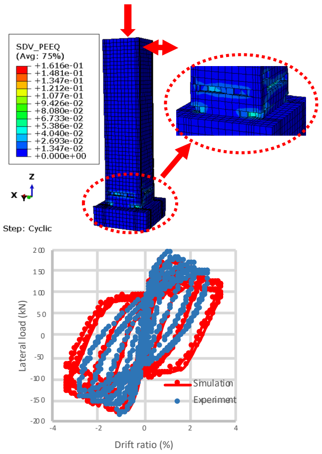
Validation Against Tests on Reinforced Concrete Elements



Reinforced concrete column (Gill 1979)



Beam Column joint (Beckingsale 1980)



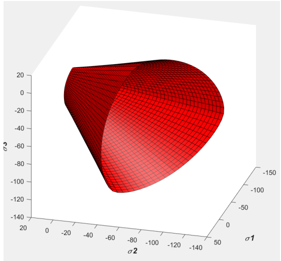
Reinforced Concrete column (Ang 1981)

The model is validated against experiments conducted on reinforced concrete columns and beam-column joint subjected to cyclic loading

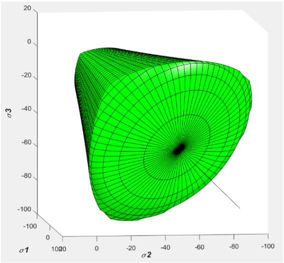
---

Advanced Concrete Model

- Modification of the existing triaxial concrete model by introducing a CAP in stress along the hydrostatic axis
- The current model is combined with the standard two-invariant CAP model developed by DiMaggio and Sandler (1971)
- The advanced model is developed to control the strength of concrete in triaxial (hydrostatic) compression stress state



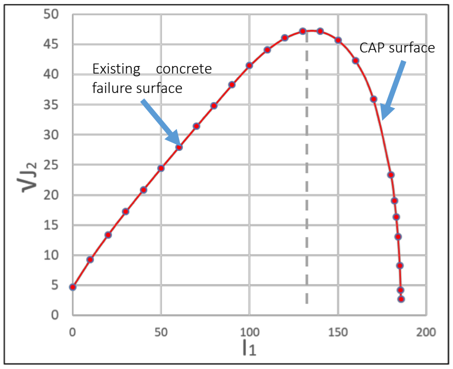
Current model without CAP



Proposed model with CAP

Failure Surface of the Proposed Model

- Existing concrete model combined with CAP model



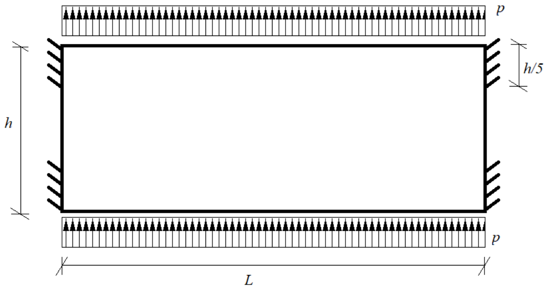
Example of the proposed model with CAP- √J2 – I1 plot

Leveraging simulation to improve understanding of behavior

Vertical vs. Horizontal LFRS – B. Schafer
Building scale simulations – M. Eatherton
Bringing fracture into models – J. Hajjar
Optimization – B. Schafer

Application of topology optimization

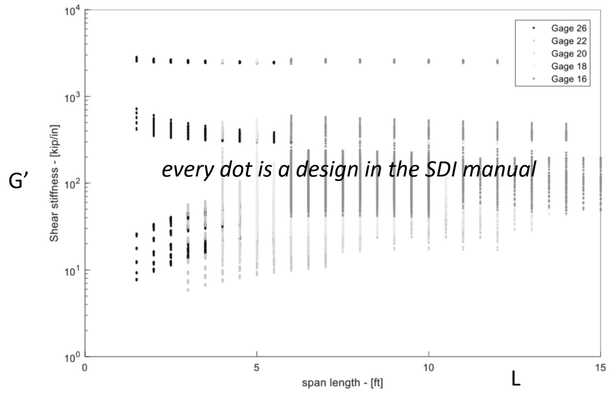
Nervi/Gotti Wool Factory – intuitive optimization



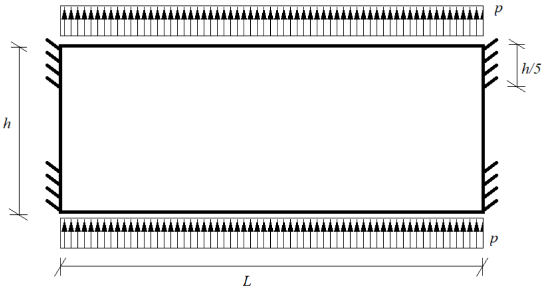
Optimal layout for floor acting as a diaphragm?

Application of topology optimization

Ground search to real properties:

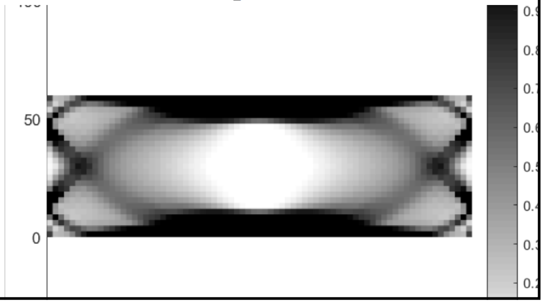


every dot is a design in the SDI manual



For fixed material, this is initial optimal layout:

Looking at gravity load constraints, material optimization, deflection criteria, more..



Schedule

8:05 – 10:00 Overview of SDII (Schafer)
Compiling and analyzing existing data (Eatherton)
New cyclic testing to characterize performance across scales
    Connector (fastener shear) (Schafer)
    Interface (pushout) (Hajjar)
    Diaphragm (cantilever) (Easterling)
Planned large scale testing (Hajjar)
Leveraging Simulation
    Vertical vs. horizontal LFRS (Schafer)
    Building scale archetype simulations (Eatherton)
    Bringing fracture into models (Hajjar)
    Optimization (Schafer)
Conclusions (Schafer)
10:00 – 10:30 Break
10:30 – 11:30 SDII Codes and Standards - Proposals and Future Pathways

Breadth of Activities from SDII plan

Innovation and Practice

- Building and Diaphragm Archetypes
- Evaluation of Existing Design Methods
- Evaluation of Existing Steel Diaphragm Technologies
    - Gap Analyses: Seismic and Non-seismic performance
- Candidate Design Methods
    - Methods proposed by others
    - Methods proposed by SDII
- Candidate Technologies
    - Revised profiles, material, manufacture, fuses…
- Seismic Standards Work

Experiments

- Existing Tests
- Test Technologies
- Connector Tests
- Interface Tests
- Diaphragm Tests
- Building Bay Tests
- Full Building Tests *
- Test Database
- Test Standards

Modeling

- Conventional Design Models
- Modeling for Experimental Program
- Diaphragm Models
- Whole Building Models
    - Reduced Order
    - OpenSees/Frame Modeling
- Next-generation Models
- Non-Structural Models
- Optimization Models

* SDII industry funding and NSF funding do not fully fund building tests, team has submitted proposals to supplement this effort and is collaborating with Fleischman et al. NSF Project

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Assertions/Conclusions

- SDII has significant activities underway to provide a path for steel diaphragms to leapfrog current conditions in understanding and design
- SDII is building out design methods, benchmark test results, and modeling methods and protocols that can broadly benefit all steel buildings and provide pathways for improving overall (seismic) building design/performance
- SDII is keen to receive feedback on the activities herein – really looking forward to the discussion this afternoon - the research space is large and the need is an important one
- Coming after break - SDII is fully engaged with standards processes to advance findings and improve/remove gaps in coverage for steel diaphragms in design…

Appendix 2: SDII Codes and Standards Slides

Schedule

10:00 – 10:22	Break
10:22 – 11:22	**SDII Codes and Standards - Proposals and Future Pathways**
	Overview (Schafer)
	This code cycle
	Bare deck (Schafer), Concrete-filled (Easterling, Eatherton)
	Future code cycles (many questions here!)
	Ideas and Observations (Eatherton)
11:22 – 11:25	Introduction to SDII Questionnaires – Challenges and Innovation
11:25 – 12:00	Individual Time to work on questionnaires
12:00 – 12:45	Lunch

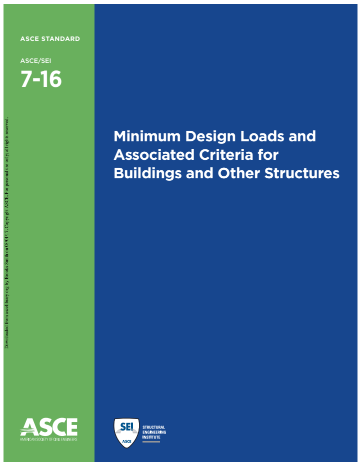
SDII Codes and Standards Work

Active Proposals, Proposals in Development, Ideas for the Future

Diaphragm Design Today - Demand

- Prediction of diaphragm demand can also involve a fair bit of guidance on diaphragm and building modeling in seismic applications, today:

- Traditional Diaphragm Design (R)
  - ASCE 7-16 12.10.1
- Alternative Diaphragm Design (R_s)
  - ASCE 7-16 12.10.3
- New RWFD* Diaphragm Design (R_{diaph})
  - FEMA P-1026
  - BSSC ballot approved for wood diaphragms


ASCE STANDARD
ASCE/SEI
7-16
Minimum Design Loads and Associated Criteria for Buildings and Other Structures
ASCE
SEI

*RWFD = Rigid Wall Flexible Diaphragm building

Diaphragm Design Today – Capacity

Relevant diaphragm design guidance does, and will in the near future continue to, exist across a wide variety of standards. AISI S310/AISI S400 are the long term planned home for capacity, performance, modeling.

ASCE 7
- General guidance
- Ch. 14 call outs

AISI
- AISI S310
- AISI S400
- AISI Test Standards

ASCE 41 (Demand here too)
- Ch. 9 = AISC 342

AISC
- AISC 341
- AISC 342 = Ch.9 ASCE 41
- AISC 360

SDII researchers serve with you across all the relevant committees for this standards development

Implementing SDII efforts into standards

- This cycle / next cycle approach
  - **This cycle (2022) is already almost complete**, second BSSC PUC ballot for NEHRP due in February, so decisions must be made with available information
  - **This cycle**, work with basic frameworks developed to date and **extend existing methods** to cover steel deck diaphragms appropriately. Provide performance-based pathways, wherever possible. Make decisions that will in the long term lead to a coherent, internally consistent, and centralized set of provisions.
  - **Next cycle**, **expand** design philosophies to reflect research, update developed provisions to reflect new findings – particularly from experimental benchmark testing, expand and support more accurate model-based predictions, more…
- Bare deck (roofs) / filled deck (floors) approach
  - The design, research, and standardization communities have only had limited overlap for bare vs. filled deck systems in the past. The behavior, although both include steel deck, is obviously quite different.
  - **This cycle**, develop **separate, but parallel**, pathways for bare and filled deck
  - **Next cycle**, work with new composite design committee at AISI and other organizations AISC, ASCE, BSSC, to bring steel deck diaphragm standards **under one "roof"**

Advancing seismic design for bare steel deck diaphragms (roofs)

Research & efforts for this code cycle – based on active work in AISC TC7, BSSC IT9

In many ways a template for what is to happen for deck with fill (floors)

Advancing seismic design for bare steel deck diaphragms (roofs)

Research & efforts for this code cycle – based on active work in AISC TC7, BSSC IT9

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

Background

Strength and stiffness from code provisions, and thoughts on ductility

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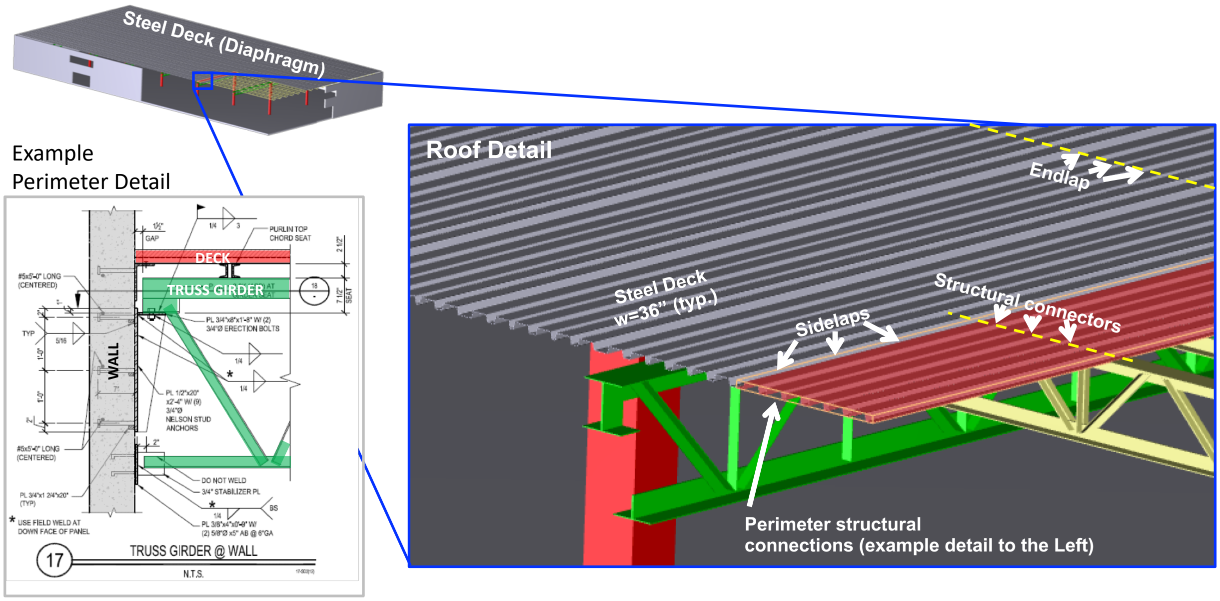
Steel deck diaphragm nomenclature and features



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

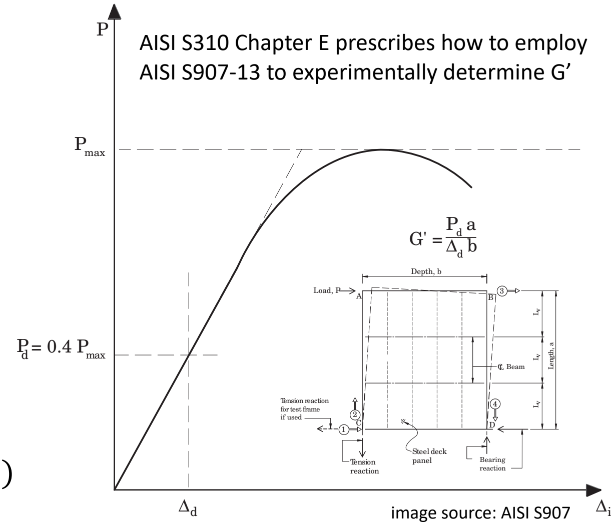
sidelap

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

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

interior — corner — 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

$P_n = P_{max}$

$S_n = P_n/b$

$\phi S_n \geq S_r$

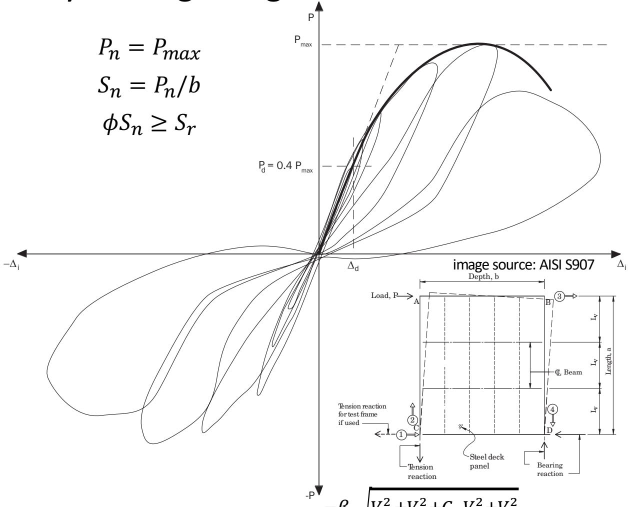


image source: AISI S907

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

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

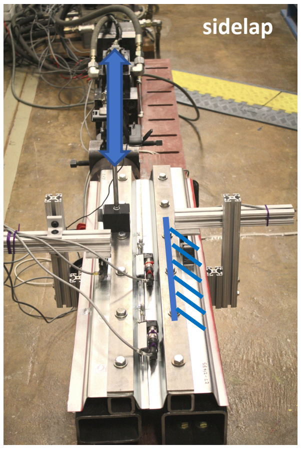
source: AISI S310 (2016) / SDI DDM04 (2015) / AISI S907 (2013)

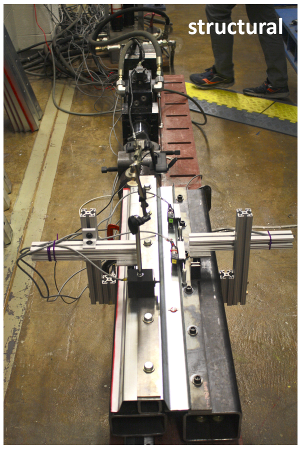
Connector Performance

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





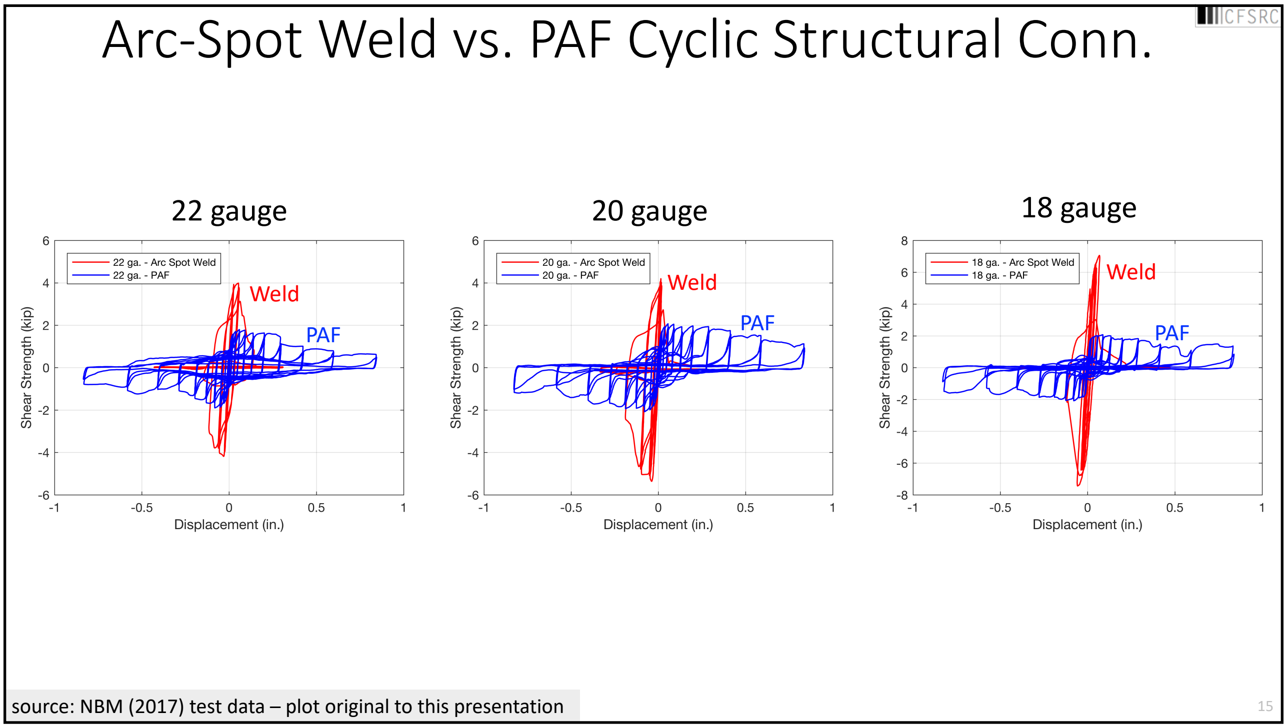
AISI S905 test standard
FEMA 461 Protocol 1 Cyclic Profile ($a_{i+1}=1.4a_i$)

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)



Arc-Spot Weld vs. PAF Cyclic Structural Conn.

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

Cantilever Deck Diaphragm Experimental Performance

Impact of fasteners and other details on ductility performance

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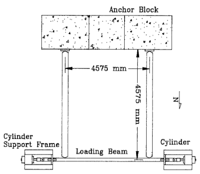
SDII Cantilever Diaphragm Test Database

Overview

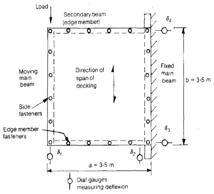
Testing Program	# of Specimens
Cornell University, 1950s-1960s	40
S. B. Barnes and Associates, 1950s -1960s	38
West Virginia University, 1960s-70s	246
Development Lab of Inland Ryserson Co.	1
University of Salford, Manchester 1970s-80s	5
ABK, a Joint Venture, California 1980s	3
Iowa State University, 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

Types of Experimental Studies Included

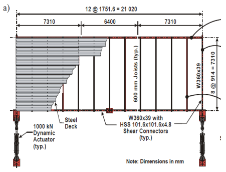


Group from Iowa State in 1980's and 1990's

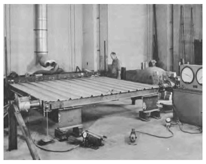
Diaphragm Tests by Industry (e.g. Hilti)



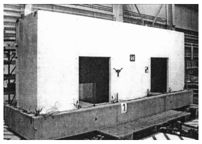
Research from Europe (e.g. Davies and Fisher 1979)



Work by Tremblay and Rogers in Canada

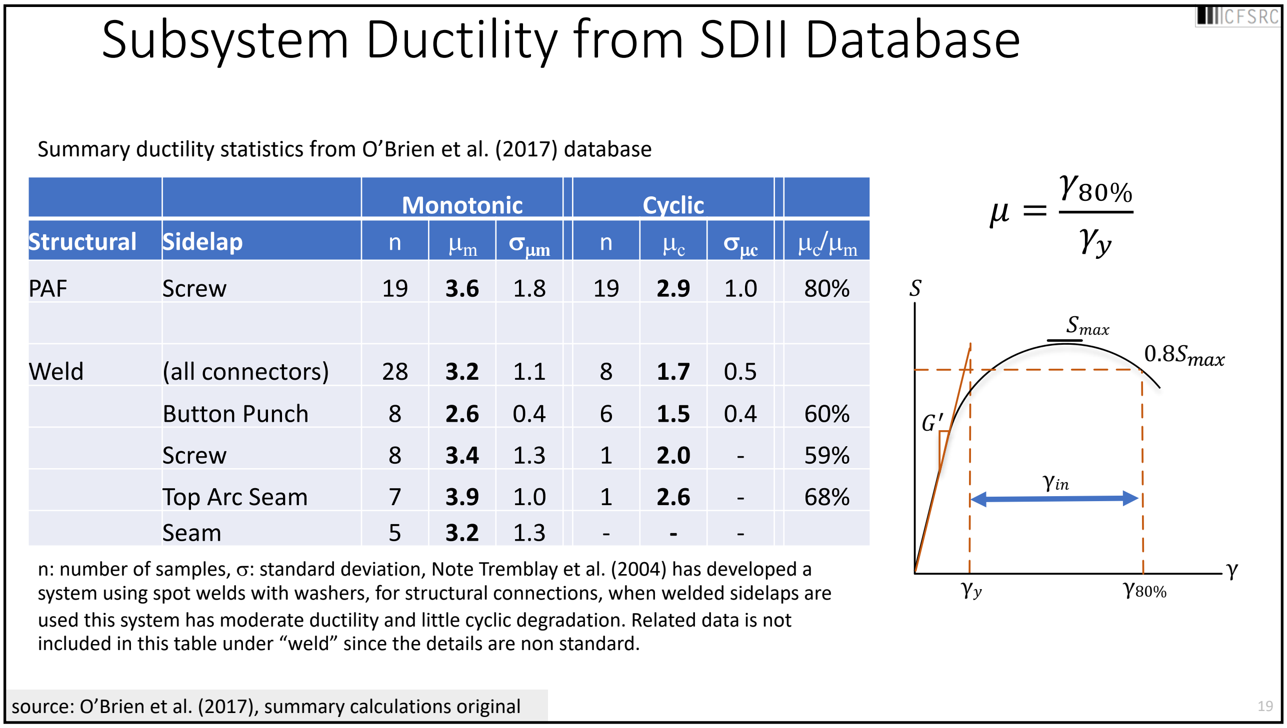


Larry Luttrell's group at West Virginia



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

source: O'Brien et al. (2017)



Subsystem Ductility from SDII Database

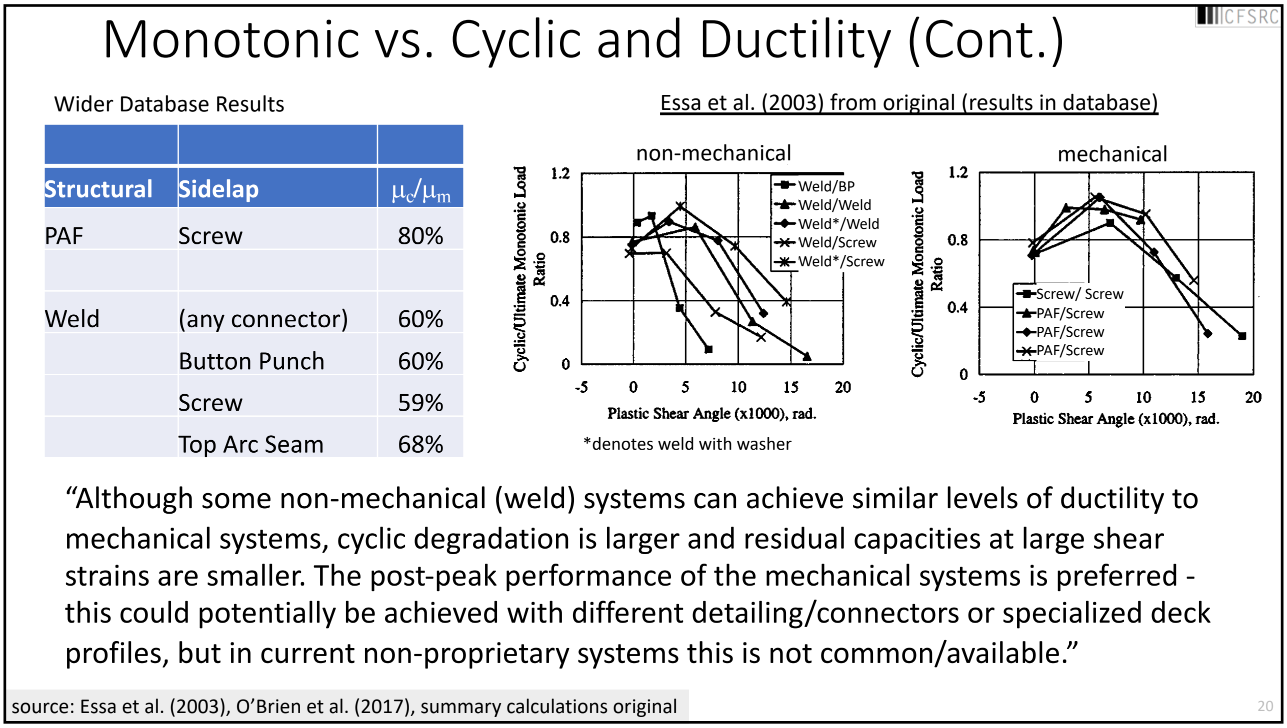
Summary ductility statistics from O’Brien et al. (2017) database

		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 (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)

*denotes weld with washer

“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

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

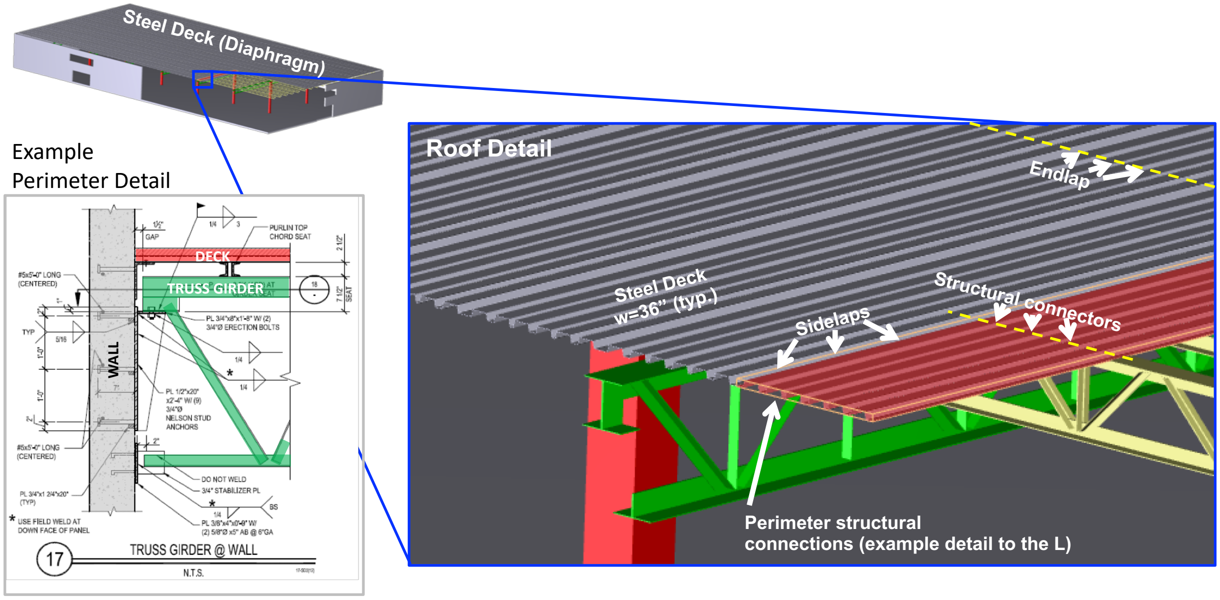
source: O'Brien et al. (2017) database

Building Applications
Steel RWFD Buildings

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

SDII

RWFD Building – Steel Deck Roof



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

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RWFD Buildings

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

FEMA P-1026/March 2015

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 and ballot passed

FEMA P-1026 simulation engine

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

FEMA P-1026/March 2015

FEMA

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

by

Maria Koliou

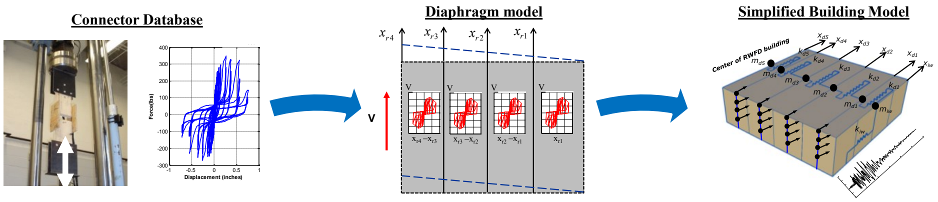
June 05, 2014

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 results for steel deck

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

FEMA P-1026/March 2015

FEMA

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

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FEMA P-1026 modeling on steel deck roofs

- predicts **conventional design** will **not** have an **acceptable** collapse margin ratio in P695 analysis, and larger buildings perform worse than smaller
- predicts a **modified design** with protected zones (higher force zones) around perimeter will have **acceptable** collapse margin ratio

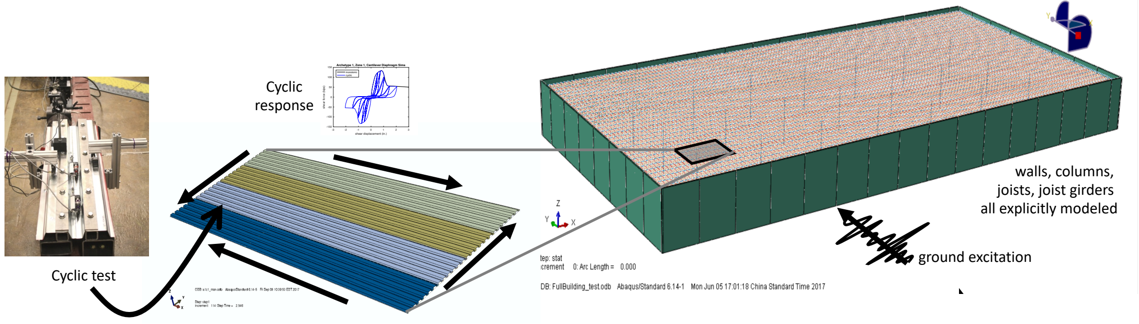
However, FEMA P-1026 concludes "At this time the alternate design procedure [with protected perimeters] is not intended to apply to RWFD buildings with steel deck diaphragms. There are several reasons...

- tests results of a large scale diaphragm showed significantly less distribution of yielding than analyses ...,
- design strengths are based on monotonic tests,
- data for reverse cyclically loaded connections is sparse ...,
- the post-yield stiffness of connectors is positive for only a small deformation, …
- few reverse cyclically loaded diaphragm tests have been performed …, &
- 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), Koliou (2014), Koliou et al. papers

New 3D simulation of RWFD steel buildings



(a) Connector tests

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

(b) 3D Roof submodel

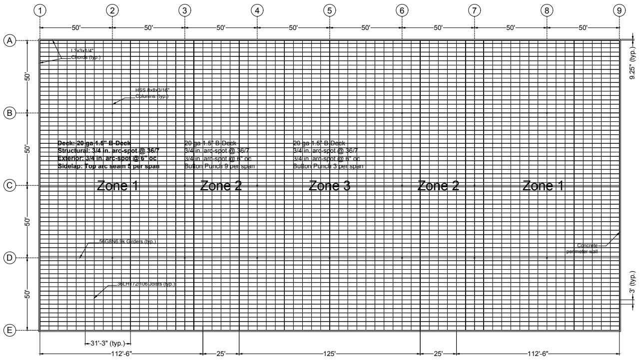
- Shell FE model, material and geometric nonlin.
- Similar to cantilever diaphragm testing
- Nonlinear connectors
- Establish cyclic performance of roof segment
- Validated against testing

(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 and CFSRC see Schafer et al. (2018) summary

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

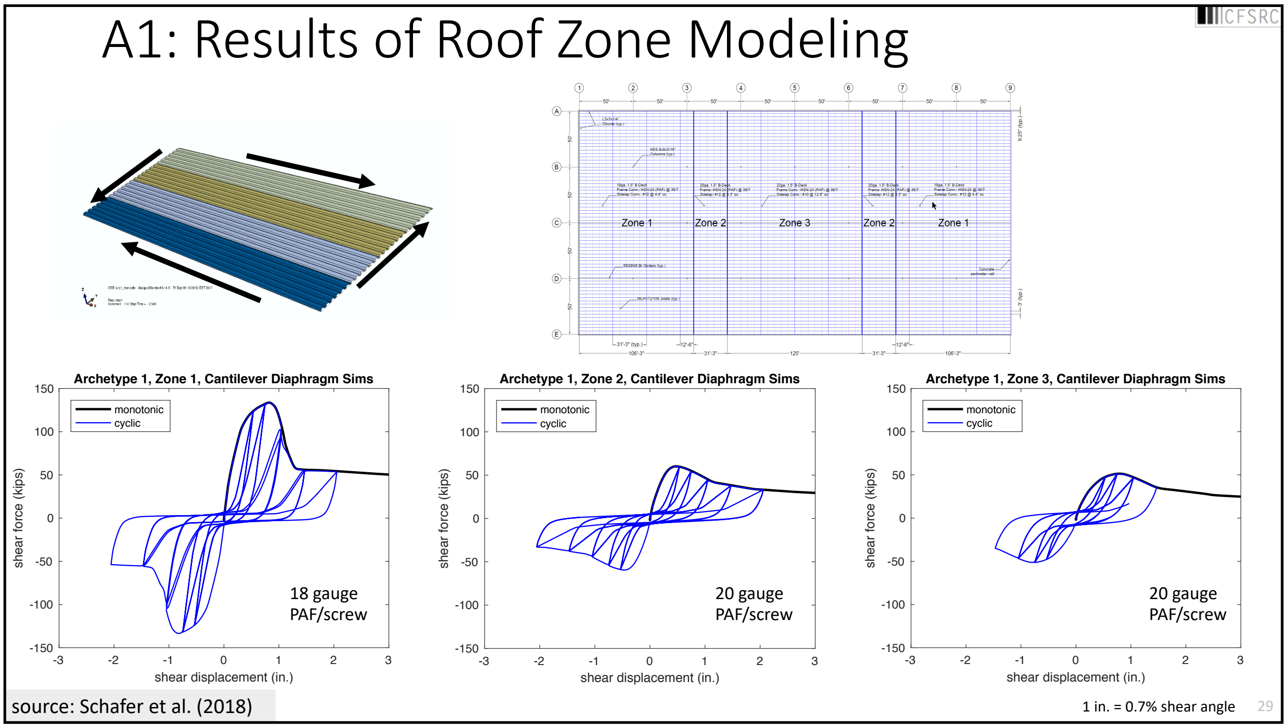


- ”Large” 200x400 building design, SDC D
- Design per AISI S310-16 and ASCE7-16
- Summary of A1 and now A3 designs to the right
- Roof designed in three zones

	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

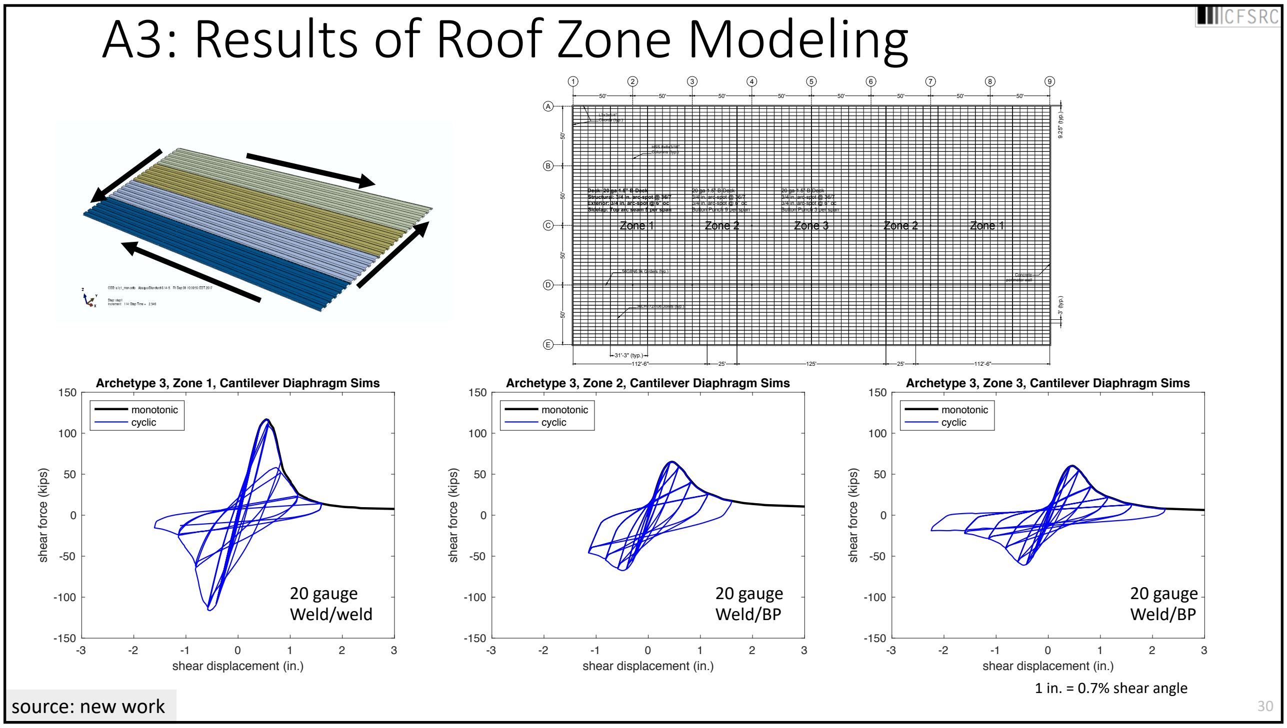


A1: Results of Roof Zone Modeling

CFSRC

source: Schafer et al. (2018)

1 in. = 0.7% shear angle



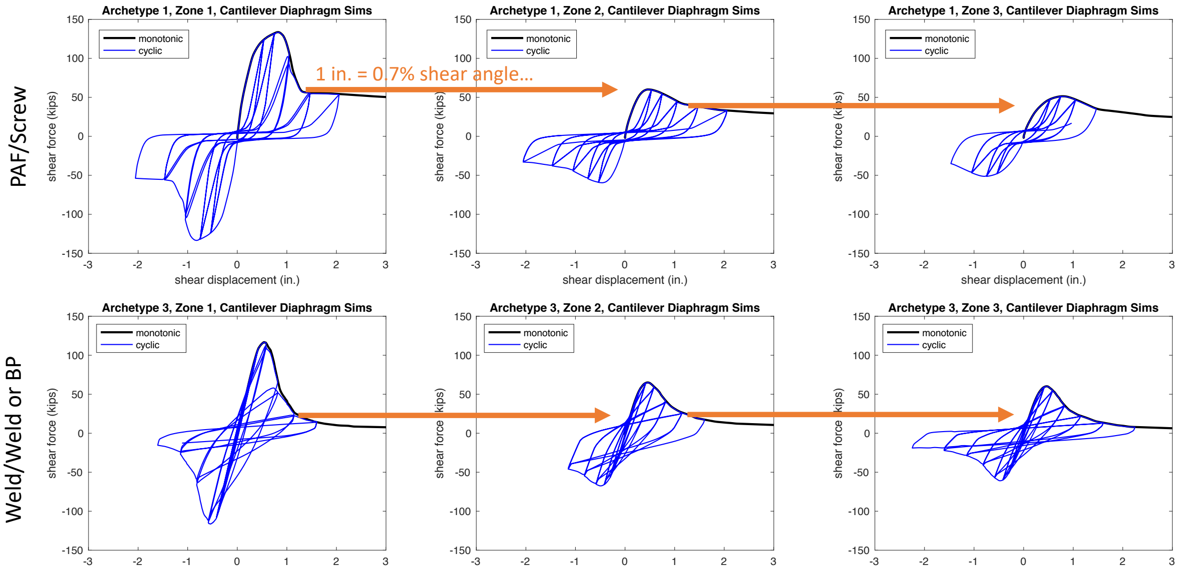
A3: Results of Roof Zone Modeling

CFSRC

1 in. = 0.7% shear angle

source: new work

Comparison of A1 and A3 roof performance



Building Simulation Details (P695 details)

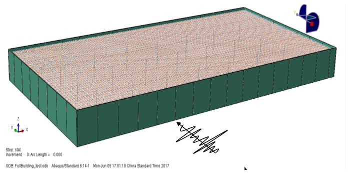
- Apply FEMA P695 11.3 Collapse Evaluation of Individual Buildings

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

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

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

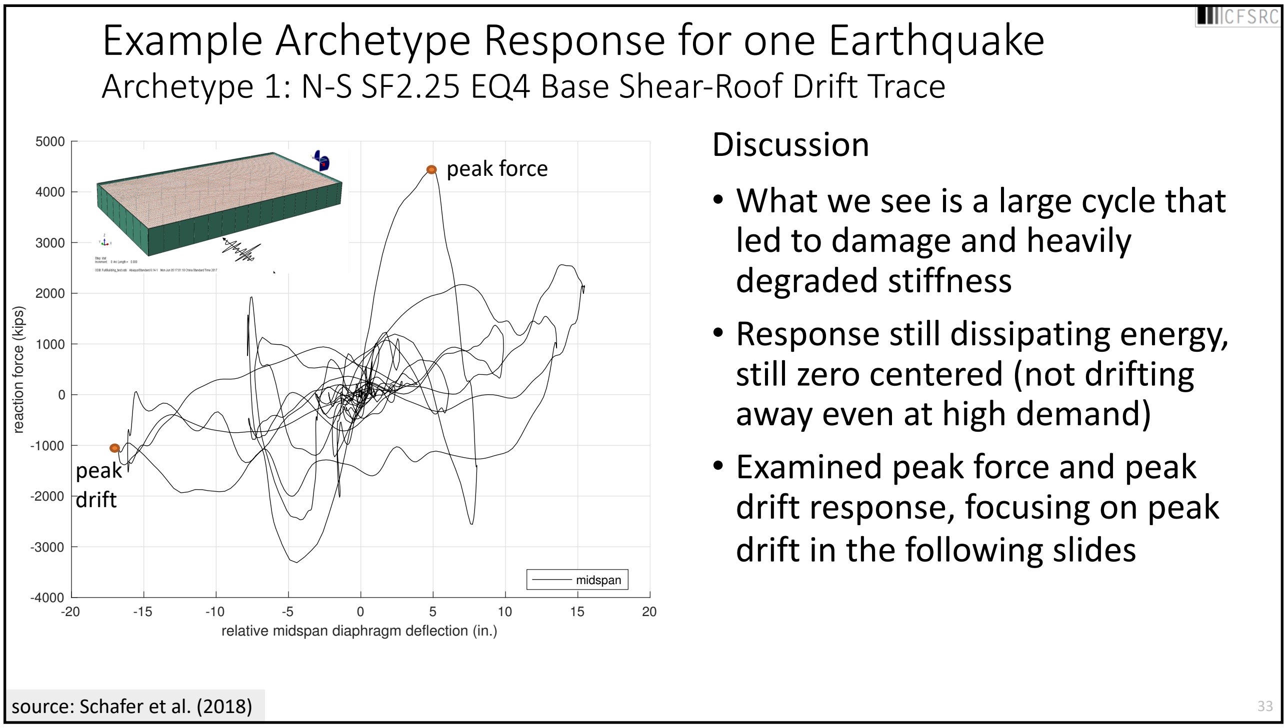
  - Run 44 P695 earthquake motions at this scale factor
  - If median is acceptable then building "passes" examination



- 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



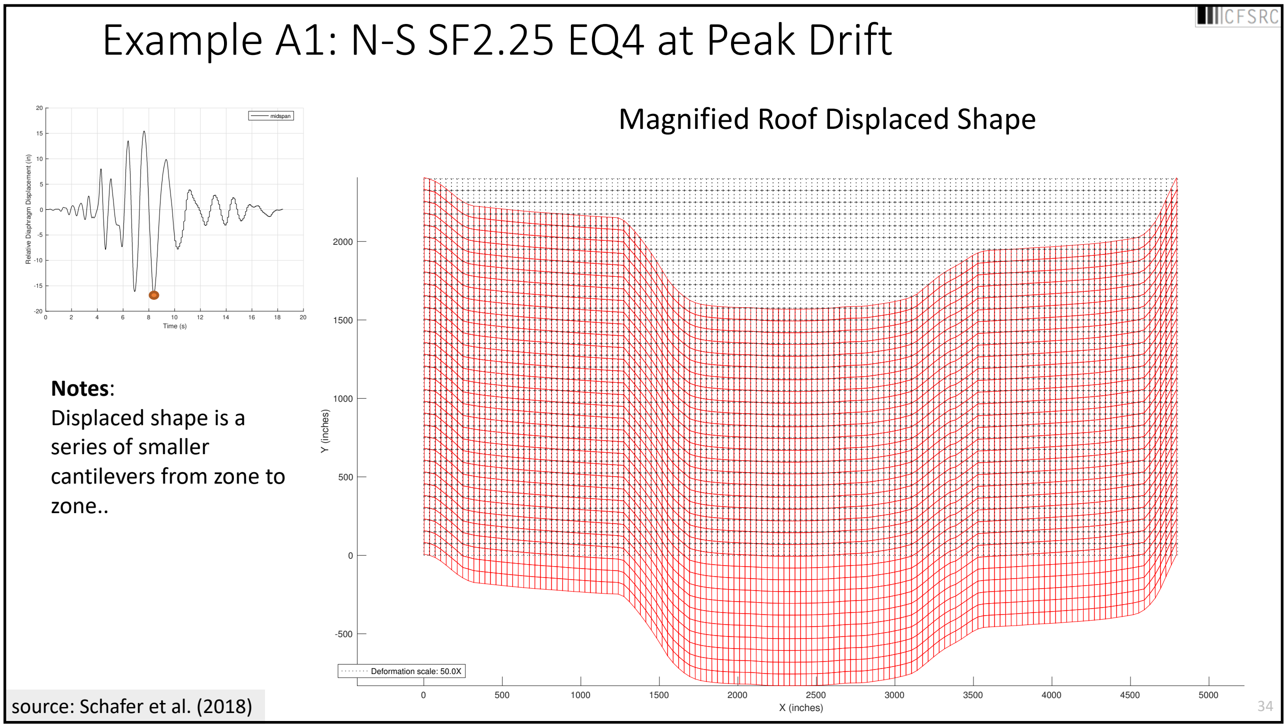
Example Archetype Response for one Earthquake

Archetype 1: 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)

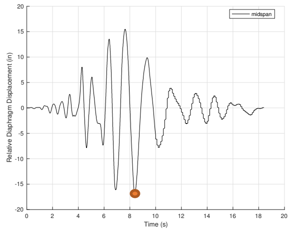


Example A1: N-S SF2.25 EQ4 at Peak Drift

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



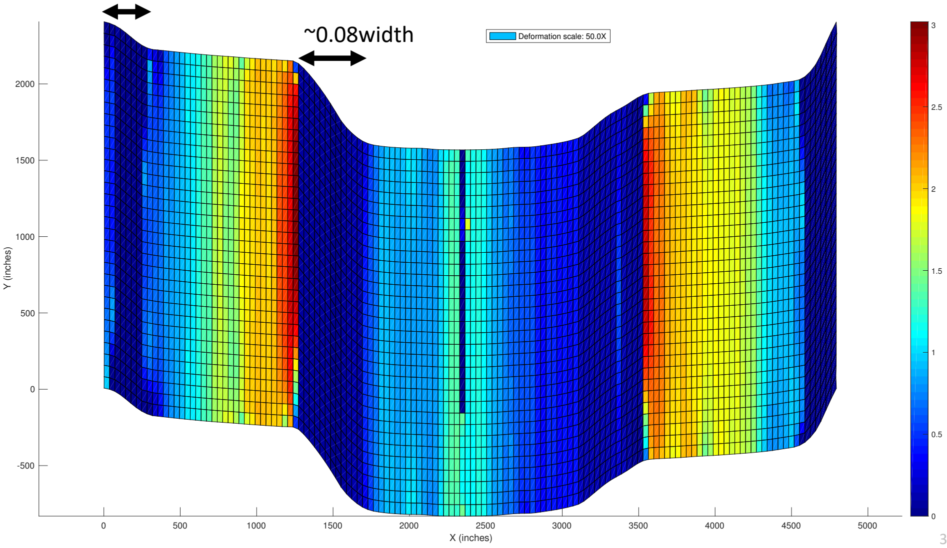
Notes:
Diaphragm edge and zone boundaries experience high shear strains. Length of "plastic" zone reduced for edge, but 2nd zone created at zone transition.
(Width ~ joist girder spans… in this case)

source: Schafer et al. (2018)

G' contour

~0.05width

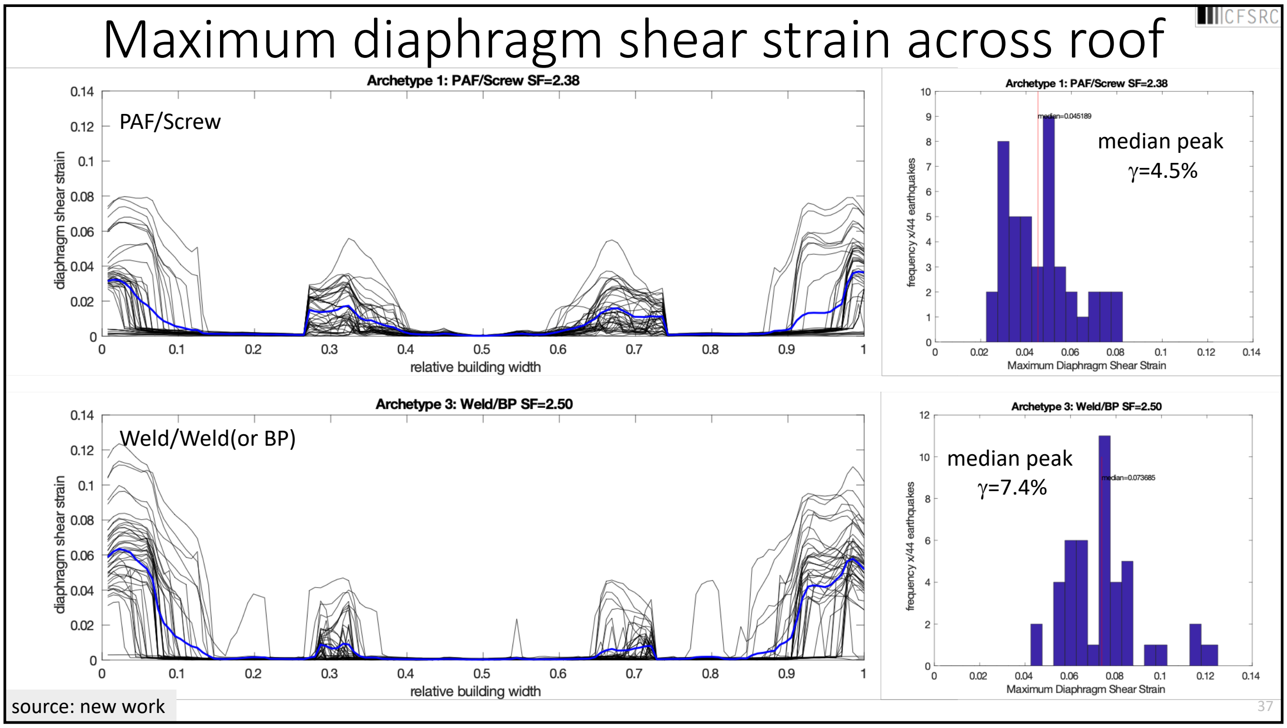
~0.08width



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$_{10\%}$

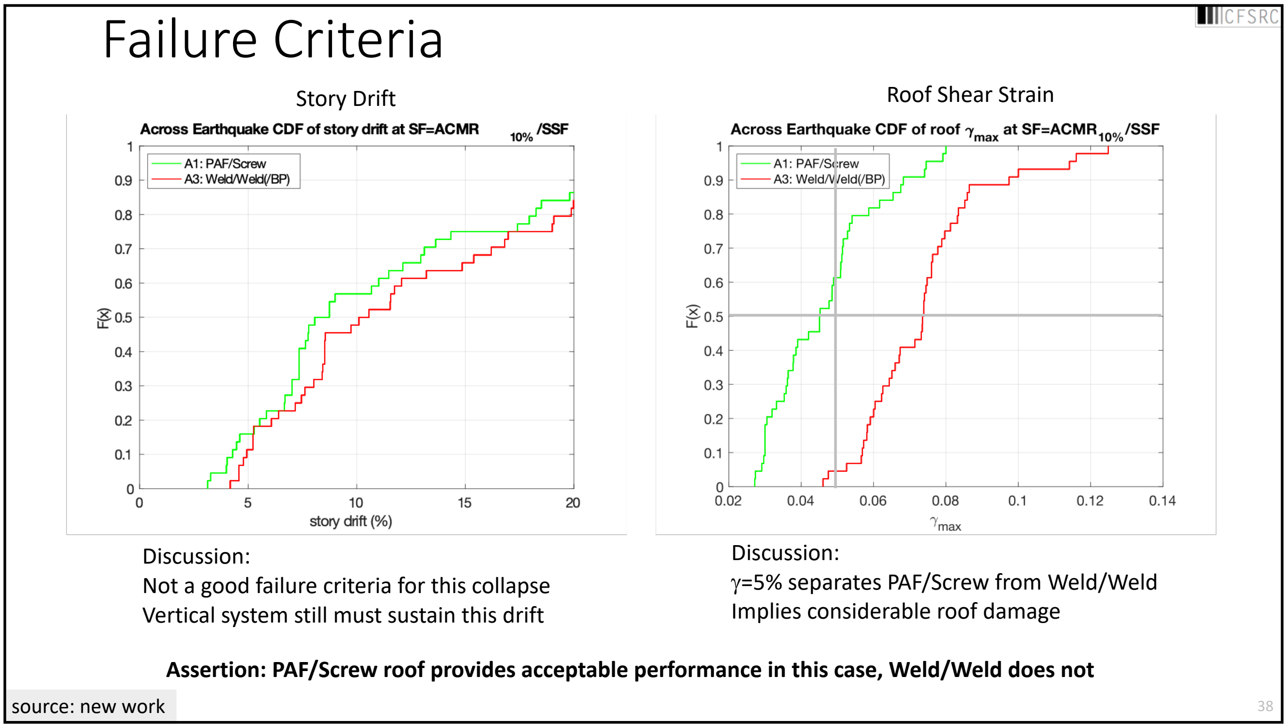


Maximum diaphragm shear strain across roof

median peak γ=4.5%

median peak γ=7.4%

source: new work



Failure Criteria

Story Drift

Roof Shear Strain

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

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

Assertion: PAF/Screw roof provides acceptable performance in this case, Weld/Weld does not

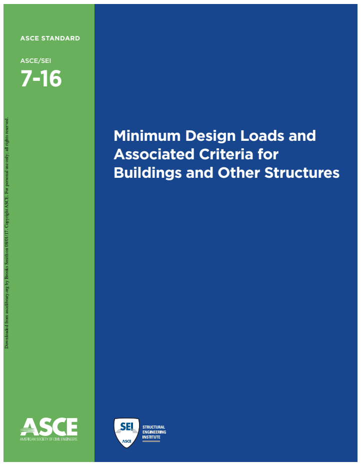
source: new work

Transitioning to design methods

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

Diaphragm Design - Demand

- For the purposes of this presentation, assuming quite a bit of familiarity with the three diaphragm demand options currently available, please feel free to ask questions regarding these methods
- 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



Basic Design Philosophy for this Cycle

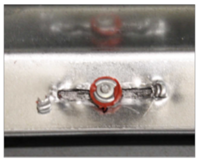
- If inelasticity and ductility is desired in the steel deck diaphragm then it should meet "special seismic detailing" requirements. These requirements should ultimately be in AISI S400.
- If a diaphragm meets "special seismic detailing" requirements then its force levels should be appropriately reduced from elastic demands, regardless of the design philosophy adopted in ASCE 7 (R, R_s, R_{diaph}).
- If seismic design does not control then conventional diaphragm design, utilizing peak capacity and initial stiffness, should not change
- If it is unclear whether inelasticity and ductility is required, but seismic performance is a concern and the diaphragm does not meet "special seismic detailing" then some form of capacity protection or elastic design should be utilized for such a case.

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 ensure 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, limit to qualified lists of 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 deck gauge
    - Spaced no less than 6 in., and no more than 12 in.

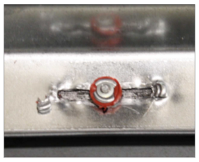


- Path 2: Performance criteria for special seismic
  - Cyclic Cantilever diaphragm test that matches PAF/Screw performance
    - $\gamma_{80\%}/\gamma_y = \mu \geq 3$, 40% residual at max($4\gamma_y$, 2%)
  - Connector testing and diaphragm simulation meeting same criteria

---

Introduce “special seismic” detail: *in progress*

- Amend AISI S400 in ASCE 7 Chapter 14
- “special seismic” detail for bare steel deck diaphragms created to ensure 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

Diaphragm Design - Demand

- For the purposes of this presentation, assuming quite a bit of familiarity with the three diaphragm demand options currently available, please feel free to ask questions regarding these methods
- Traditional Diaphragm Design (R)
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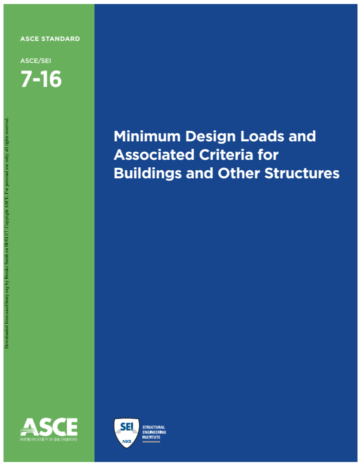


ASCE STANDARD

ASCE/SEI

7-16

Minimum Design Loads and Associated Criteria for Buildings and Other Structures

ASCE

SEI

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
  - Engineering judgment ”If ductility desired by EOR…”
  - **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 (currently proposed trigger)

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, i.e. $R_s>1$)
  - Ordinary (non-ductile or unknown, i.e. $R_s=1$)
- 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

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	long T	medium 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

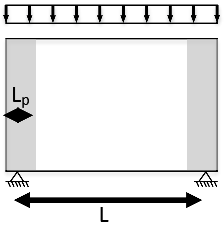
$$\mu_{system} = 1 + 4(\mu_{sub} - 1)\left(\frac{L_p}{L}\right)$$

Long Period

$$R_\mu = \mu_{system}$$

Medium Period

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



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[a]
  - Near the edge, create a zone that has 50% higher demands

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

FEMA P-1026/March 2015

FEMA

nehrp

source: FEMA P-1026

[a] Studies supporting FEMA P-1026 for steel used R_{diaph}=4.5 (R_{diaph}=2.25 around edge)

Addressing FEMA P-1026 concerns about extensions to steel deck

concerns	*resolution*
1. tests results of a large scale diaphragm showed significantly less distribution of yielding than analyses ...,	1. Created 3D model to more fully explore large scale diaphragms, identified conditions where ductility is lost and separated
2. … design strengths are based on monotonic tests,	2. Examined test-to-predicted strength for cyclic results
3. data for reverse cyclically loaded connections is sparse …,	3. Increased the cyclic test database substantially
4. the post-yield stiffness of connectors is positive for only a small deformation, …	4. Identified connectors with best ductility and integrated real behavior into model
5. few reverse cyclically loaded diaphragm tests have been performed …, and	5. Compiled available testing and utilized data to inform modeling and design results
6. many diaphragms in high seismic regions are designed using proprietary sidelaps for which no test data was available	6. Creating a performance pathway for proprietary systems to be included

In Process Ballots for Bare Steel Deck Diaphragms

- 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 (R_{diaph})
  - Special R_{diaph} = 4.5 interior (2.25 perimeter)
  - Ordinary R_{diaph} = 1.5 interior (1.00 perimeter)
  - Follow same procedure as adopted for wood
- AISC 342 update – not discussed here

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.
- 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.
- Torabian, S., Folk, H., Schafer B.W. (2018b) "Effect of connections details on the cyclic behavior of nestable screw sidelaps." Proceedings of the Int'l Spec. Conf. on Cold-Formed Steel, St. Louis, MO, 7-8 Nov. 2018.
- Torabian, S., Fratamico, D., Shannahan, K. , Schafer B.W. (2018) "Cyclic Performance and Behavior Characterization of Steel Deck Sidelap and Framing Connections." Proceedings of the Int'l Spec. Conf. on Cold-Formed Steel, St. Louis, MO, 7-8 Nov. 2018.
- NBM (2017) "Cyclic performance and characterization of steel deck connections." NBM Technologies report to AISI/SDI/SJI, 14 December 2017
- NBM (2018) "Button Punch Sidelaps (Cyclic testing program)" NUCOR-Verco/Vulcraft Group, Project number 103-042-18, 11 July 2018 (released by Verco for public use)
- Schafer, B.W., Smith, B.H., Torabian, S., Meimand, V., Eatherton, M.R. (2018). "Modeling and Performance of Thin-walled Steel Deck in Roof Diaphragms under Seismic Demands." Proc. of the Int'l. Conf. on Thin-walled Structures. Lisbon, Portugal.
- Verco (2018) private communication for perimeter details of RWFD building
- Essa H.S., Tremblay, R., Rogers, C.A. (2003). "Behavior of Roof Deck Diaphragms under Quasistatic Cyclic Loading." J. Struct. Eng., 2003, 129(12): 1658-1666.
- Rogers, C.A., Tremblay, R. (2003a). "Inelastic Seismic Response of Side Lap Fasteners for Steel Roof Deck Diaphragms." J. Struct Eng., 129(12), 1637-1646.
- Rogers, C.A., Tremblay, R. (2003a). "Inelastic Seismic Response of Frame Fasteners for Steel Roof Deck Diaphragms." J. Struct Eng., 129(12), 1647-1657.

Changes this Code Cycle for Concrete-filled diaphragms

Strength equation in AISI, Supporting R_s, Horizontal truss diaphragms

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Composite Diaphragms

- Load path
- Iowa State research project
- Current nominal strength (AISI S310; SDI DDM04)
- Comparison of two approaches

Composite Diaphragms – Load Path

- Load transfer is "from" composite slab
- Thru concrete to deck interface near the edge of slab
- Thru edge connectors
- Into framing members

Composite Diaphragms – Load Path

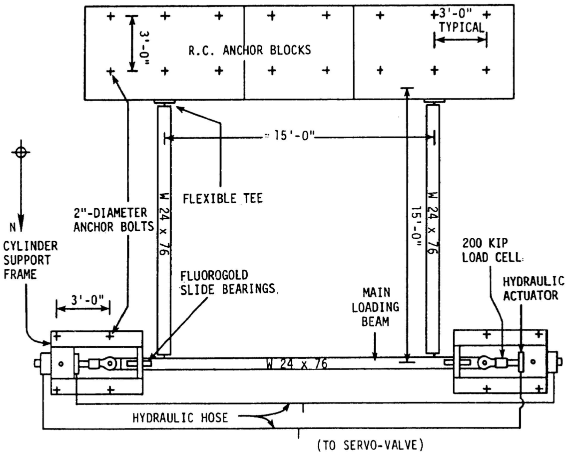
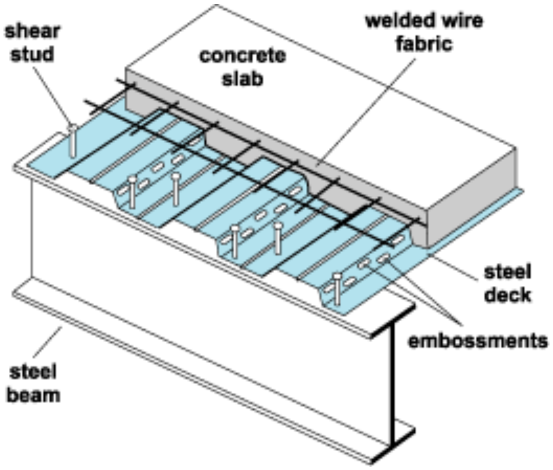


Figure 1. Diaphragm test frame schematic (Porter and Greimann, 1980)



Composite Diaphragms – ISU Project

- 32 composite diaphragm tests
  - 15 ft depth, 15/12 ft length, single span, cantilever configuration
  - No secondary reinforcing
  - Various edge connector details (welds, studs, combination)
  - Cyclic load program
- Analytical development
  - Finite element analysis
  - Hand calculation for three limit states

Composite Diaphragms – ISU Project

- Hand Calculation procedure
  - Three limit states
    - Diagonal tension of composite slab
    - Shear transfer mechanism
    - Edge connectors
  - Recommendation “today”
    - Diagonal tension of composite slab
    - Edge connectors

Composite Diaphragms – ISU Project

$$V_n = 0.0032\sqrt{f_c'}\,t_e b$$

V_n = nominal strength, k/ft

f'_c = concrete compressive strength, psi

$t_e = t_c + n_s t_s$ (d/s)

t_c = average concrete thickness

b = unit length (12 in.)

Composite Diaphragms – AISI S310

$$S_n = \beta P_{nf} / L + kbd_c\sqrt{f_c'}$$

where for US units $k = \dfrac{w^{1.5}}{585\,(10^3)}$

- Ignoring fastener component and using 145 pcf, the equation reduces to

$$S_n = 0.003\sqrt{f_c'}\,d_c b$$

S_n = nominal strength, k/ft

d_c = cover depth

Composite Diaphragms – Comparison

$$V_n = 0.0032\sqrt{f_c^{'}}\,t_e b$$ ISU

$$S_n = 0.003\sqrt{f_c^{'}}\,d_c b$$ AISI (SDI)

- Subset of 32 diaphragm tests that exhibited a diagonal tension limit state (15 diaphragms)
- Ratio of test to calculated
  - ISU – mean 1.11, coefficient of variation 0.115
  - AISI/SDI – mean 1.25, coefficient of variation 0.171
    (fastener contribution of calculated strength ranges from 19-50%)
  - AISI/SDI concrete term only – mean 1.97

Recommendation

- Use ISU calculation for diagonal tension nominal strength, including the transformed, average concrete thickness
- Replace “k” with a 0.75 reduction factor when using *lightweight concrete*

$$V_n = 0.0032\sqrt{f_c^{'}}\,t_e b$$

- If the approach is approved, ϕ and Ω to be determined, and a ballot will be prepared

AISI S310 Ballot

Structural Fill

Strength - proposed

$$S_n = k_c \lambda_{LW} b t_e \sqrt{f'_c}$$

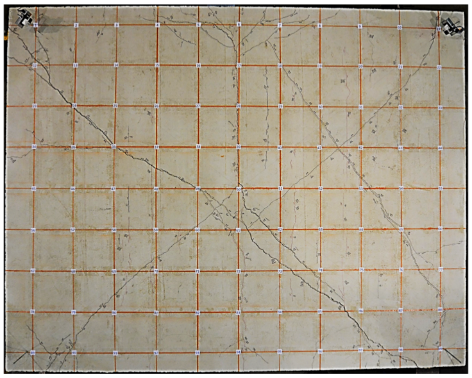
Stiffness - current

$$G' = \frac{Et}{2(1+\mu)\frac{s}{d} + C} + K_3$$

K_3 = *Stiffness* contribution of the *structural concrete* fill

= $3.5 d_c (f'_c)^{0.7}$, kip/in. for U.S. customary units

Composite Slab Diaphragms - AISC



Current AISC 341 Section D1.5
<u>"Composite Slab Diaphragms"</u>

- See ACI for shear strength or get from tests
- Only use concrete above the flutes

<u>Coming ballot change:</u>

- Reference to AISI for shear strength
- Support Alternative Diaphragm Design Procedure (Rs)
  - Just like any ductile vertical LFRS, need detailing requirements

Composite Slab Diaphragms - AISC

Changes being proposed

1. Reference AISI S310 for diaphragm strength
2. Special seismic detailing requirements
    a) Required when Rs>1 or R>3
    b) Match what has ductility in tests
    c) Deck, concrete, and studs conform to AISC 360 I3.2c.1
    d) Stud minimum Fu and minimum spacing
    e) Still in progress based on testing and modeling…

Similar Approach as Bare Deck

Horizontal Truss Diaphragms - AISC

Current AISC 341 Section B5.2

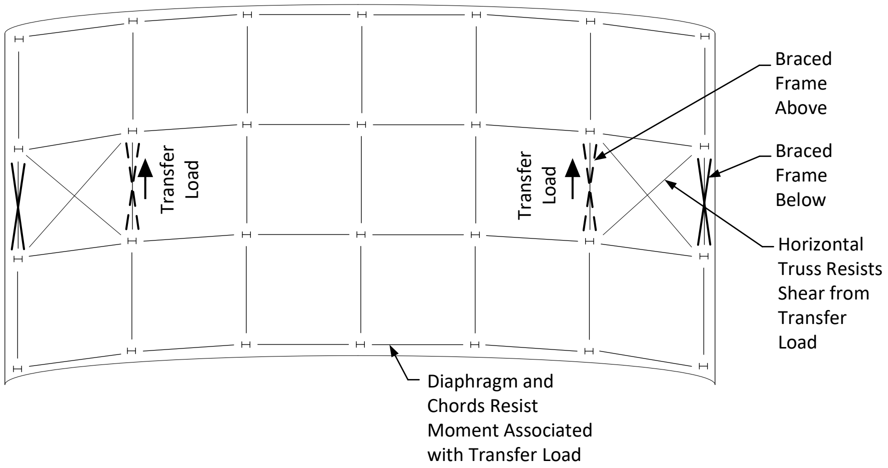
Three options for design of horizontal truss diaphragms

1. Design the horizontal truss and connections for overstrength

2. Do not need to design for overstrength if the horizontal truss follows some SCBF requirements.

3. Do not need to design for overstrength if designed as "3D System", the vertical system is OMF or OCBF, and horizontal truss satisfies some OCBF requirements.

Horizontal Truss Diaphragms - AISC



<u>Issues:</u>

1. Transfer creates shear and moment in rest of diaphragm – use overstrength
2. Horizontal SCBF should not be used with systems that have higher R than SCBF
3. More guidance need in commentary about how to design “3D System”

Horizontal Truss Diaphragms - AISC

<u>Changes being proposed</u>

1. Effects of transfer including chords, collectors, and shear in rest of diaphragm design for overstrength
2. Define overstrength loads as being capacity limited
3. Horizontal SCBF not allowed for high ductility systems (unless designed for overstrength)
4. More guidance in commentary
   a) Issues with transfer
   b) Multi-bay horizontal truss will likely act like multi-tiered braced frame – deformation concentration
   c) Design of 3D structural systems

Potential Future Code Needs

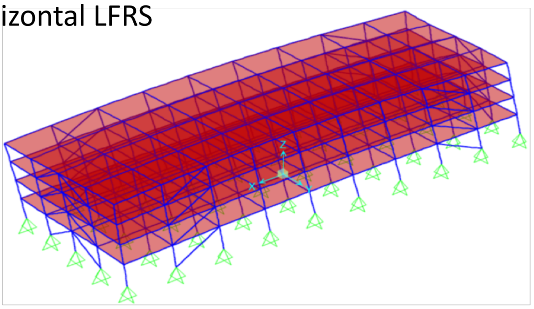
Opinions / Questions on where things might be going

Evaluating building performance

Evaluating building behavior considering diaphragm inelasticity

Two Approaches:

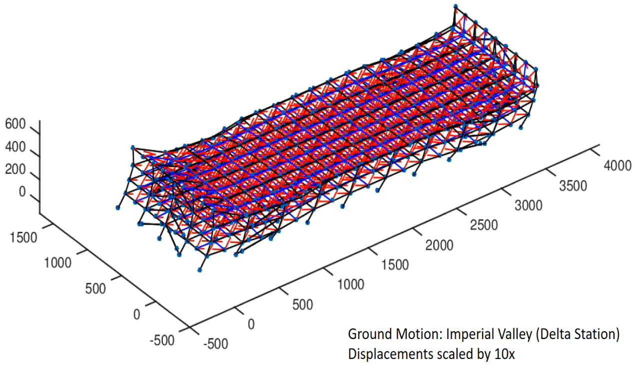
1. Define R factor for combined vertical and horizontal LFRS
    a) Unique combinations of vert. LFRS and hor. LFRS
    b) Will lead to more efficient systems
2. Independent analysis of vertical LFRS and horizontal LFRS
    a) Analyze each assuming other is elastic (e.g. don't consider benefit of inelasticity in vert. LFRS in design of hor. LFRS)
    b) Most cases will be quite (overly?) conservative
    c) Some cases may be unconservative?



Methods for Evaluating R_s (FEMA P695?)

Challenges Associated with Using FEMA P695:

1. Archetype buildings have effect of vert. LFRS and hor. LFRS. Solutions not clear:
    - a) Need many archetypes with wide range of vert. LFRS to evaluate the diaphragm?
    - b) Rs that is keyed to both the diaphragm type and vert. LFRS?
    - c) Model vert. LFRS as elastic when evaluating diaphragm?
2. Acceptability criteria the same when considering inelastic diaphragm?
    - a) P695 typically applied to 2d frames or 3d frames with rigid/semi-rigid diaphragms
    - b) Do the same acceptance criteria apply for models inelastic in vert and hor?
3. Idea of nonlinear RHA on diaphragms alone?



Ground Motion: Imperial Valley (Delta Station)
Displacements scaled by 10x

Example of analysis where BRB frame and diaphragm are both inelastic leading to collapse

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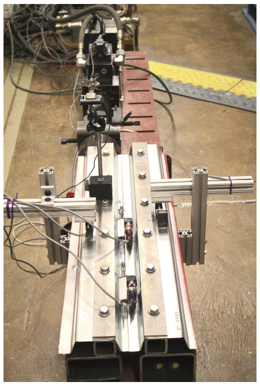
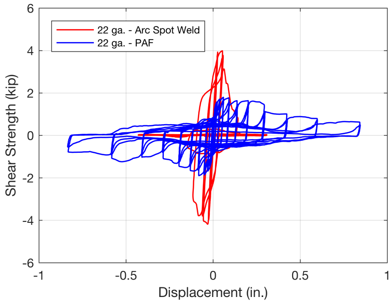
Testing standards

Need for testing standards that address ductility e.g. performance criteria for “special seismic”

- Diaphragm testing and/or connector testing
- Test setup requirements? Loading protocol?
- Procedure for calculating diaphragm ductility residual strength
- Performance based acceptability criteria





From Torabian et al. (2018) / NBM (2017)

Support 3d building design / behavior

Instead of designing horizontal and vertical systems separately - Design 3d structure

Potential advantages:

1. Get diaphragm forces from 3d models
2. More accurate view of behavior
3. Accommodate more complex structures with transfers

Other Issues:

1. How to design around openings / reentrant corners. Do we need to consider nonuniform shear stresses? Unzipping?
2. Diaphragms with structural fuses

From https://www.safdiearchitects.com/

Questionnaire

Explanation of workshop questionnaire and afternoon activities

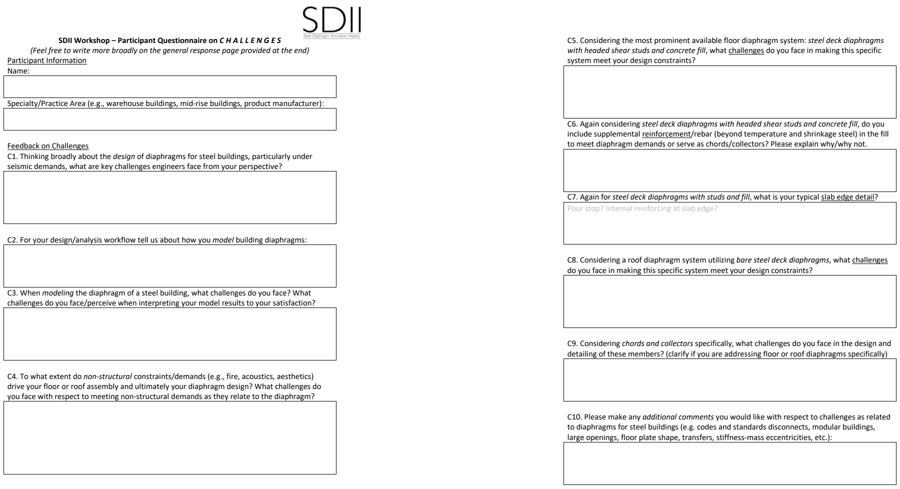
Schedule

11:30 – 11:35	**Introduction to SDII Questionnaires – Challenges and Innovation**
11:35 – 12:00	Individual Time to work on questionnaires
12:00 – 12:45	Lunch
12:45 – 1:15	Facilitated small group work, posting of key points (All)
1:15 – 1:30	Designers Perspective on Challenges (Sabelli)
1:30 – 2:10	Discussion and consensus on challenges (Sabelli + Eatherton)
2:10 – 2:25	Designers Perspective on Innovation (Sabelli)
2:25 – 2:55	Discussion and consensus on innovation (Sabelli + Hajjar)
2:55 – 3:00	Wrap-up and next steps (Schafer)

Questionnaire

- Three parts
    - Challenges
    - Innovation
    - Everything Else
- Objectives
    - Seed discussion for the afternoon
    - Identify key challenges/gaps and opportunities
    - Maximize impact of the SDII research effort
    - Provide more permanent feedback to the SDII team

Challenges



SDII Workshop – Participant Questionnaire on *CHALLENGES*

(Feel free to write more broadly on the general response page provided at the end)

Participant Information

Name:

Specialty/Practice Area (e.g., warehouse buildings, mid-rise buildings, product manufacturer):

Feedback on Challenges

C1. Thinking broadly about the *design* of diaphragms for steel buildings, particularly under seismic demands, what are key challenges engineers face from your perspective?

C2. For your design/analysis workflow tell us about how you *model* building diaphragms:

C3. When *modeling* the diaphragm of a steel building, what challenges do you face? What challenges do you face/perceive when interpreting your model results to your satisfaction?

C4. To what extent do *non-structural* constraints/demands (e.g., fire, acoustics, aesthetics) drive your floor or roof assembly and ultimately your diaphragm design? What challenges do you face with respect to meeting non-structural demands as they relate to the diaphragm?

C5. Considering the most prominent available floor diaphragm system: *steel deck diaphragms with headed shear studs and concrete fill*, what challenges do you face in making this specific system meet your design constraints?

C6. Again considering *steel deck diaphragms with headed shear studs and concrete fill*, do you include supplemental reinforcement/rebar (beyond temperature and shrinkage steel) in the fill to meet diaphragm demands or serve as chords/collectors? Please explain why/why not.

C7. Again for *steel deck diaphragms with studs and fill*, what is your typical slab edge detail?

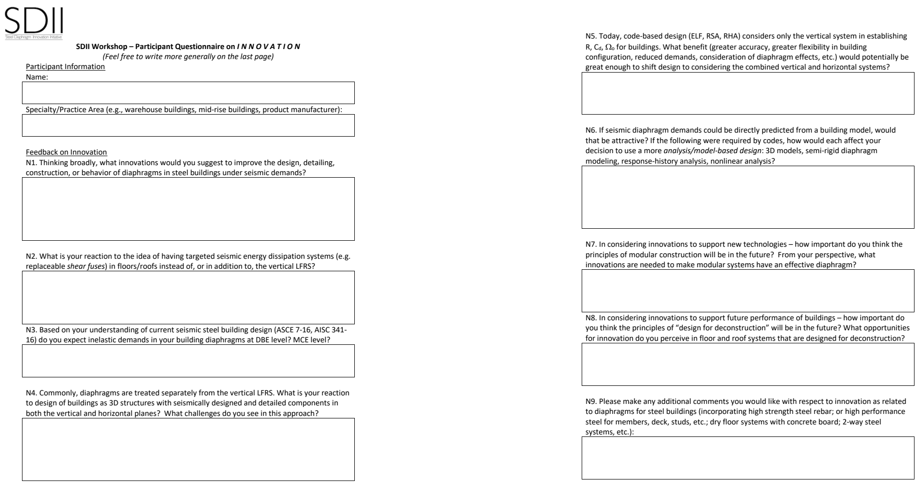
Pour stop? Internal reinforcing at slab edge?

C8. Considering a roof diaphragm system utilizing *bare steel deck diaphragms*, what challenges do you face in making this specific system meet your design constraints?

C9. Considering *chords and collectors* specifically, what challenges do you face in the design and detailing of these members? (clarify if you are addressing floor or roof diaphragms specifically)

C10. Please make any *additional comments* you would like with respect to challenges as related to diaphragms for steel buildings (e.g. codes and standards disconnects, modular buildings, large openings, floor plate shape, transfers, stiffness-mass eccentricities, etc.):

Innovation



SDII Workshop – Participant Questionnaire on *INNOVATION*

(Feel free to write more generally on the last page)

Participant Information

Name:

Specialty/Practice Area (e.g., warehouse buildings, mid-rise buildings, product manufacturer):

Feedback on Innovation

N1. Thinking broadly, what innovations would you suggest to improve the design, detailing, construction, or behavior of diaphragms in steel buildings under seismic demands?

N2. What is your reaction to the idea of having targeted seismic energy dissipation systems (e.g. replaceable *shear fuses*) in floors/roofs instead of, or in addition to, the vertical LFRS?

N3. Based on your understanding of current seismic steel building design (ASCE 7-16, AISC 341-16) do you expect inelastic demands in your building diaphragms at DBE level? MCE level?

N4. Commonly, diaphragms are treated separately from the vertical LFRS. What is your reaction to design of buildings as 3D structures with seismically designed and detailed components in both the vertical and horizontal planes? What challenges do you see in this approach?

N5. Today, code-based design (ELF, RSA, RHA) considers only the vertical system in establishing R, C_d, Ω_o for buildings. What benefit (greater accuracy, greater flexibility in building configuration, reduced demands, consideration of diaphragm effects, etc.) would potentially be great enough to shift design to considering the combined vertical and horizontal systems?

N6. If seismic diaphragm demands could be directly predicted from a building model, would that be attractive? If the following were required by codes, how would each affect your decision to use a more *analysis/model-based design*: 3D models, semi-rigid diaphragm modeling, response-history analysis, nonlinear analysis?

N7. In considering innovations to support new technologies – how important do you think the principles of modular construction will be in the future? From your perspective, what innovations are needed to make modular systems have an effective diaphragm?

N8. In considering innovations to support future performance of buildings – how important do you think the principles of "design for deconstruction" will be in the future? What opportunities for innovation do you perceive in floor and roof systems that are designed for deconstruction?

N9. Please make any additional comments you would like with respect to innovation as related to diaphragms for steel buildings (incorporating high strength steel rebar; or high performance steel for members, deck, studs, etc.; dry floor systems with concrete board; 2-way steel systems, etc.):

General/Everything Else

SDII Workshop – Participant General Response

Name:

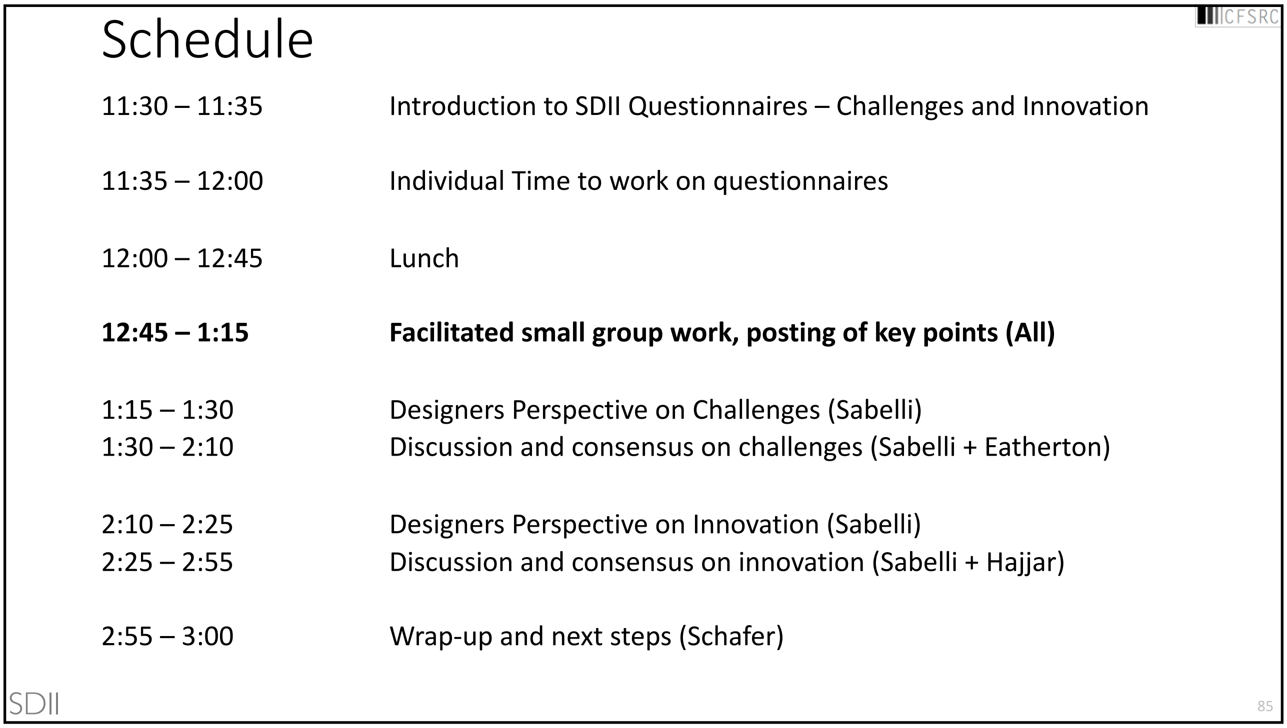
The questions are great, but what you really need to know is:

and you should know this too!

Do you have specific suggestions of how SDII can help overcome the challenges, or develop the innovations, you have detailed above? If yes, please provide that feedback here:

Instructions

- Skim all the questions, start by answering the ones that resonate with you, or that you have some passion/opinion about first.
- Fill out as much as you can, partial credit counts, and all participants receive an A if they turn back their questionnaire
- After we make good progress on answering the questions we will work to create some initial prioritization – so also be thinking about which observations you make that you think are the most important
- We will read and digest all of your responses, even if they are not discussed today – none of your input will go to waste!



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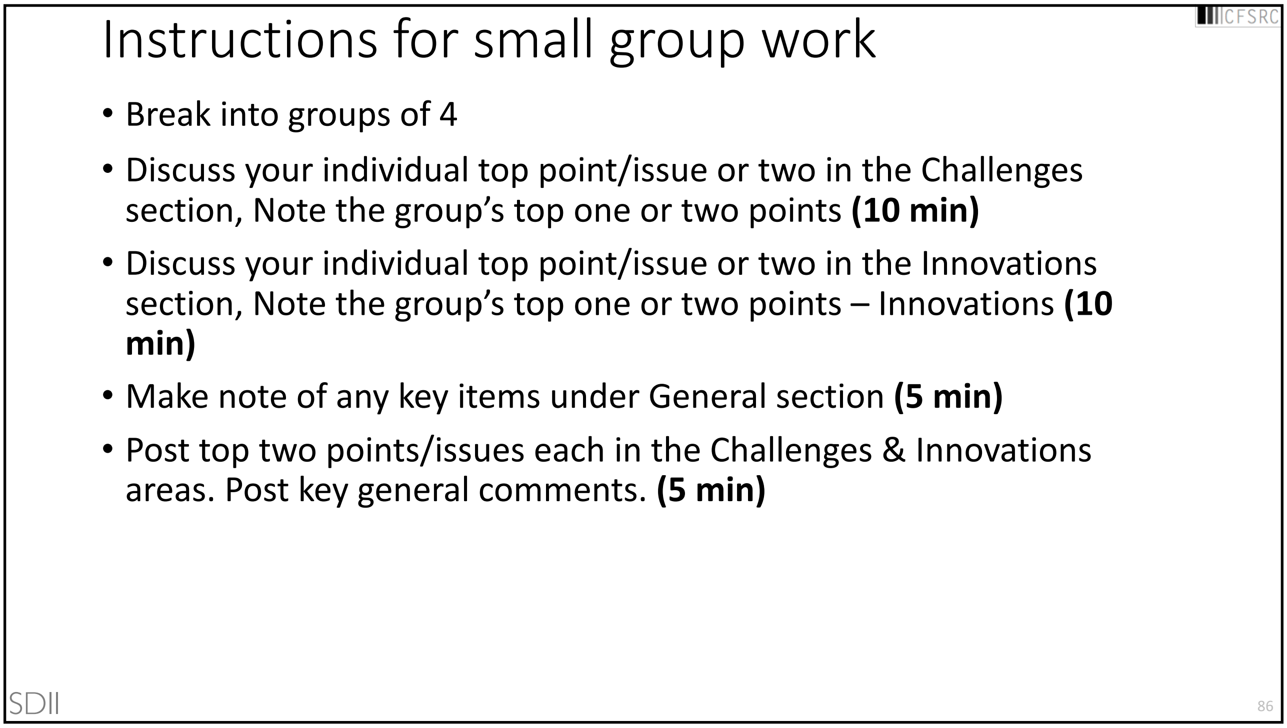

Instructions for small group work

- Break into groups of 4
- Discuss your individual top point/issue or two in the Challenges section, Note the group's top one or two points **(10 min)**
- Discuss your individual top point/issue or two in the Innovations section, Note the group's top one or two points – Innovations **(10 min)**
- Make note of any key items under General section **(5 min)**
- Post top two points/issues each in the Challenges & Innovations areas. Post key general comments. **(5 min)**

SDII Workshop – Participant Questionnaire on *CHALLENGES*

(Feel free to write more broadly on the general response page provided at the end)

Participant Information

Name:

Specialty/Practice Area (e.g., warehouse buildings, mid-rise buildings, product manufacturer):

Feedback on Challenges

C1. Thinking broadly about the *design* of diaphragms for steel buildings, particularly under seismic demands, what are key challenges engineers face from your perspective?

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C4. To what extent do *non-structural* constraints/demands (e.g., fire, acoustics, aesthetics) drive your floor or roof assembly and ultimately your diaphragm design? What challenges do you face with respect to meeting non-structural demands as they relate to the diaphragm?

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C9. Considering *chords and collectors* specifically, what challenges do you face in the design and detailing of these members? (clarify if you are addressing floor or roof diaphragms specifically)

C10. Please make any *additional comments* you would like with respect to challenges as related to diaphragms for steel buildings (e.g. codes and standards disconnects, modular buildings, large openings, floor plate shape, transfers, stiffness-mass eccentricities, etc.):

SDII
Steel Diaphragm Innovation Initiative

SDII Workshop – Participant Questionnaire on *I N N O V A T I O N*

(Feel free to write more generally on the last page)

Participant Information

Name:

Specialty/Practice Area (e.g., warehouse buildings, mid-rise buildings, product manufacturer):

Feedback on Innovation

N1. Thinking broadly, what innovations would you suggest to improve the design, detailing, construction, or behavior of diaphragms in steel buildings under seismic demands?

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SDII Workshop – Participant General Response

Name:

The questions are great, but what you really need to know is:

and you should know this too!

Do you have specific suggestions of how SDII can help overcome the challenges, or develop the innovations, you have detailed above? If yes, please provide that feedback here:

Complete responses were provided by 26 participants. The practice area of the participants was grouped into mid-/high-rise, low-rise and industrial, metal building systems, and academic participants. All responses were logged. An overall narrative summary of the response to each question is provided in Section 7 of this report. Here the individual responses were considered and organized and the key details brought to light by the engineer respondents provided.

Feedback on Challenges

C1. Thinking broadly about the *design* of diaphragms for steel buildings, particularly under seismic demands, what are key challenges engineers face from your perspective?

General Challenges
Stiffness: verification codes are correct
Stiffness: simpler tool for bare deck
Stiffness: When is rigid or flexible close enough
Strength: trust concrete-filled for v high demands?
Demand: ELF equilibrium vs. diaphragm demand
Demand: use of R vs Rs guidance
Demand: How to properly distribute +2
Demand: How to capacity protect/apply CBD?
Irregularity: plan irregular +3 how to handle
Irregularity: interior (many) supports, handle correctly
Metal building: standing seam roof contribution
Metal building: multiple semi-rigid interior supports
Metal building: horizontal truss interaction, guidance
Design: simplest method possible needed
New products: provide clear path
Workflow: when is semi-rigid unneccessary
Workflow: too many load cases, simplify
Workflow: tools don't have diaphragm checks
Workflow: no time to improve diaphragm design
Training: only better firms do diaphragms right or close
Training: need solid diaphragm design examples for practice
Training: explain load path through diaphragms
Best practices: best detail to perimeter/ C&C
Best practices: best fasteners to choose in steel deck

C2. For your design/analysis workflow tell us about how you *model* building diaphragms:

How we model
Rigid or flexible assumption used wherever possible
Elastic truss or shells used for semi-rigid as needed
Large transfers, spans, openings --> model to semirigid

C3. When *modeling* the diaphragm of a steel building, what challenges do you face? What challenges do you face/perceive when interpreting your model results to your satisfaction?

Model challenges

Challenges with model inputs

Stiffness: Get it right/with confidence +3 (bare & filled)
How to include secondary elements
Handling eccentricities in models
How to model non-discrete/perimeter C&C elements
How to model inelastic response of diaphragm
Dealing with openings, and forces around openings

Challenges with model outputs

Converting shell FE to force on discrete components
Equilibrium & understanding of chord and collector force

General modeling challenges

Concerns about limit states outside of model
Annoyance: diaphragm analysis is separate model!
Software limitations

Challenges with simplified modeling methods

Issues with simple diaphragm models when multiple supports
Plan irregularities
Bare deck fastener zone effects, how to include

C4. To what extent do *non-structural* constraints/demands (e.g., fire, acoustics, aesthetics) drive your floor or roof assembly and ultimately your diaphragm design? What challenges do you face with respect to meeting non-structural demands as they relate to the diaphragm?

Non-structural

Steel deck with fill:

Fire essentially controls fill depth, all agree
Acoustic (mass) may control fill depth, many comment
Openings other architectural choice can drive diaphragm
Embed depths for anchorage can drive thickness

Bare steel deck:

MEP vibration can rule out bare steel roof deck
Roof steps, slopes, and openings create challenges

C5. Considering the most prominent available floor diaphragm system: *steel deck diaphragms with headed shear studs and concrete fill*, what challenges do you face in making this specific system meet your design constraints?

Challenges with steel deck with composite fill
Capacity and design:
What is the correct capacity/design strength?
How to combine gravity and diaphragm forces on studs
Correct approach for defining number of studs
No consensus on shear demand w/ int. shear elements
Chords and collectors:
Can the diaphragm be the C&C or do I need discrete
Rebar in the slab or use discrete steel
Strength at collectors
No consensus on how to get shear in rigid diaphragms
Transfer to collectors, capacity?
Misc.
Openings
Anchorage for nonstructural equipment driving thickness
Can this redistribute load?

C6. Again considering *steel deck diaphragms with headed shear studs and concrete fill*, do you include supplemental reinforcement/rebar (beyond temperature and shrinkage steel) in the fill to meet diaphragm demands or serve as chords/collectors? Please explain why/why not.

Supplemental reinforcement
Team reinforcement:
Use when demands are higher
Clearer load path with reinforcement as C&C
Adequate confinement?
How to transfer out to frame?
Limit cracking
Some say common, others say not common.
Team discrete steel:
Don't trust rebar in thin slabs
Clearer load path with discrete steel C&C

C7. Again for *steel deck diaphragms with studs and fill*, what is your typical slab edge detail?

Pour stop (1) CFS or (2) bent plate
Detail depends on whether slab supports cladding
If loaded then some use studs to embed bent plate
If loaded then some use rebar to strengthen plate/Pour stop? Internal reinforcing at slab edge?

C8. Considering a roof diaphragm system utilizing *bare steel deck diaphragms*, what <u>challenges</u> do you face in making this specific system meet your design constraints?

Challenges with bare steel deck

<u>Fundamental</u>

Insufficient capacity for high seismic demands
Large drift accommodation for gravity columns

<u>Connectors</u>

Connector ductility
Welding QC
Too many fastener options

<u>Functional</u>

MEP vibration
MEP anchorage/support

<u>Detailing</u>

Openings, collectors at openings
Detailing for chord, collector, steps
Detailing simple so that it gets constructed properly

<u>More</u>

Delegated responsibilities --> incoherent system
How to handle standing seam roofs

C9. Considering *chords and collectors* specifically, what challenges do you face in the design and detailing of these members? (clarify if you are addressing floor or roof diaphragms specifically)

Chords and Collectors (C&C)

<u>Slab related in steel deck with fill</u>

Force transfer in composite floors
Understanding slab as C&C vs. dragging into steel frame
When slab is C&C how to check confinement?

<u>Steel framing C&C issues</u>

Collector makes braced frames moment frames, ok?
Chord and collector splices costly
C&C continuation at opening conflicts w/ gravity system
C&C connection design criteria, and stability criteria
Beam used as chord, proper shear stud design for C&C?
Support on collector beams / not having LTB support
Connection detailing
Collectors transverse to primary framing: costs!

<u>More</u>

Good details that transfer for diaphragm to C&C
Movement detailing in (1-dir) at hard walls
Training - lack of knowledge in many engineers

C10. Please make any *additional comments* you would like with respect to challenges as related to diaphragms for steel buildings (e.g. codes and standards disconnects, modular buildings, large openings, floor plate shape, transfers, stiffness-mass eccentricities, etc.):

More challenges
Modular building diaphragms
How to model semi-rigid diaphragm in complex cases
Consequence of failure for steel deck with fill
Quantitatively established diaphragm ductility
When do we move to strut and tie model for fill?
Building corner issues
Definitive guidance on stiffness, that software embraces
Limitations of AISI S310 diaphragm analytical methods
Limited deisgn time for engineer
Irregularities are really a challenge

Feedback on Innovation

N1. Thinking broadly, what innovations would you suggest to improve the design, detailing, construction, or behavior of diaphragms in steel buildings under seismic demands?

Needed innovation
Behavior and Design Improvements
Understanding inelastic demands in hLFRS vs vFLRS
Handle hLFRS in a manner compatible with vLFRS
Create combined R systems
Establish when simple methods work well enough
Clearer direction and design philosophy
Define connector classes
Determine diaphragm ductility vs response trade off
Software improvements
Models that are easy to pull forces from for diaphragm
Automated design (sp. Steel deck with fill) in software
Design support for deck over CFS
Technology innovation
Better diaphragm to framing connections +4
Inexpensive cellular deck
Nextgen standing seam roof
Concentrated/isolated energy dissipating fuses
Isolation solutions, high damping solutions
Detailing the promotes ductility in diaphragm

N2. What is your reaction to the idea of having targeted seismic energy dissipation systems (e.g. replaceable *shear fuses*) in floors/roofs instead of, or in addition to, the vertical LFRS?

Reaction to fuses in diaphragm?

Difficulties

Vertical and lateral system deformation compatibility +3
Openings at columns and fire separation big problem +2
Difficult to detail, reliability concerns
Cost +4
Energy dissipation not needed here
Compatibility issues with finishes
New Hilti detail already in the works.

Opportunity

Possible for high end projects, very unique structures
As long as gravity support maintained, high potential
Real merit but real complication at the same time
Paired with BRBs?
Excellent potential for such systems

N3. Based on your understanding of current seismic steel building design (ASCE 7-16, AISC 341-16) do you expect inelastic demands in your building diaphragms at DBE level? MCE level?

Expect inelastic demands?

DE level
no -12
some -5
yes -3
MCE level
no -4
some -6
yes -10

N4. Commonly, diaphragms are treated separately from the vertical LFRS. What is your reaction to design of buildings as 3D structures with seismically designed and detailed components in both the vertical and horizontal planes? What challenges do you see in this approach?

3D building design?

Technical Challenges

How to get what you want if vLFRS & hLFRS both yield?
How to handle dist. Floor mass
System categorization is going to be challenging
Separate R and Rs don't conflate - important for 3D

Practical Challenges

It is going to be really complex

How to handle low fee projects
Baffling & disturbing increased complexity for engineer
Make it practical, keep it simple where you can +3
Too complex except for RWFD type structure
Define when you need to go 3D.. Critical
Opportunity/advantage
We may need to this to avoid unanticipated limit states
Need to provide this in code for complex geometry bldg
For complex industrial bldg, conv center, stadia, di it
Great addition to the code as an OPTION only

N5. Today, code-based design (ELF, RSA, RHA) considers only the vertical system in establishing R, C_d, Ω_o for buildings. What benefit (greater accuracy, greater flexibility in building configuration, reduced demands, consideration of diaphragm effects, etc.) would potentially be great enough to shift design to considering the combined vertical and horizontal systems?

Benefit that moves the design needle?
Better performance
Reliability
Repairability
Reduced demands --> $ savings. +7
Reduced collectors/reinforcing
Greater accuracy +2
Design flexibility

N6. If seismic diaphragm demands could be directly predicted from a building model, would that be attractive? If the following were required by codes, how would each affect your decision to use a more *analysis/model-based design*: 3D models, semi-rigid diaphragm modeling, response-history analysis, nonlinear analysis?

Demands direct from building model?
Yes
Mid-/high-rise respondents are all yes
If it also includes C&C
More motions? Bounds?
Semi-rigid would complicate things
Yes, but don't require overly complex model
No
Low-rise Industrial respondents about 50% no
Keep it simple
Black box concerns
Too much for ELF
Not billable work

N7. In considering innovations to support new technologies – how important do you think the principles of modular construction will be in the future? From your perspective, what innovations are needed to make modular systems have an effective diaphragm?

Modular?
Pros
Going to increase, popular, time efficient +2
Panelization yes, modularization not so sure
Concerns
Diaphragm stability for columns challenging
Diaphragm continuity a concern
Connections of modules needs more thinking +7
QA/QC at connections
Architects will have reservations +2
Just do $\Omega_o$for modular?

N8. In considering innovations to support future performance of buildings – how important do you think the principles of “design for deconstruction” will be in the future? What opportunities for innovation do you perceive in floor and roof systems that are designed for deconstruction?

Deconstruction?
Couple this concept with modularization!
Good for special type of owner
Design life more important
Not important +5
Benefit that is not aligned with decision-makers +2
Connections the key +2
Cost does not make it worth it
Only if regulation, government drives this direction +2

N9. Please make any additional comments you would like with respect to innovation as related to diaphragms for steel buildings (incorporating high strength steel rebar; or high performance steel for members, deck, studs, etc.; dry floor systems with concrete board; 2-way steel systems, etc.):

RWFD/big box biggest immediate concern
Diaphragms brace gravity columns, cant lost sight of this
Consider broader suite of building types
Concrete panels on deck good niche system needs study
Greater focus on constructability
Increase inspectability for post EQ
Guidance for verifying output of models

The questions are great, but what you really need to know is:

Stay focused on specification impact
Keep it simple or it can’t be used +4
Standardize steel deck to spur its use, like studs did
Will anchorage forces change?
Rebar in steel deck with fill, need definite method
Plan irregular RWFD, what to do?
AC vs new AISI S310, difference?
How to handle wind vs. seismic in diaphragm design
Are we losing welds in bare steel deck
Keep it simple