# R-Factors for Coupled Composite Plate Shear Walls—Concrete Filled (Coupled-C-PSW/CF)

Presented to BSSC PUC
San Francisco, April 4, 2018

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University at Buffalo
Amit Varma
Purdue University



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## Project Scope

- This project seeks R-Factors developed from FEMA P-695 studies for Coupled Composite Plate Shear Walls—Concrete Filled (Coupled-C-PSW/CF), for inclusion in ASCE-7, higher than R-factors for corresponding non-coupled walls.
- Interacting with BSSC IT-4 (parallel work being conducted on coupled RC walls – and possibly coupled SPSW)

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## Agenda

- Introduction - 10 minutes
- Status Report on Archetypes Development – 15 minutes
- Status Report on Inelastic Modeling – 15 minutes
- Tasks Ahead – 5 minutes
- Q&A – 15 minutes
- Next meetings with PUC

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## Introduction

- Research Team, Project Scope, Sponsors, Timeline
- Description of composite walls system
- Past experiments (focusing on cyclic inelastic response)
- Overview of on-going experiments (Pankow/AISC)
  - University at Buffalo
  - Purdue University

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## Research Team, PAG, PRP

- PI and co-PI (Michel Bruneau and Amit Varma)
- Project Advisory Group
  - Larry Kruth, AISC
  - Jim Malley, Degenkolb Engineers
  - Rafael Sabelli, Walter P. Moore & Associates
  - Tom Sabol, Englekirk Institutional
  - John Hooper, Magnusson Klemencic Associates
  - Bonnie Manley, AISI
- FEMA P-695 Peer Review Panel members
  - Greg Deierlein, Stanford University
  - Rafael Sabelli, Walter P. Moore & Associates
  - Ron Klemencic, Magnusson Klemencic Associates

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## Pankow/AISC Project (Started 7/1/2017):

"R-Factors for Coupled Composite Plate Shear Walls--Concrete Filled (Coupled-C-PSW/CF)"

- Collaborative Research Effort (UB and Purdue)
- FEMA P-695 Analyses to investigate relative R-factor values for uncoupled CF-CPSW versus Coupled CF-CPSW

## Pankow/AISC Project (Started 9/1/2016):

"Seismic and Wind Behavior and Design of Coupled CF-CPSW Core Walls for Steel Buildings"

- Collaborative Research Effort (Purdue and UB)
- Experimental and Analytical Investigation of Core Walls Systems (with composite link beams)

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## FEMA P-695 REQUIREMENTS

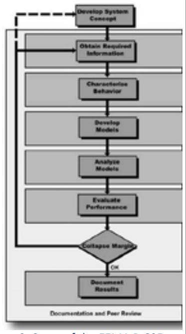


Figure 2: Steps of the FEMA P-695 Methodology



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## PLANS FORWARD – Updated March 31st 2018

| Task | Duration | Tentative Schedule |
|---|---|---|
| 1 – Selection of Building Archetypes* (Archetype generator developed – first set or archetypes produced – PRP reviewing) | 2 months | 6/1/2017 - 7/31/2017* |
| 2 – Selection of Representative Coupling Ratios (Agreed with PRP that these would be based on beam-to-depth ratios) | 3 months | 8/1/2017 – 10/31/2017 |
| 3 – Determination of non-linear models (Completed – PRP reviewing) | 4 months | 11/30/2017 – 2/28/2018 |
| 4 – FEMA-P-695 Analyses (IDA trial runs with 3 and 8 story models – PRP reviewing) | 2 months | 3/1/2018 – 4/30/2018 |
| 5 – Summary of Findings and Formulation of Recommendations (Interim Technical Report: March 31st 2018. PRP feedback due 4/11/18) | 1 month | 5/1/2018 – 5/31/2018 |

* Start date (subcontract with Purdue executed): September 18th 2017
* Notified Peer Review Panel Assembled: November 8th 2017

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## C-PSW/CF

- Concrete-filled steel sandwich
- Steel serves as formwork and able to resist gravity loads during erection
- Shipped assembled in segments

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## Description of Composite Walls System (C-PSW/CF)

BI-Steel Core Fast walls – Corus Steel

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## Non-Seismic Applications

http://www.archiexpo.fr/prod/tata-steel/product-88366-1176033.html

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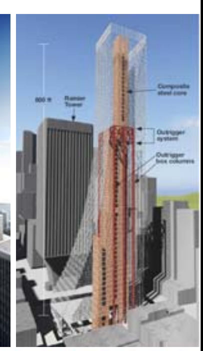
## Implementation

- Rainier Square Project
  - 58 Stories
  - Seattle
  - Under construction
  - MKA Project



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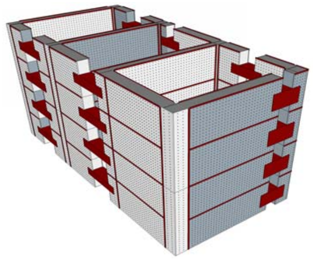
## Concept — Composite-Plate Shear Wall / Concrete Filled



*Courtesy of John Hooper, MKA Engineers*





## Concept — Composite-Plate Shear Wall / Concrete Filled

*Courtesy of Ron Klemencic, MKA Engineers*





## ASCE-7 2010

- Table 12-2-1 of ASCE-7 2010 refers to "composite plate shear walls" and ASCE Section 14.3 for detailing requirements, which itself, refers to AISC 341-10.

CHAPTER 12 SEISMIC DESIGN REQUIREMENTS FOR BUILDING STRUCTURES

**Table 12.2-1** *(Continued)*

| Seismic Force-Resisting System | ASCE 7 Section Where Detailing Requirements Are Specified | Response Modification Coefficient, R | Overstrength Factor, Ω | Deflection Amplification Factor, $C_d$ | Structural System Limitations Including Structural Height, $h_n$ (ft) Limits — Seismic Design Category B | C | D | E | F |
|---|---|---|---|---|---|---|---|---|---|
| 4. Special reinforced concrete shear walls | 14.2 | 6 | 2½ | 5 | NL | NL | 160 | 160 | 100 |
| 5. Ordinary reinforced concrete shear walls | 14.2 | 5 | 2½ | 4½ | NL | NL | NP | NP | NP |
| 6. Detailed plain concrete shear walls | 14.2 and 14.2.2.8 | 2 | 2½ | 2 | NL | NP | NP | NP | NP |
| 7. Ordinary plain concrete shear walls | 14.2 | 1½ | 2½ | 1½ | NL | NP | NP | NP | NP |
| 8. Intermediate precast shear walls | 14.2 | 5 | 2½ | 4½ | NL | NL | 40 | 40 | 40 |
| 9. Ordinary precast shear walls | 14.2 | 4 | 2½ | 4 | NL | NP | NP | NP | NP |
| 10. Steel and concrete composite eccentrically braced frames | 14.3 | 8 | 2½ | 4 | NL | NL | 160 | 160 | 100 |
| 11. Steel and concrete composite special concentrically braced frames | 14.3 | 5 | 2 | 4½ | NL | NL | 160 | 160 | 100 |
| 12. Steel and concrete composite ordinary braced frames | 14.3 | 3 | 2 | 3 | NL | NL | NP | NP | NP |
| 13. Steel and concrete composite plate shear walls | 14.3 | 6½ | 2½ | 5½ | NL | NL | 160 | 160 | 100 |
| | | | 2½ | 4 | NL | NL | NP | NP | NP |
| | | | 2½ | 2 | NL | 160 | NP | NP | NP |
| | | | 2½ | 2 | NL | NP | NP | NP | NP |
| | | | 2½ | 1½ | NL | NP | NP | NP | NP |
| | | | 2½ | 1½ | NL | NP | NP | NP | NP |
| | | | 2½ | 4½ | NL | NL | 65 | 65 | 65 |
| | | | 2½ | 4½ | NL | NL | 65 | 65 | 65 |
| | | | 2½ | 2½ | NL | NL | 35 | NP | NP |
| 25. Steel buckling-restrained braced frames | 14.1 | 8 | 2½ | 5 | NL | NL | 160 | 160 | 100 |
| 26. Steel special plate shear walls | 14.1 | 7 | 2 | 6 | NL | NL | 160 | 160 | 100 |

| | | | | |
|---|---|---|---|---|
| 13. Steel and concrete composite plate shear walls | 14.3 | 6½ | 2½ | 5½ |

**14.3.3 Seismic Requirements for Composite Steel and Concrete Structures**

Where a response modification coefficient, *R*, in accordance with Table 12.2-1 is used for the design of systems of structural steel acting compositely with reinforced concrete, the structures shall be designed and detailed in accordance with the requirements of AISC 341.





## AISC 341-10 H6

- AISC H6.1: Scope indentifies two types of C-PSW:
  - Steel plate encased in concrete, designed like SPSW (i.e., with HBEs and VBEs), but yielding in shear at 0.6AFy (instead of diagonal tension yielding)
  - Concrete sandwiched between steel plates: plastic flexural behavior is more likely (instead of plate shear yielding).

**H6. COMPOSITE PLATE SHEAR WALLS (C-PSW)**

**1. Scope**

*Composite plate shear walls* (C-PSW) shall be designed in conformance with this section. Composite plate shear walls consist of steel plates with reinforced concrete encasement on one or both sides of the plate, or steel plates on both sides of reinforced concrete infill, and structural steel or composite *boundary members*.





## AISC-341-16

**H6. COMPOSITE PLATE SHEAR WALLS – CONCRETE ENCASED (C-PSW/CE)**

**H6.1. Scope**

... l be ... steel ... f the

**H7. COMPOSITE PLATE SHEAR WALLS—CONCRETE FILLED (C-PSW/CF)**

**H7.1. Scope**

Composite plate shear walls-concrete filled (C-PSW/CF) shall be designed in conformance with this section. This section is applicable to composite plate shear walls that consist of two planar steel web plates with concrete fill between the plates, with or without boundary elements. Composite action between the plates and concrete fill shall be achieved using either tie bars or a combination of tie bars and shear studs. The two steel web plates shall be of equal thickness and shall be placed at a constant distance from each other and connected using tie bars. When boundary members are included, they shall be either a half circular section of diameter equal to the distance between the two web plates or a circular concrete-filled steel tube.



## Composite Plate Shear Walls (C-PSW) - Concrete Filled (CFSSP/CF)

- Scope of AISC 341-16 Article H7
  - Composite Steel Plate Walls with Boundary Elements
  - Composite Steel Plate Walls without Boundary Elements

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## Composite Plate Shear Walls (C-PSW) - Concrete Filled (CFSSP/CF)

- Scope of AISC 341-16 Article H7
  - Composite Steel Plate Walls with Boundary Elements
  - Composite Steel Plate Walls without Boundary Elements
- Supported by Results of Experiments on Cyclic Inelastic Behavior of Concrete-Filled Steel Sandwich Walls
  - Research Project Funded by AISC

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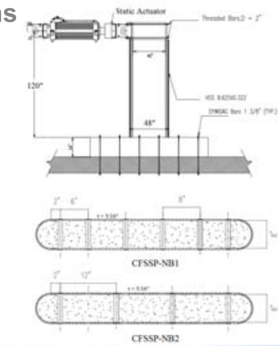
## Group NB Specimens

Height to Width ratio, h/w =2.5

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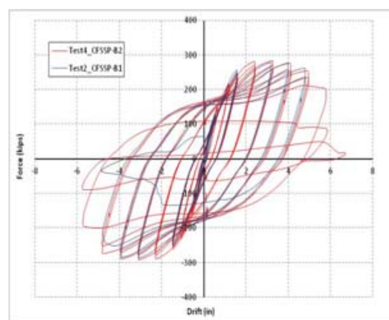
## Comparison between CFSSP-B1 &CFSSP-B2

- Initiation of fracture in tie bar steel web connection occurred at higher drift values in CFSSP-B2 (fillet welded tie bars)
- Deterioration of specimen CFSSP-B2 occurred less abruptly

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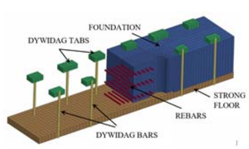
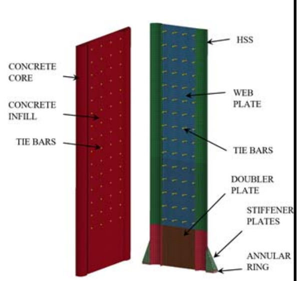
## LS-DYNA MODELING

- *winfrith_concrete* (*Mat 085*) model
- *plastic_kinematic* (*Mat 003*) bilinear material model with kinematic hardening

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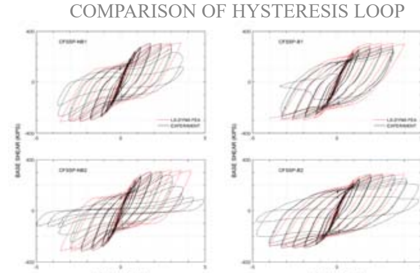
## LS-DYNA RESULTS

COMPARISON OF HYSTERESIS LOOP



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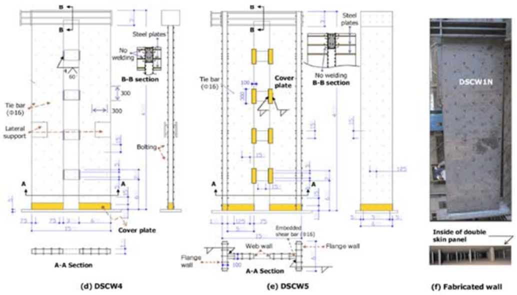
## Cyclic Load Testing on Concrete Filled Sandwich Panels, Park et al (2009)





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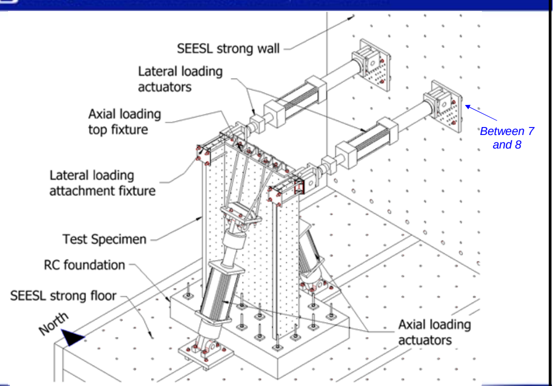
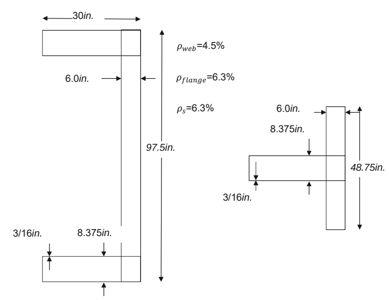







## Cross-section dimensions








Test Matrix: Proposed alternative 5 (4 Specimens)

C-shape, $f_c'$=6ksi, P=$0.15 f_c' A_c$ (2)

Effect of axial load

C-shape, $f_c'$=6ksi, No axial

C-shape, $f_c'$=4ksi, P=$0.22 f_c' A_c$ (1)

High range axial load (i.e., 22%) on C-shape.

C-shape to T-shape

T-shape, $f_c'$=6ksi, P=$0.15 f_c' A_c$ (3)

Effect of axial load

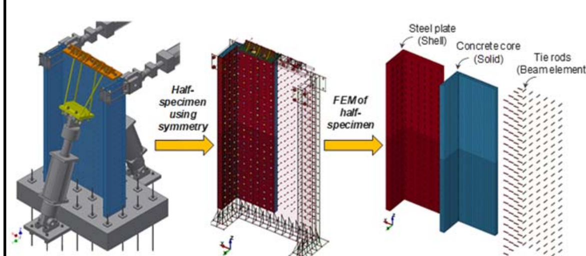
T-shape, $f_c'$=6ksi, P=$0.30 f_c' A_c$ (4)

- *Higher range axial load (i.e., 30%) with expected range of concrete strength (i.e., 6ksi) is included.*
- *Comparison between behavior of C-shape and T-shape wall is included.*

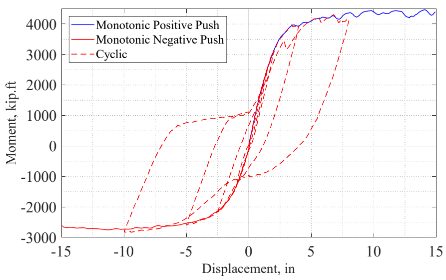




## Finite Element Model



## Pre-test finite element results








## FEMA P-695 REQUIREMENTS

- Peer Review Panel (PRP) responsible for reviewing and commenting on at all steps of the approach taken by the system development team (see figure)
- PRP members to evaluate the development and provide an unbiased assessment (FEMA 2009)
- PRP meetings to review:
  - Archetype selection
  - Inelastic models
  - IDA results and interpretation
  - Selection of β factors
  - Others?

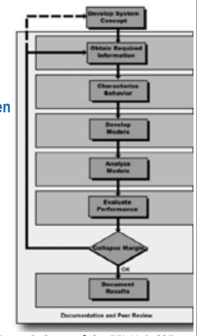


Figure 2: Steps of the FEMA P-695 Methodology





## Status Report on Archetypes Development

- Process/Rationale to Develop Archetypes
- Essence of design checks
- List of archetypes planned
- Archetypes designed so far





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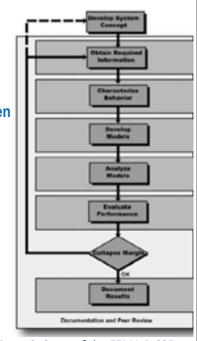


Figure 2: Steps of the FEMA P-695 Methodology





## Status Report on Inelastic Modeling

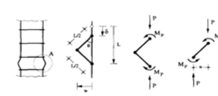
- Selection of OpenSees material models
  - *Reinforcing Steel Material (*Kunnath)
- Model calibration
- Trial IDA runs and results



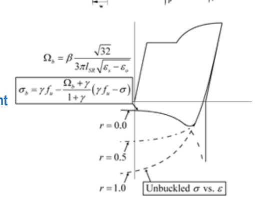


## CONSTITUTIVE EQUATIONS (cont'd)

- Cyclic Buckling:
- Two buckling models were introduced. Both are empirical:
  - Gomes and Appleton model [3]



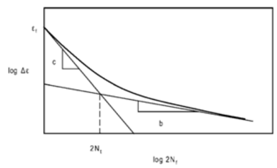
- GOMES and APPLETON Model [3]:
  - A stress equation was derived from buckled reinforcement in the deform configuration and compatibility between transversal and longitudinal displacement
  - It includes the axial force and plastic moment capacity of bars.
  - We have different cross section, so the parameters will be chosen to calibrate against experimental results



## CONSTITUTIVE EQUATIONS (cont'd)

- Low Cycle Fatigue:
  - If the cyclic response mostly dominated by plastic strain, the strain amplitude can be expressed by Coffin [4] and Manson [5] equation:

$$\Delta\varepsilon^p = \varepsilon_f' * \left(2 * N_f\right)^c$$




  - Cumulative damage is calculated by Palmgren-Miner rule [7]

$$2N_f = \left(\frac{\Delta\varepsilon^p}{\varepsilon_f'}\right)^{-\frac{1}{c}} \rightarrow D = \Sigma\left(\frac{1}{2*N_f}\right)$$

if D = 1.0 ➔ Steel area fractures and stress goes to zero !

  - Model uses "cycle counting" method for earthquakes to determine cycles to failure ($2*N_f$); gives same result as "rainflow counting" (Matsuishi and Endo [11]).



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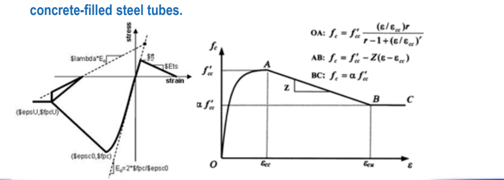
## CONSTITUTIVE EQUATIONS (cont'd)

- *CONCRETE MODEL:*
- Concrete02 [2] was used to define concrete properties as
  - It reloads where it unloaded.
  - Inputs are in the figure
- The inputs are from Susantha et al. [8] paper
  - It is for calculating the confined concrete properties based on the shape of section and plate width-to-thickness (or radius-to-thickness) ratios for concrete-filled steel tubes.

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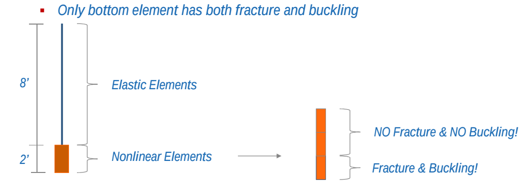
## ELEMENT TYPE

- *Displacement based elements (dispBeamColumn) were used for plastic hinge length.*
- *According to Neuenhofer and Filippou [12], accuracy of results improves with number of elements used in plastic hinge region.*
- *Example below:*
  - *3 x 8" elements in plastic hinge length*
  - *Only bottom element has both fracture and buckling*

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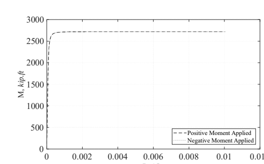
## TRIAL MODELS

- OpenSees [2] does not have a graphic post-processor. Therefore, trial models were conducted to make sure that the model is correct and worked as intended!
- Checking Cross Section:
  - Plastic moment capacity (Mp) from OpenSees [2] and hand calculation should be exact!
  - Elastic-Perfectly Plastic (EPP) for steel and EPP with no tensile material for concrete.




OpenSees Fiber Analysis: => Mp = 32616 kip*in, PNA = 9.5"

Hand Calcs : => Mp = 32622 kip*in, PNA = 9.57"

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## TRIAL MODELS (cont'd)

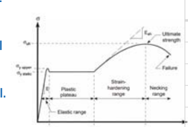
- Fracturing and propagating through cross section in cyclic loading:
- Grade 50 steel and EPP with 4ksi strength and no tensile material for concrete model with only fatigue properties; and fatigue and buckling together. Fatigue properties for Grade 50 steel are from Kaufmann et al. [9]

  c = 0.47  $\varepsilon_f$ = 0.4

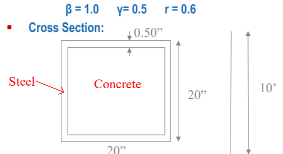
- Buckling Parameters (trial):

  β = 1.0  γ = 0.5  r = 0.6

- Cross Section:




| | Steel |
|---|---|
| Fy (ksi) | 50 |
| Fu (ksi) | 65 |
| Es (ksi) | 29000 |
| Esh (ksi) | 1/30*Es |
| εsh | 10*εy |
| εu | 0.15 |




- Number of layers in top steel = 40 (to make area : 0.5x0.5 = 0.25 in^2)
- Number of layers in web steel = 38 (to make it square)

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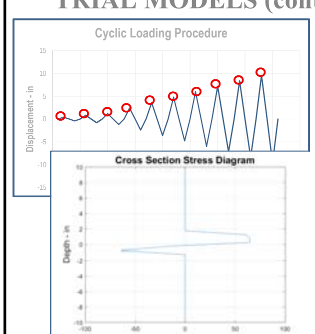
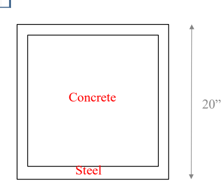
## TRIAL MODELS (cont'd)




- Fatigue only Model:

Δy = 1.2 "

So the protocol is: Δy/3 ; 2Δy/3 ; Δy ; 2Δy ; 3Δy , etc.

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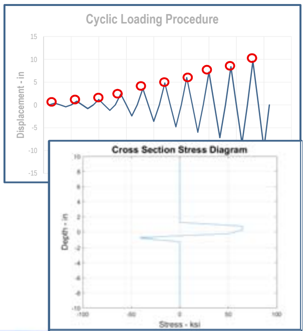
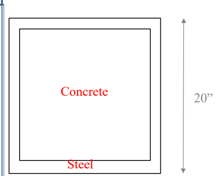
## TRIAL MODELS (cont'd)




- Fatigue & Buckling Model:

$\Delta y = 1.2$ "

So the protocol is: $\Delta y/3$ ; $2\Delta y/3$ ; $\Delta y$ ; $2\Delta y$ ; $3\Delta y$ , etc.



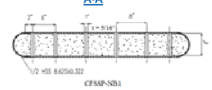


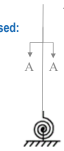



## MODEL VERIFICATION

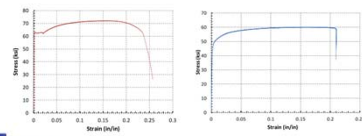
- Alzeni [10] test results for no-boundary (NB1) section were used:

A-A




- *Steel Model Parameters*
- *Backbone Curve:*
  - Two Backbone Curves from Coupon Test for this cross section for web plate steel and HSS




| | Web Steel | HSS |
|---|---|---|
| Fy (ksi) | 63 | 44 |
| Fu (ksi) | 72.5 | 61 |
| Es (ksi) | 29800 | 27500 |
| Esh (ksi) | 0.0072*Es | 0.0514*Es |
| εsh | 0.021 | 0.0017 |
| εu | 0.15 | 0.15 |





## MODEL VERIFICATION (cont'd)

- Cyclic Buckling :
- Alzeni's NB1 specimen [10] first buckled at 2.4% drift at HSS between first and second tie rows. So following parameters were chosen:

$\beta = 1.0$ $\gamma = 1.0$ $r = 0.4$

- Low Fatigue Life:
- Alzeni's NB1 specimen [10] first fractured at 3.6% drift at HSS between first and second tie rows (mid of buckled wave). So following parameters were chosen:

$c = 0.45$ $\varepsilon'_f = 0.3$ $C_d = 0.5$ (for cyclic degradation)

| Cumulative Plastic Strain | Number of Cycle |
|---|---|
| 0.01 | 1916 |
| 0.05 | 54 |
| 0.1 | 11 |
| 0.15 | 5 |
| 0.2 | 2 |
| 0.25 | 1 |

- Number of Fibers in model of Alzeni's wall :
- Number of layers along boundary arc = 44
- Number of layers along web steel = 128

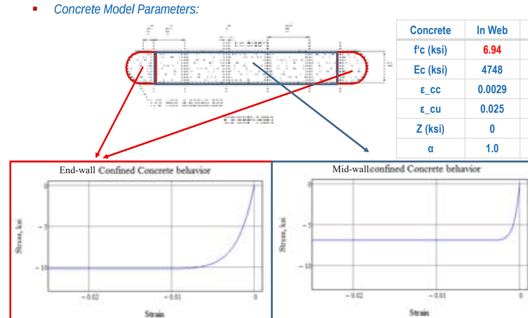




## MODEL VERIFICATION (cont'd)

- *Concrete Model Parameters:*

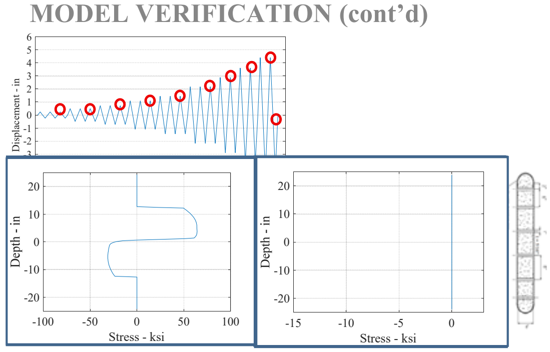
| Concrete | In Web | In HSS |
|---|---|---|
| f'c (ksi) | 6.94 | 10.21 |
| Ec (ksi) | 4748 | 4748 |
| ε_cc | 0.0029 | 0.0093 |
| ε_cu | 0.025 | 0.025 |
| Z (ksi) | 0 | 0 |
| α | 1.0 | 1.0 |








## MODEL VERIFICATION (cont'd)



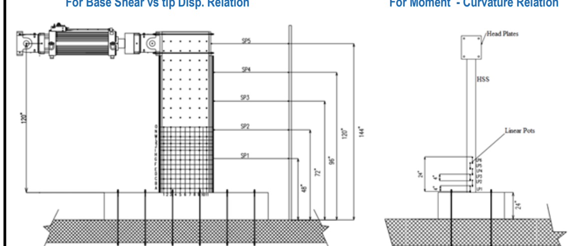





## TEST DATA LOCATIONS

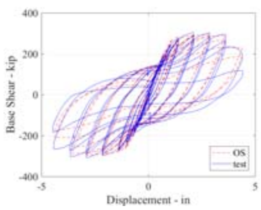
For Base Shear vs tip Disp. Relation

For Moment - Curvature Relation

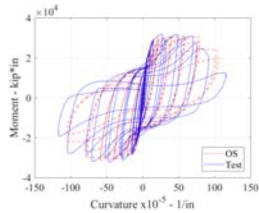


## VERIFICATION OF OpenSees

For Base Shear vs tip Disp. Relation



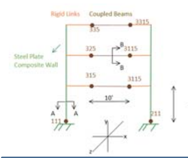

For Moment - Average Curvature Relation

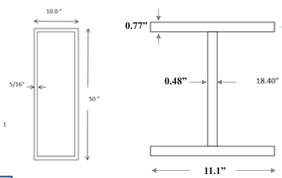


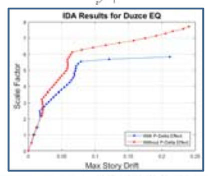





## TRIAL IDA



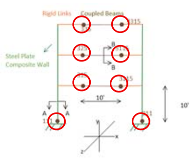




- 30 % coupling ratio
- Nonlinearity only assigned to hinges
- Elastic beam column elements having E/1000 stiffness placed on top of nonlinear elements for computational stability after full cross-section fracture

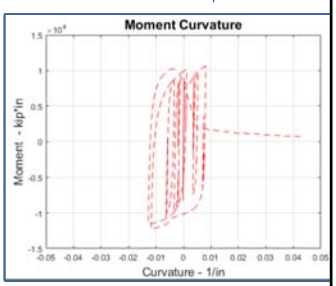




## TRIAL IDA (cont'd) – Moment Curvature




- Moment Curvature Curves at last step.

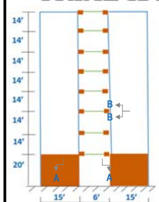



- Number of Layers :
- For Wall:
  - Number of layers in top steel = 20
  - Number of layers in web steel = 158
- For Coupling Beams:
  - Number of layers in top steel = 16
  - Number of layers in web steel = 36

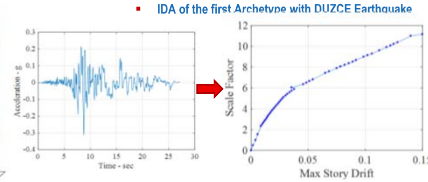




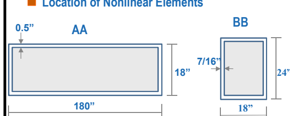
## TRIAL IDA ON FIRST ARCHETYPE




- IDA of the first Archetype with DUZCE Earthquake




- Location of Nonlinear Elements

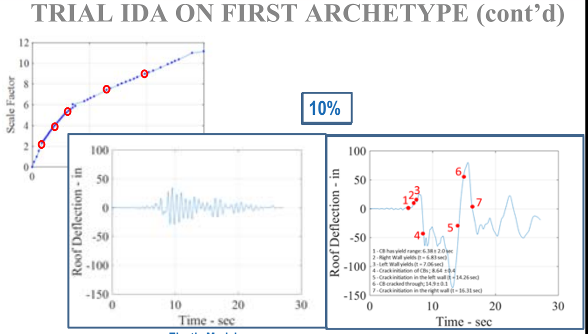



- Number of Layers :
- For Wall:
  - Number of layers in top steel = 42
  - Number of layers in web steel = 358
- For Coupling Beams:
  - Number of layers in top steel = 42
  - Number of layers in web steel = 52





## TRIAL IDA ON FIRST ARCHETYPE (cont'd)




- Elastic Model
- Nonlinear Model





## Tasks Ahead

- IDA
  - Need PRP approval of archetypes and non-linear models
  - Start analysis of all archetypes
- Beta factors
  - Two M.Sc. Students reviewing past FEMA P-695 studies to document Beta factors
  - $\beta_{RTR}$ = record-to-record collapse uncertainty (0.20 - 0.40)
  - $\beta_{DR}$= design requirements-related collapse uncertainty (0.10 – 0.50)
  - $\beta_{TD}$ = test data-related collapse uncertainty (0.10 – 0.50)
  - $\beta_{MDL}$ = modeling-related collapse uncertainty (0.10 – 0.50)

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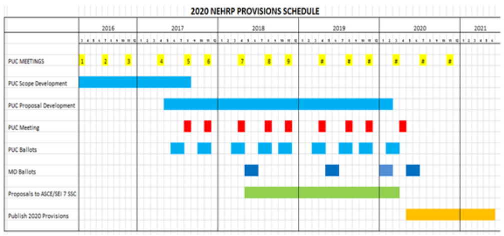
## Q&A

- Next meetings with PUC?

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