# *Seismic Design of Tilt-Up Walls*

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IN OUR GRIT, OUR GLORY™

- **Tilt Up Construction**
- **Panel Design and Detailing Procedures**
- **In-Plane Shear and Overturning**
- **Wall Connections**
- **Research Needs**

# Tilt-Up Construction

## Tilt-Up Construction

- **Site Cast Precast**
- **Panels size**
- **20 ft – 90+ ft clear height**
- **20 ft – 35+ ft width**
- **Flexible diaphragm structures**


Clayco / Concrete Strategies Construction

# Class A Architectural

Osburn Contractors

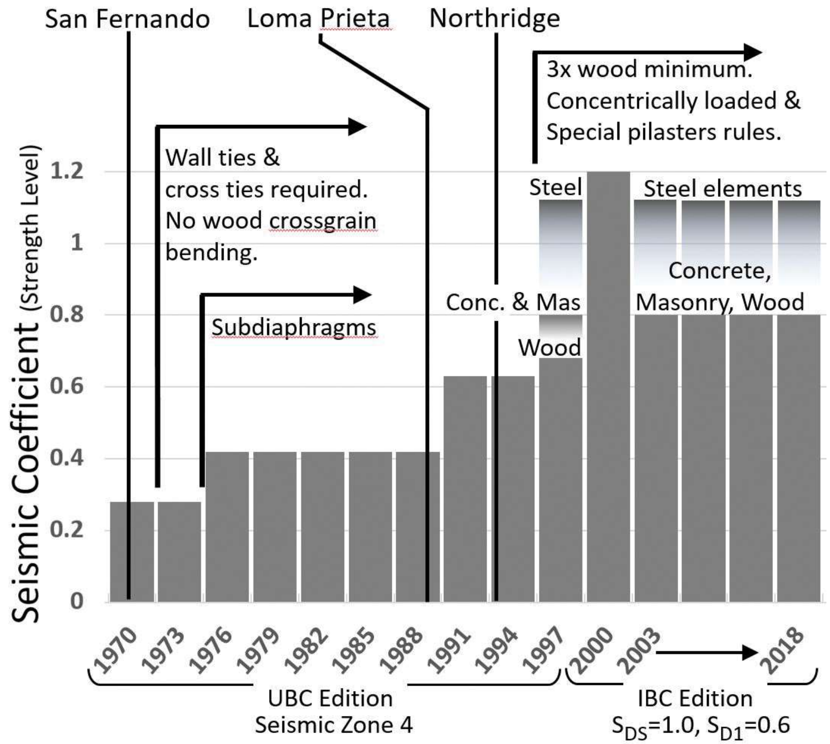
# Industrial

Southern Concrete

## Tilt-Up Construction

# Industrial

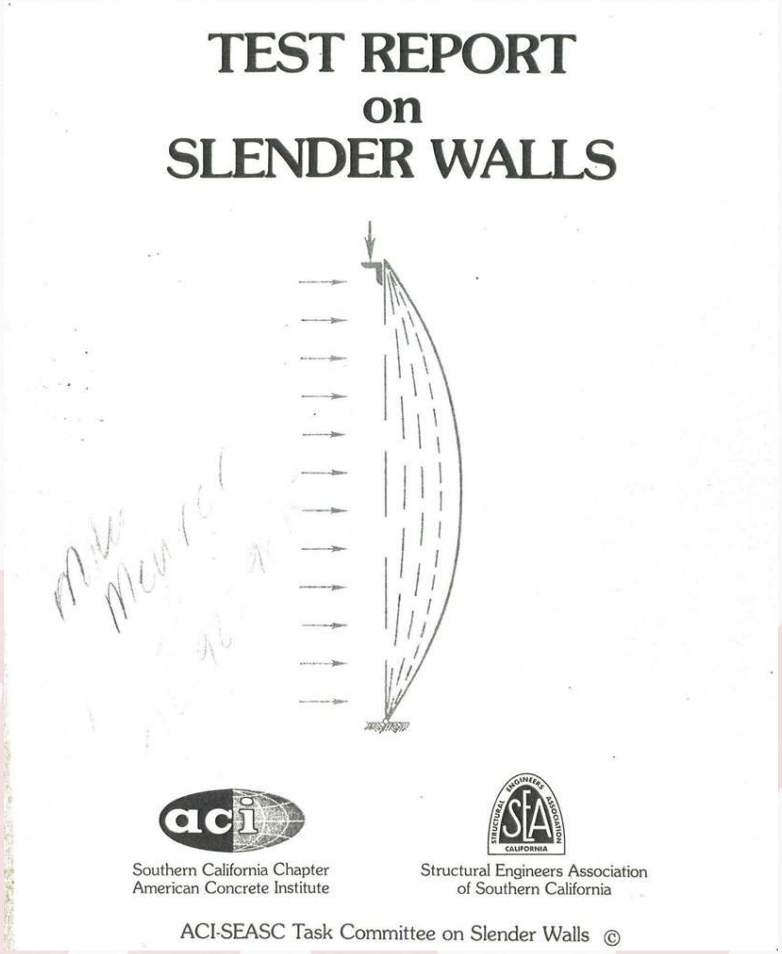
Interchange Industrial




Evolution of provisions for anchorage to flexible diaphragms (Lawson et al., 2018)

- **ASCE 7-22 Rigid Wall Flexible Diaphragms Alternative Procedure**

- **Tilt-up panels designed per** ACI 318-19 Section 11.8**. Largely defines the required vertical reinforcement.**
- **Based on research conducted in the 1980s (Green Book).**
- **Deviating from these recommendations may lead to unsafe designs (**Technote PRC-551.3-21**).**



ACI 318-19 Section 18.2 **provides guidance for designing three categories of walls applicable to tilt up construction**

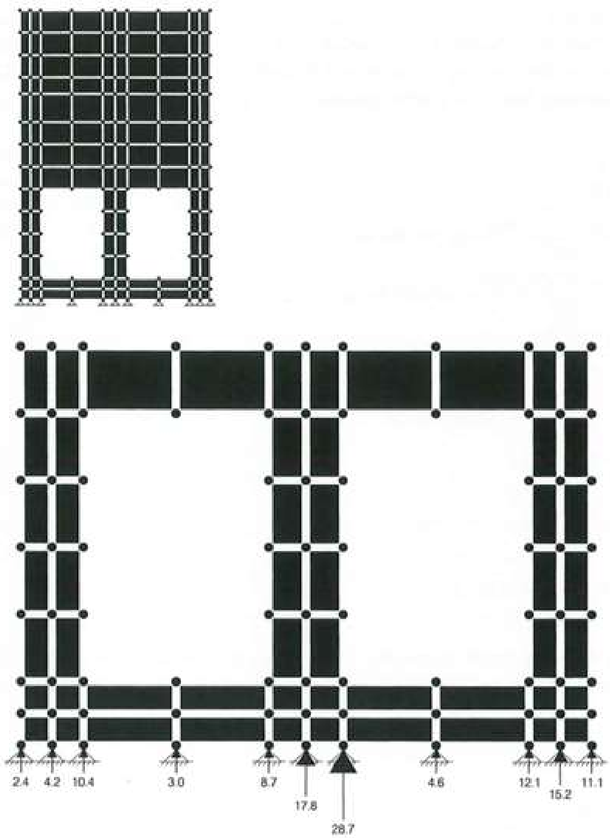
- **Intermediate Precast Shear Walls that satisfy** ACI 318-19 section 18.5 **(best fit to tilt-up walls per SEAOC blue book).**
- **Special Structural Walls: that satisfy** ACI 318-19 18.2.3-18.2.8 and 18.10**.**
- **Special Structural Walls constructed using precast concrete: that satisfy** ACI 318-19 18.2.3-18.2.8 and 18.11**.**

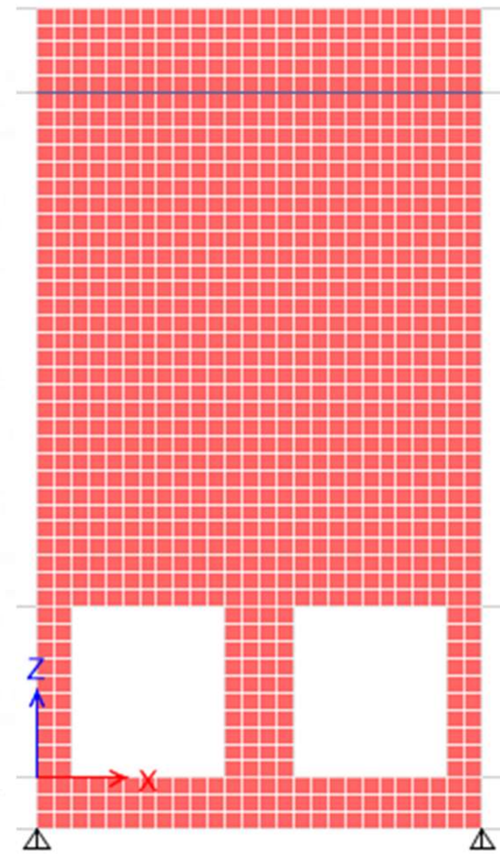
***Ordinary Precast Shear Walls: Do not apply to SDC D-F.**

- **Tilt-up walls must also be designed to resist the in-plane forces transferred from roof and floor diaphragms.**
- **The Response Modification Factor (R) used for this analysis is 4 (intermediate precast shear wall).**
- **The Canadian Building Code restricts the R to 2.0 because tilt-up walls are considered non-ductile (CSA A23.3:19).**
- **Special consideration must be given to piers around openings.**

# Base Shear – Equivalent static force procedure

- **Seismic base shear is most commonly calculated using an** equivalent static force procedure.
- Linear static **models used for panel behavior with complex geometry.**



# Panel Design Procedures: Detailing Challenges in Seismic Areas

**Table R18.10.1—Governing design provisions for vertical wall segments**[1]

| Clear height of vertical wall segment/length of vertical wall segment, $(h_w/\ell_w)$ | Length of vertical wall segment/wall thickness $(\ell_w/b_w)$ | | |
|---|---|---|---|
| | $(\ell_w/b_w) \leq 2.5$ | $2.5 < (\ell_w/b_w) \leq 6.0$ | $(\ell_w/b_w) > 6.0$ |
| $h_w/\ell_w < 2.0$ | Wall | Wall | Wall |
| $h_w/\ell_w \geq 2.0$ | Wall pier required to satisfy specified column design requirements; refer to 18.10.8.1 | Wall pier required to satisfy specified column design requirements or alternative requirements; refer to 18.10.8.1 | Wall |

[1] $h_w$ is the clear height, $\ell_w$ is the horizontal length, and $b_w$ is the width of the web of the wall segment.

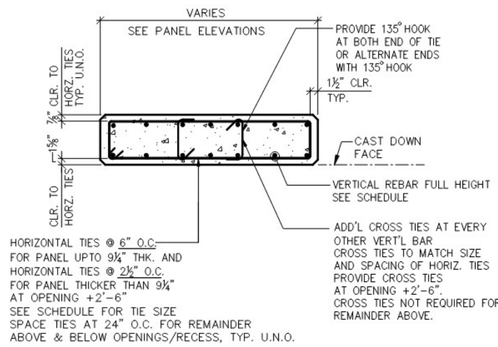
Most Common Panel Types in Tilt Up Construction

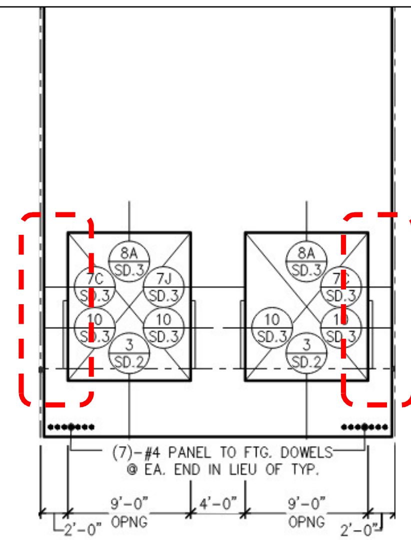
$b_w$ = 10”
$l_w$ = 2’-0”
$h_w$ = 10’-0”

$l_w / b_w$ = 2.4

$h_w / l_w$ = 5

**Column Condition**



## Intense Detailing for the Column Condition

**18.10.8** *Wall piers*

**18.10.8.1** Wall piers shall satisfy the special moment frame requirements for columns of 18.7.4, 18.7.5, and 18.7.6, with joint faces taken as the top and bottom of the clear height of

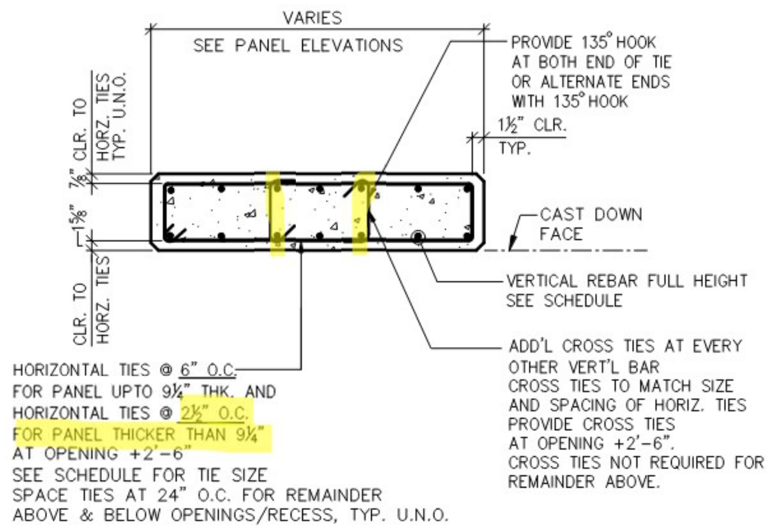
(e) Reinforcement shall be arranged such that the spacing $h_x$ of longitudinal bars laterally supported by the corner of a crosstie or hoop leg shall not exceed 14 in. around the perimeter of the column.

**18.7.5.3** Spacing of transverse reinforcement shall not exceed the smallest of (a) through (c):

(a) One-fourth of the minimum column dimension
(b) Six times the diameter of the smallest longitudinal bar
(c) $s_o$, as calculated by:

$$s_o = 4 + \left(\frac{14 - h_x}{3}\right) \qquad (18.7.5.3)$$

The value of $s_o$ from Eq. (18.7.5.3) shall not exceed 6 in. and need not be taken less than 4 in.



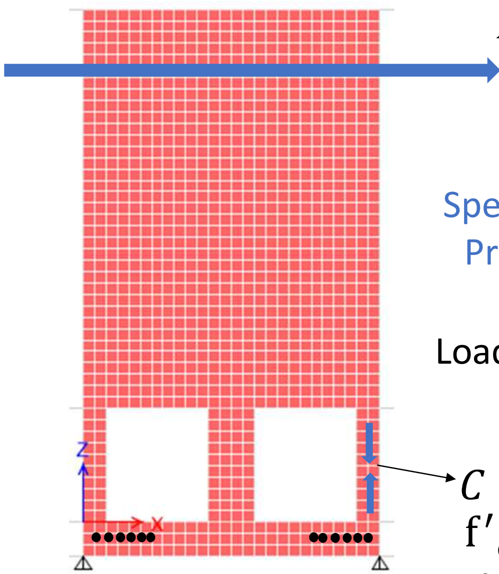
## Panel Design Procedures: Detailing Challenges in Seismic Areas (cont.)

Dock Door Panel (Max Load Transferred Through Diaphragm)

$$E_{diaphgram} = 115.4\ kips$$

$$E_{panel(R=1)} = 75.7\ kips$$

$$E_{total} = E_{diaphgram} + E_{panel(R=1)} = 191.1\ kips$$




Special boundary elements not required per Section 18.10.6.5

Proportion transverse reinforcement per columns in special moment frame

Load Combo: $(1.2 + 0.2Sds)D + E_{total}$

$$C = 315.63\ kips$$

$$f'_c = 4400\ psi$$

$$A_g = 1.667\ ft^2$$

# Panel Design Procedures: Detailing Challenges in Seismic Areas (cont.)

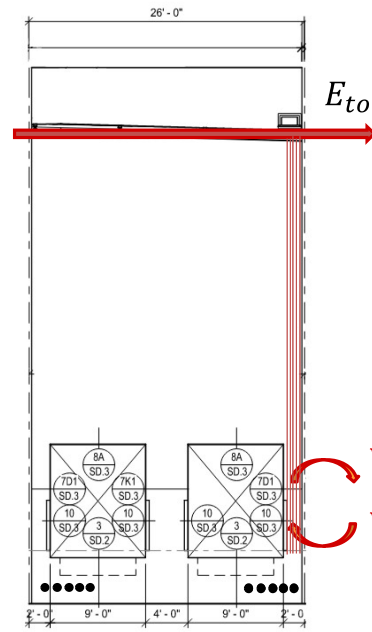
## ACI 18.7 – Columns of Special Moment Frames

**18.7.6.1.1** The design shear force $V_e$ shall be calculated from considering the maximum forces that can be generated at the faces of the joints at each end of the column. These joint forces shall be calculated using the maximum probable flexural strengths, $M_{pr}$, at each end of the column associated with the range of factored axial forces, $P_u$, acting on the column. The column shears need not exceed those calculated from joint strengths based on $M_{pr}$ of the beams framing into the joint. In no case shall $V_e$ be less than the factored shear calculated by analysis of the structure.

$V_{umax} = V_{pr} = 21.03\ kips$

$\varphi V_n = 79.48\ kips$ (#3 @ 2.5" O.C.)

$\varphi V_n = 33.12\ kips$ (#3 @ 6" O.C.)




$E_{total} = 191.1 kips$

$V_{pr} = \frac{2M_{pr}}{h}$

$V_{pr} = 21.03$ kips

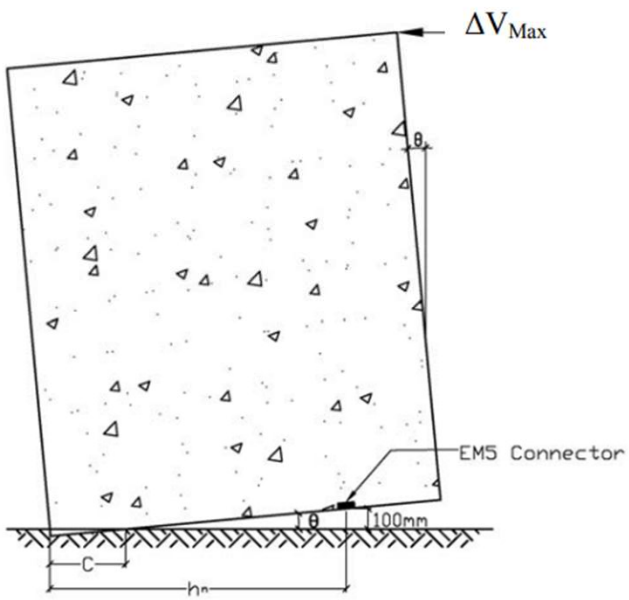
# Base Shear – Equivalent static force procedure

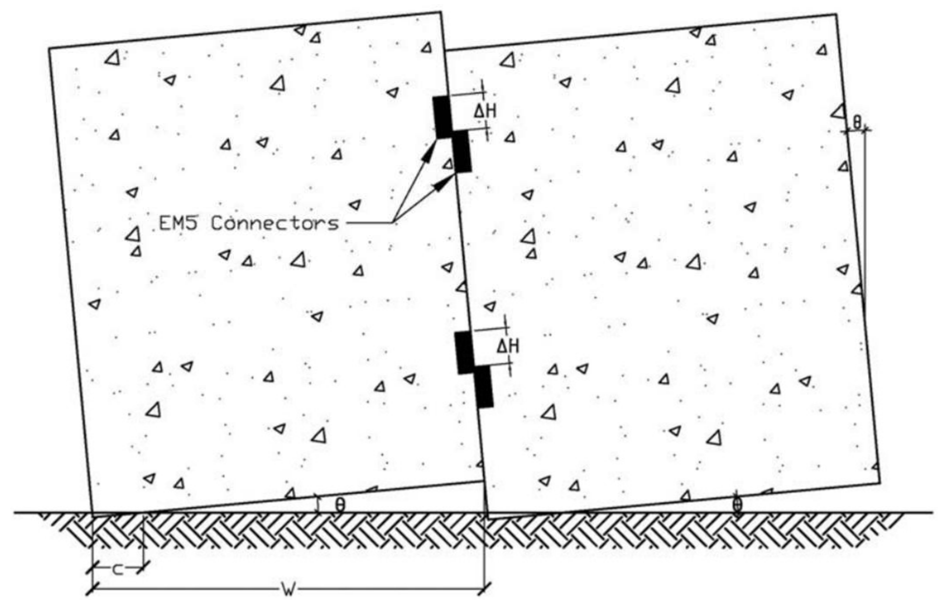
Requirements Per the National Building Code of Canada 2015, and CSA A23.3-14:

- **Seismic base shear is most commonly calculated using an** equivalent static force procedure
- **Based on research from UBC Tilt-Up walls are classified as limited ductility and get an effective R = 2.0 while enforcing a** rocking mechanism
- Dynamic Analysis is not used **as a model that incorporates rocking panel response would be highly non-linear and difficult to model.**

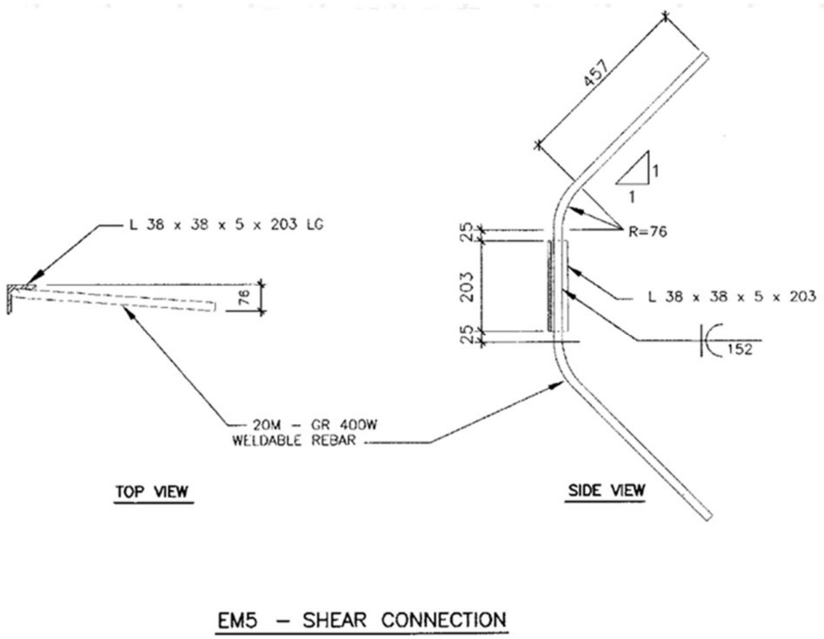
# Permitted Ductile limit states & limitations

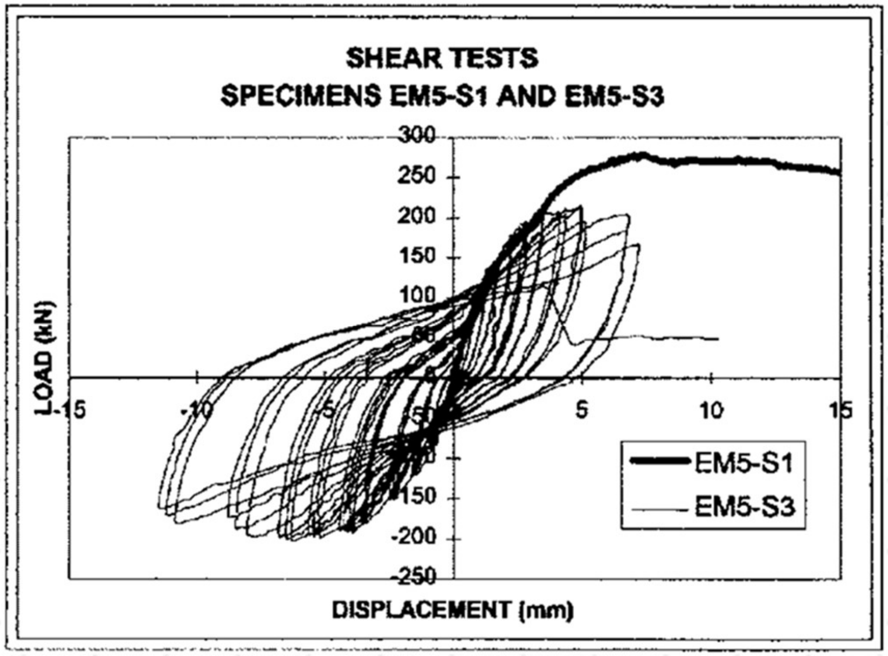
- **Based on Tellier (2013) and others at University of British Columbia, a rocking limit state is the most feasible energy dissipation mechanism (as opposed to panel sliding or non-ductile panel failure).**



# Approved ductile panel-to-panel connector – EM5



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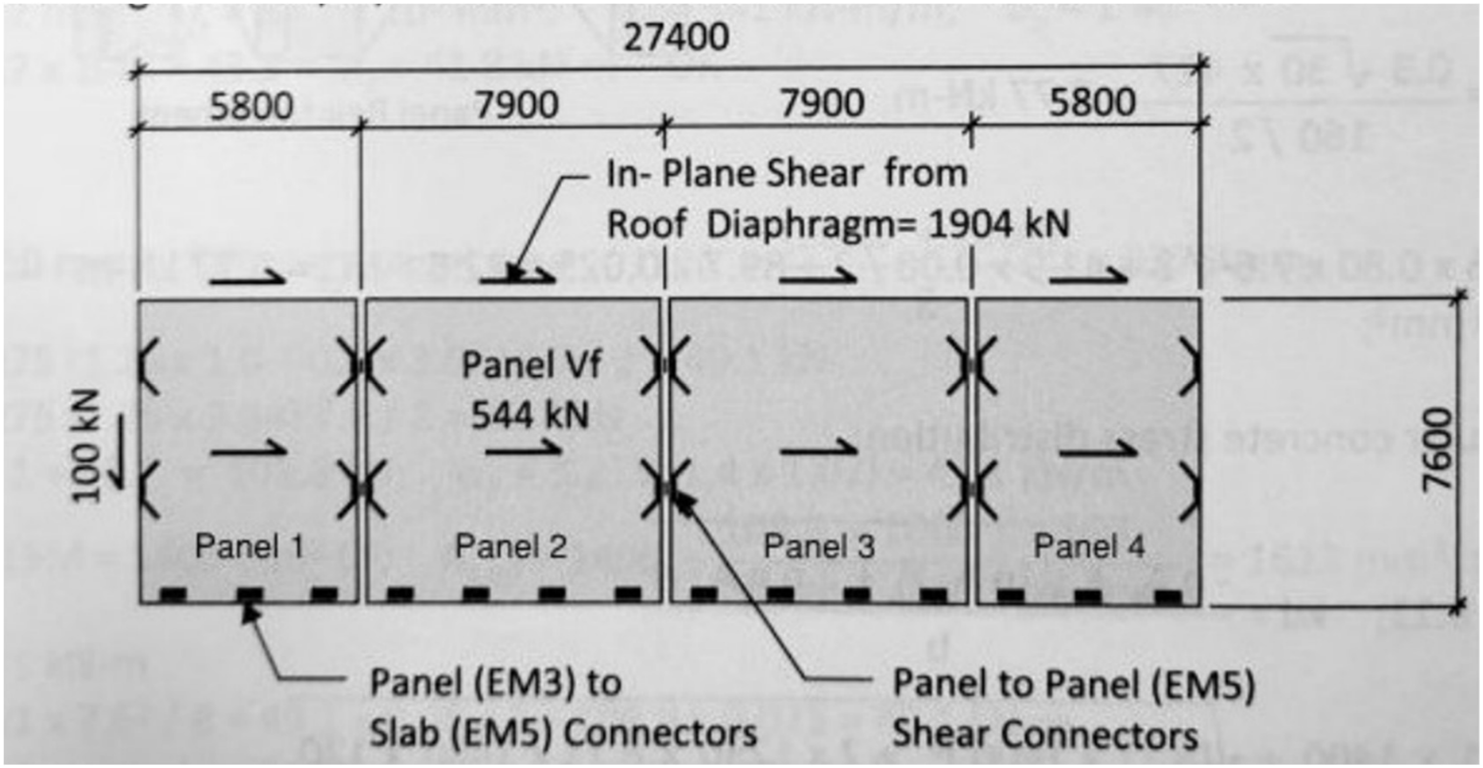
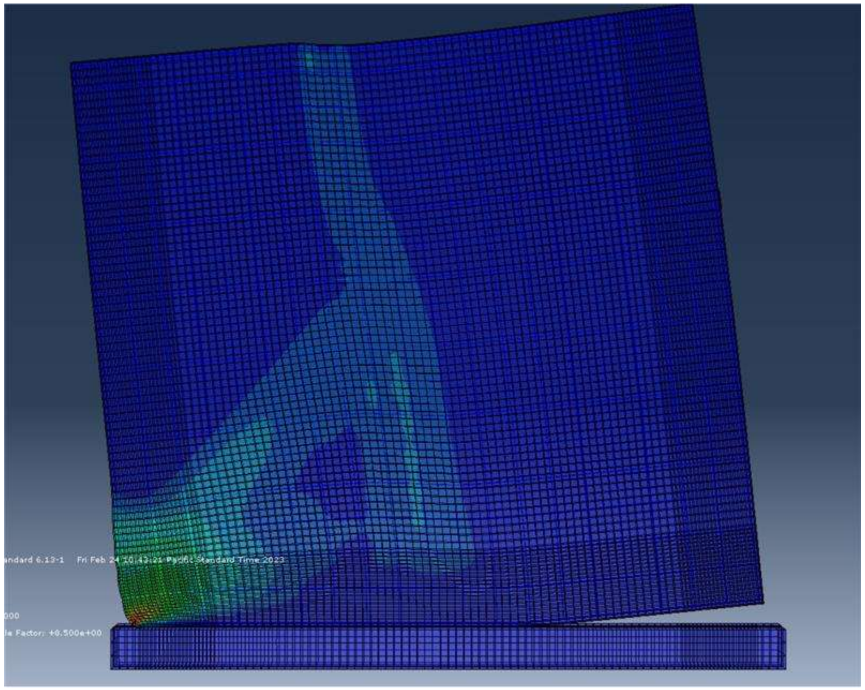
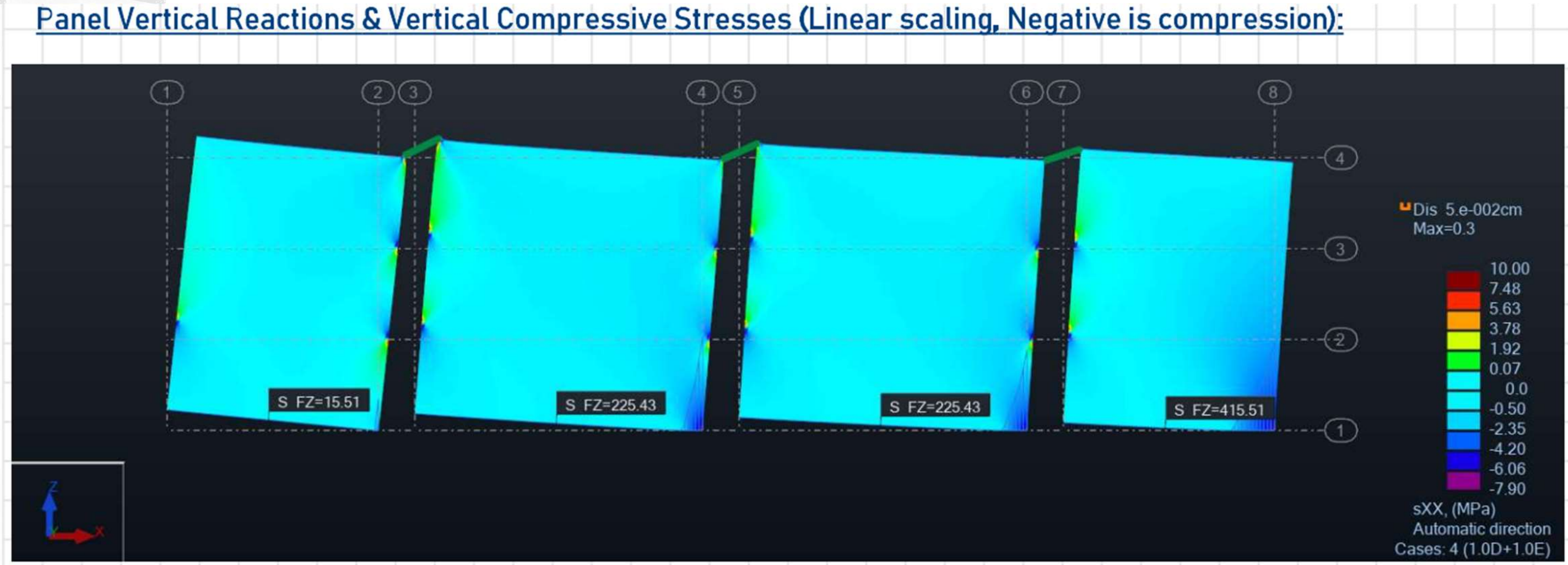


## Issues with overturning as ductile dissipation mechanism

- **The extent of the compression zone isn't easily defined.**
- **The degree of lateral support provided by panel to panel connectors is largely undefined.**



# Compression at panel edges during rocking



## Christchurch 2011 and Chile 2010

- Several groups reported good performance of tilt up structures.
- Rocking behavior observed with spalling of corners (Urmson and Toulmin 2012), backed up by Chile earthquake on thin walls (Adebar 2013).
- Henry and Ingham (2011) found many instances of poor connection performance.

Adebar (2013)

Henry & Ingham (2012)

## SEAOC Blue Book

- Is the selected method of distributing in-plane shears critical to shear wall performance?
- Can a simplified method of shear distribution achieve acceptable results?
- Are deformation limits for wall anchorage systems necessary and how should they be set?
- As wall anchorage is eliminated as the weak element of tilt-up structures, will the mode of failure simply transfer to another vulnerable portion of the system?

## Additional needs

- Development of RWFD analysis methods has simplified analyses for complex panel geometries, rocking behavior and non-diaphragm connections. None of which have experimentally been observed.
- Panel/connection interaction with foundation is a concern.
- The performance of panels with large openings is not well understood, particularly if rocking is enforced
- Are prequalified connections needed and how to define their performance and limits?
- Are seismic coefficients appropriate (US vs CAN)?

# References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