Mass Timber Rocking Wall Systems and Design

Reid Zimmerman, PE, SE
Technical Director | KPFF Portland

WoodWorks Seminar: State-of-Art Mass Timber Seismic Design and 10-Story NHERI Shake Table Test
May 19, 2023

Disclaimer: This presentation was developed by a third party and is not funded by WoodWorks or the Softwood Lumber Board.

kpff

1

Outline

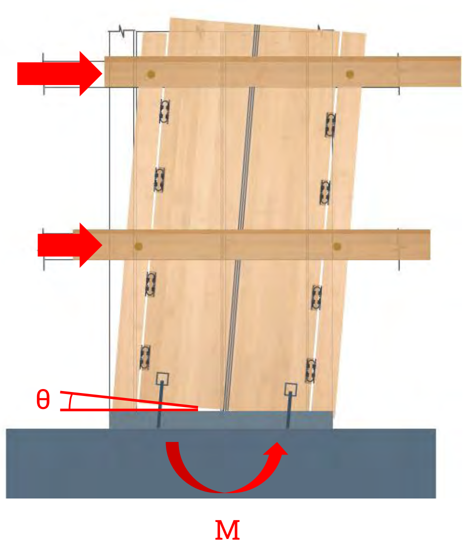
- Introduction to mass timber rocking walls
- Current state of research and project applications
- Design procedures and examples
- What comes next?

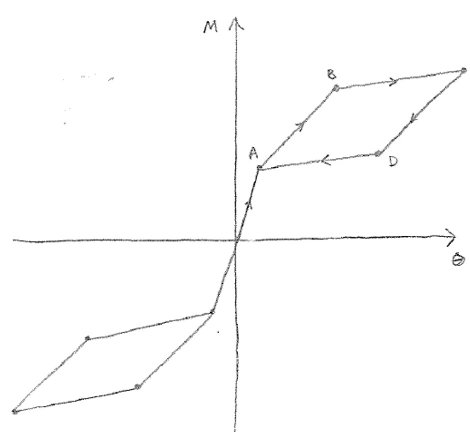
2

2

Introduction to Mass Timber Rocking Walls

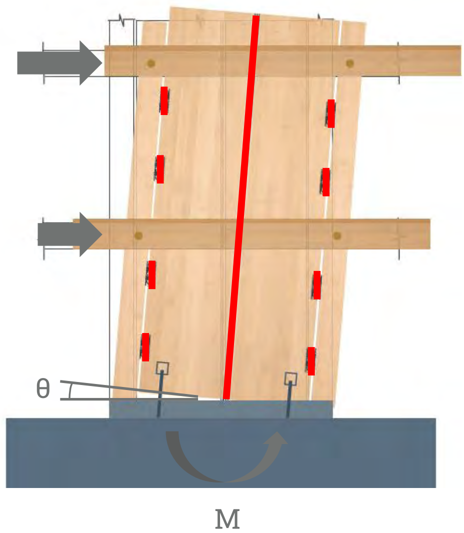
Introduction to Mass Timber Rocking Walls

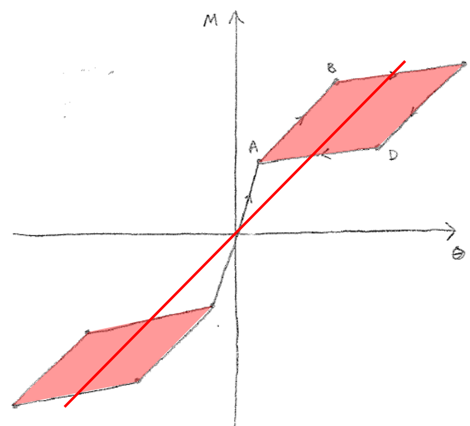




Left figure from Framework project

Introduction to Mass Timber Rocking Walls





Left figure from Framework project

5

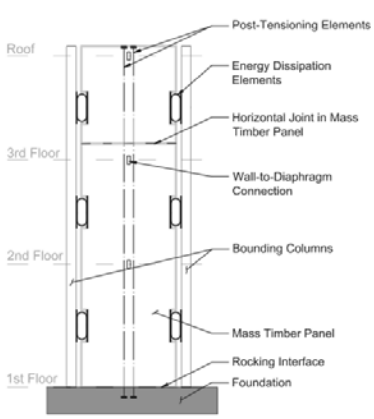
5

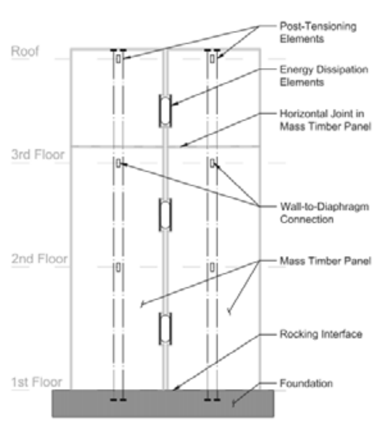
Introduction to Mass Timber Rocking Walls

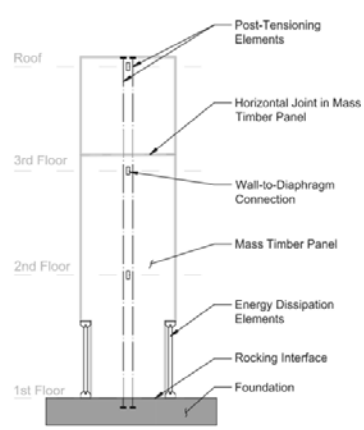
Single Panel w/ Bounding Column Configuration

Coupled Panel Configuration

Singe Panel Configuration







Figures from Busch et al (2022)

6

6

Introduction to Mass Timber Rocking Walls

Single Panel w/ Bounding Column Configuration

Coupled Panel Configuration

Singe Panel Configuration

Left figure of TallWood 10-story specimen, middle figure of Tallwood 2-story specimen, right figure of OSU 3-story specimen

7

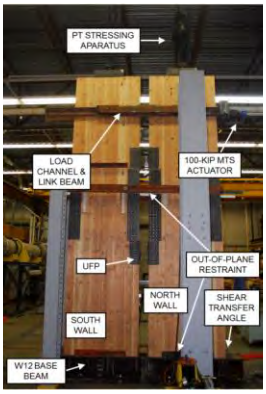
7

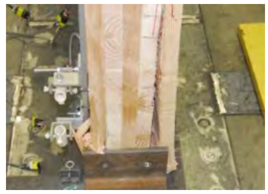
Current State of Research and Project Applications

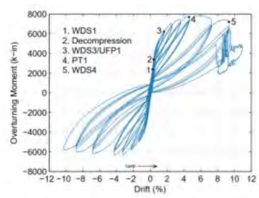
8

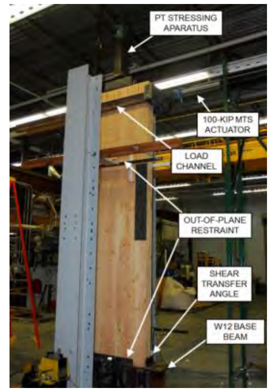
8

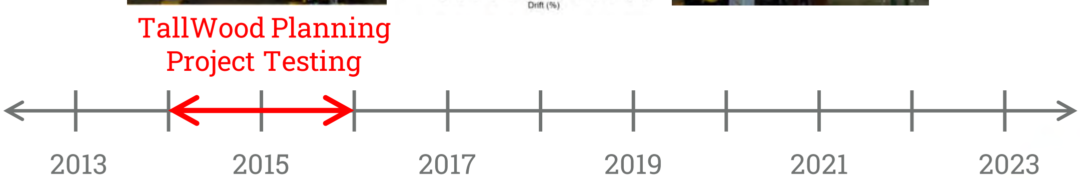
Current State of Research







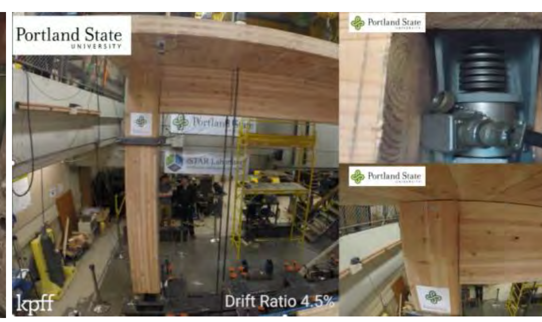




Figures from Ganey at al. (2017)

9

Current State of Research



Photos from Framework testing at Oregon State University and Portland State University

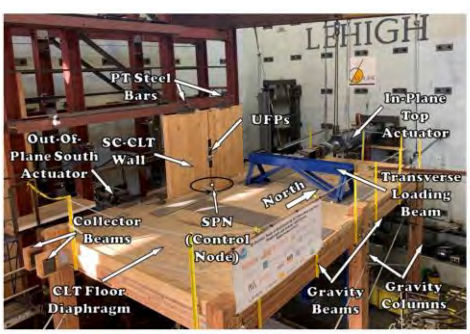
10

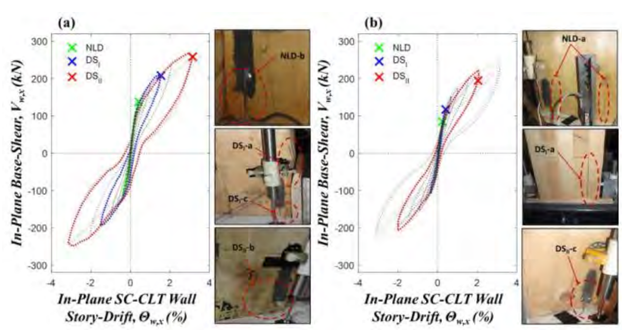
Current State of Research

TallWood 2-Story
Building Specimen Testing

2013 2015 2017 2019 2021 2023

Current State of Research



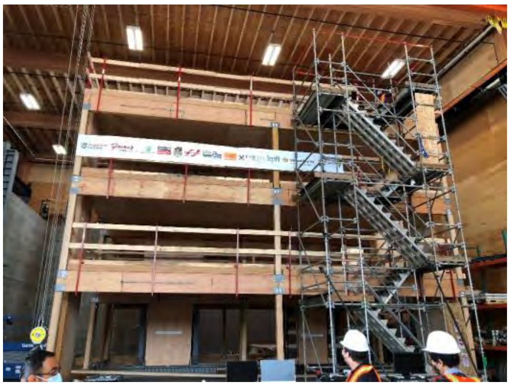


TallWood Biaxial
Testing

2013 2015 2017 2019 2021 2023

Figures from Amer (2019)

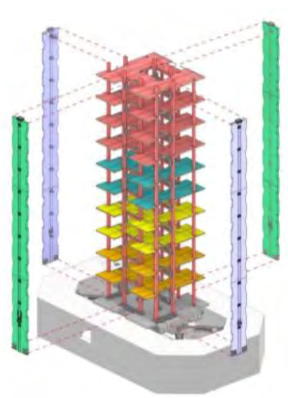
Current State of Research



OSU 3-story Building Specimen Testing

2013 2015 2017 2019 2021 2023

Current State of Research

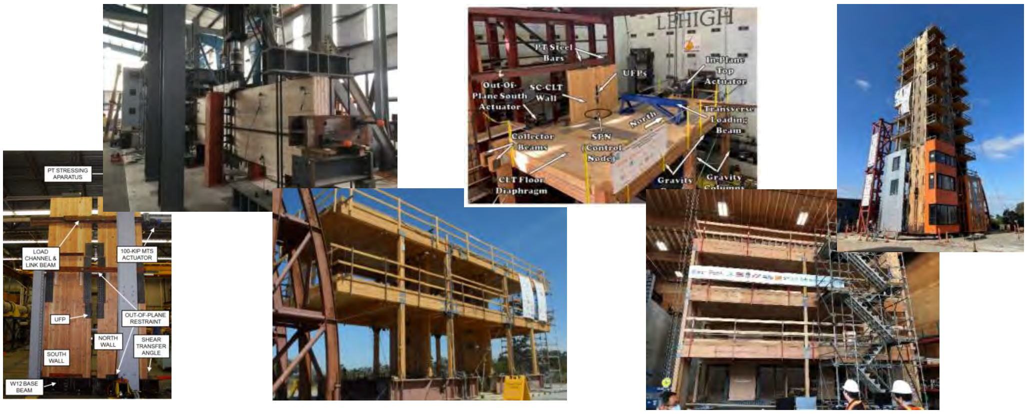


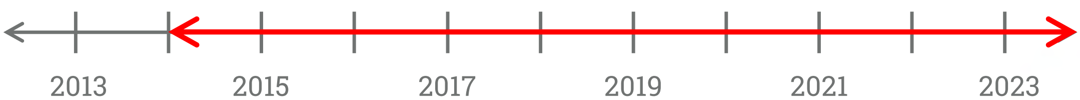
TallWood 10-story Building Specimen Testing

2013 2015 2017 2019 2021 2023

Left figures courtesy of LEVER Architecture

Current State of Research



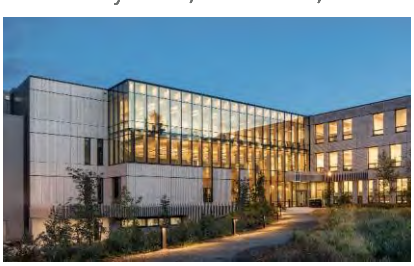


15

15

Current Project Applications in the U.S.

Oregon State University
Peavy Hall, Corvallis, OR



Framework
Portland, OR

Killingsworth
Portland, OR

Center and right renderings courtesy of LEVER Architecture

16

16

Design Procedures and Examples

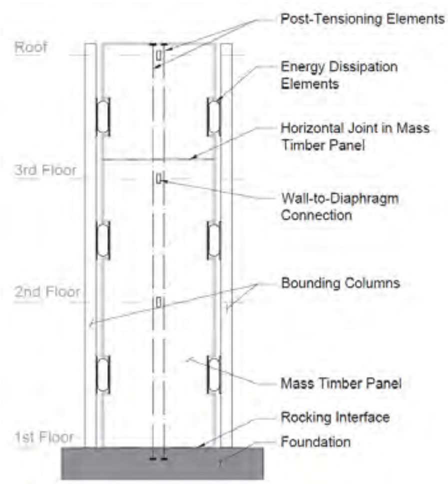
17

17

18

Prescriptive Design Procedure

- Either Equivalent Lateral Force or Modal Response Spectrum Analysis acceptable in state-of-the-practice software
- Majority of checks performed at DBE; several additional checks at MCE_R
- Based on proposed AWC SDPWS Appendix

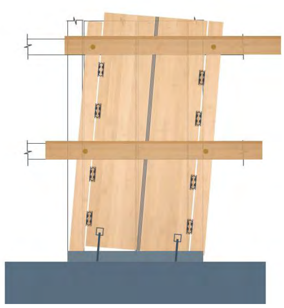


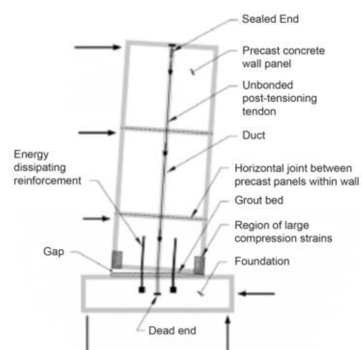
Right figure from Busch et al (2022)

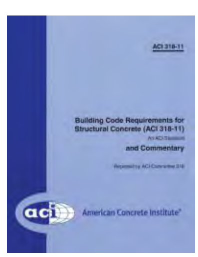
19

19

Prescriptive Design Procedure



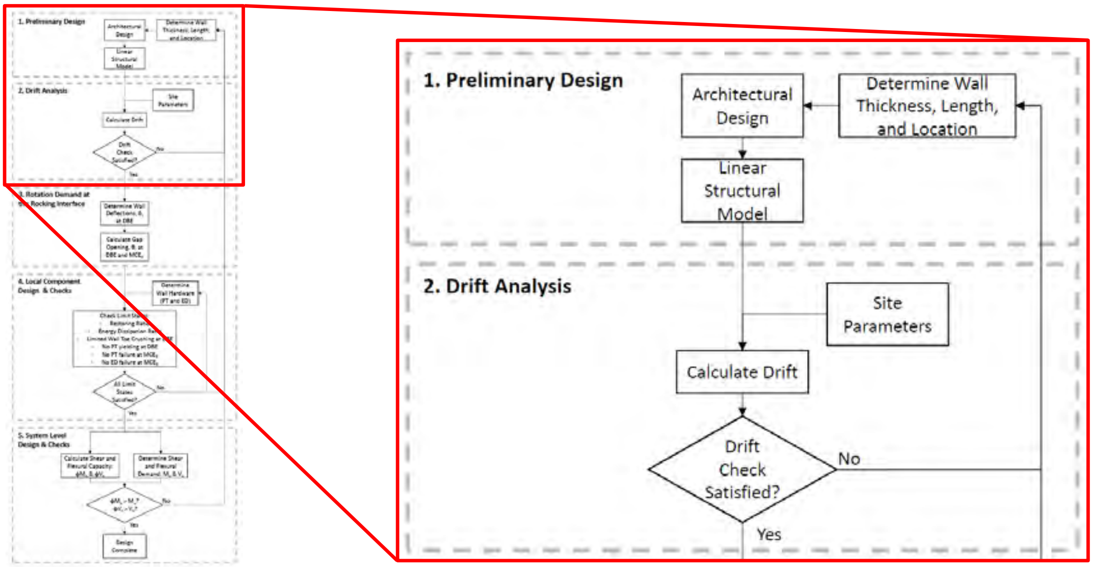




20

20

Prescriptive Design Procedure

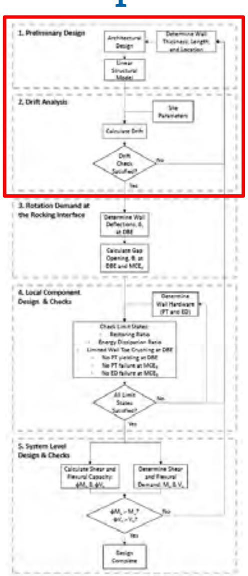


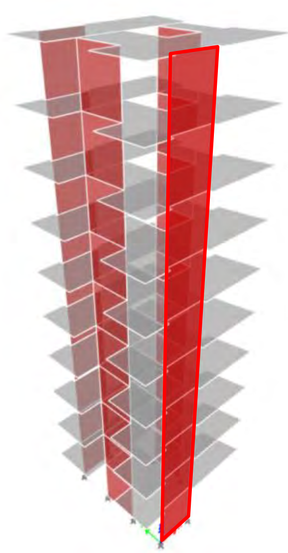
Figures from Busch et al (2022)

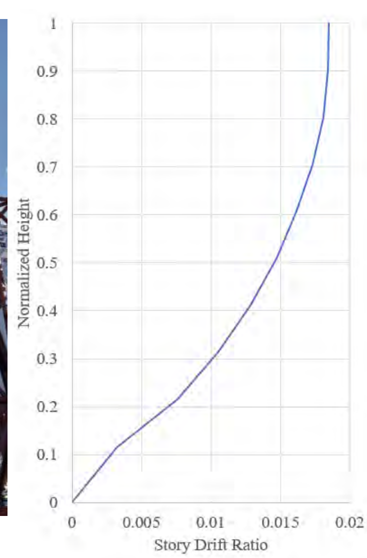
21

21

Prescriptive Design Procedure







Left figure from Busch et al (2022)

22

22

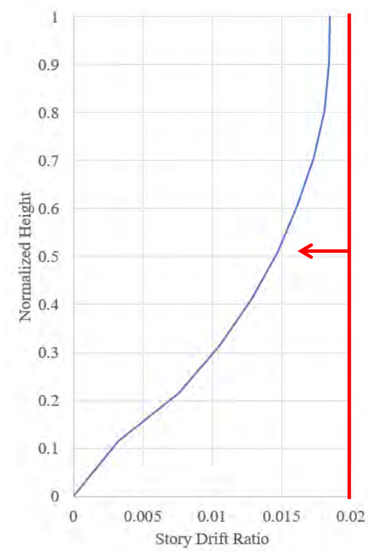
Prescriptive Design Procedure

From ASCE 7-16 Chapter 12

Table 12.12-1 Allowable Story Drift, Δ_a[a,b]

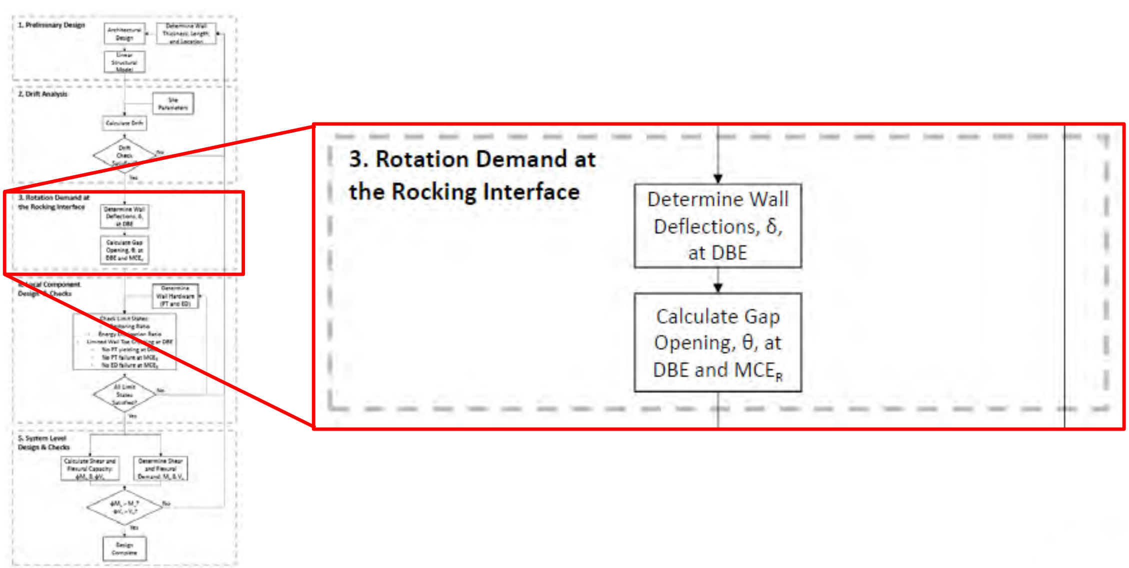
Structure	Risk Category I or II	Risk Category III	Risk Category IV
Structures, other than masonry shear wall structures, four stories or less above the base as defined in Section 11.2, with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts	$0.025h_{sx}$[c]	$0.020h_{sx}$	$0.015h_{sx}$
Masonry cantilever shear wall structures[d]	$0.010h_{sx}$	$0.010h_{sx}$	$0.010h_{sx}$
Other masonry shear wall structures	$0.007h_{sx}$	$0.007h_{sx}$	$0.007h_{sx}$
All other structures	$0.020h_{sx}$	$0.015h_{sx}$	$0.010h_{sx}$

[a] h_{sx} is the story height below level x.
[b] For seismic force-resisting systems solely comprising moment frames in Seismic Design Categories D, E, and F, the allowable story drift shall comply with the requirements of Section 12.12.1.1.
[c] There shall be no drift limit for single-story structures with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts. The structure separation requirement of Section 12.12.3 is not waived.
[d] Structures in which the basic structural system consists of masonry shear walls designed as vertical elements cantilevered from their base or foundation support that are so constructed that moment transfer between shear walls (coupling) is negligible.



23

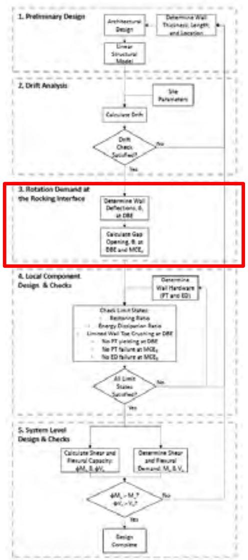
Prescriptive Design Procedure

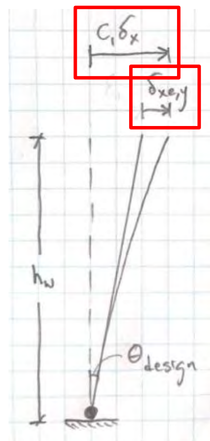


Figures from Busch et al (2022)

24

Prescriptive Design Procedure





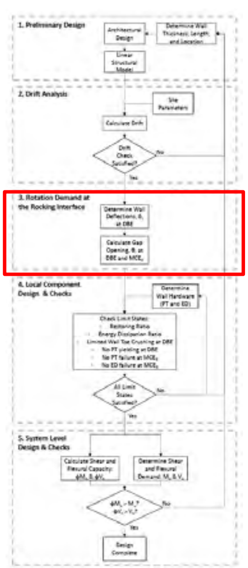
$$\theta_{design} = \frac{C_1 * \delta_x - \delta_{xe,y}}{h_w}$$

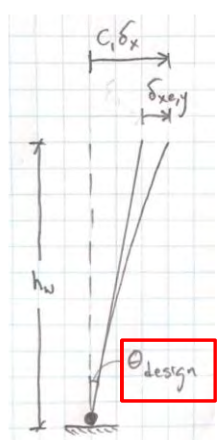
Left figure from Busch et al (2022)

25

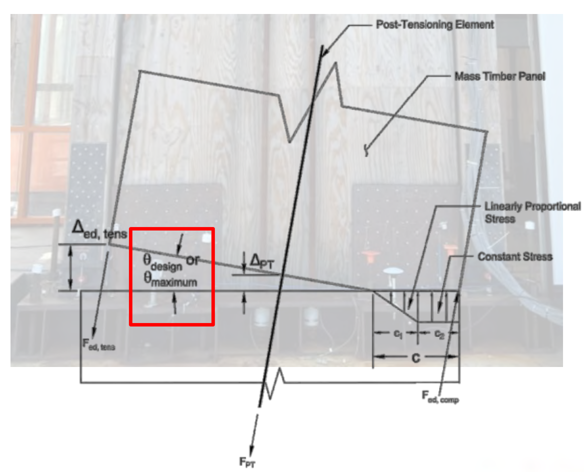
25

Prescriptive Design Procedure





$$\theta_{design} = \frac{C_1 * \delta_x - \delta_{xe,y}}{h_w}$$

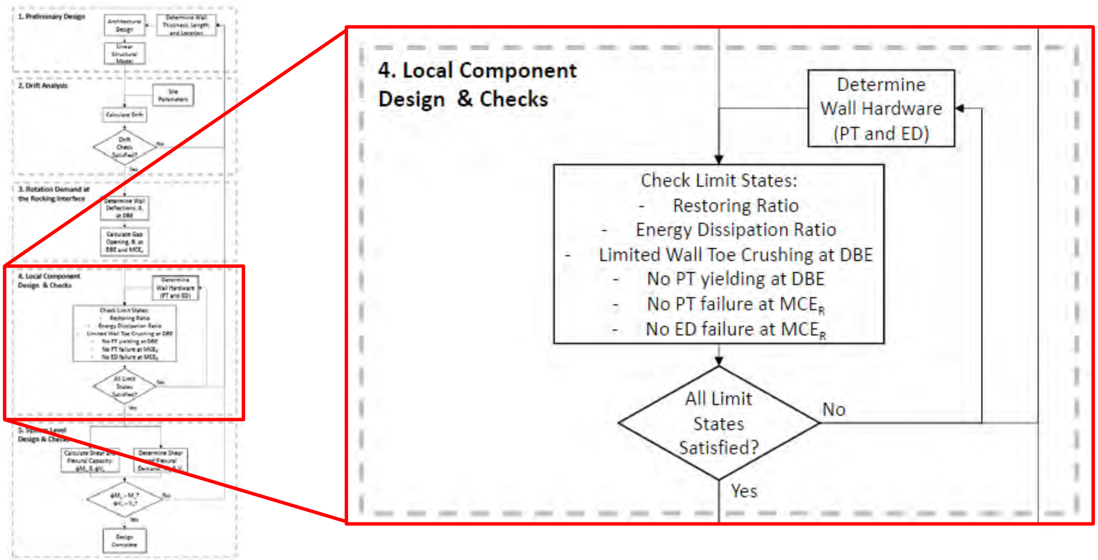


Left and right figures from Busch et al (2022)

26

26

Prescriptive Design Procedure

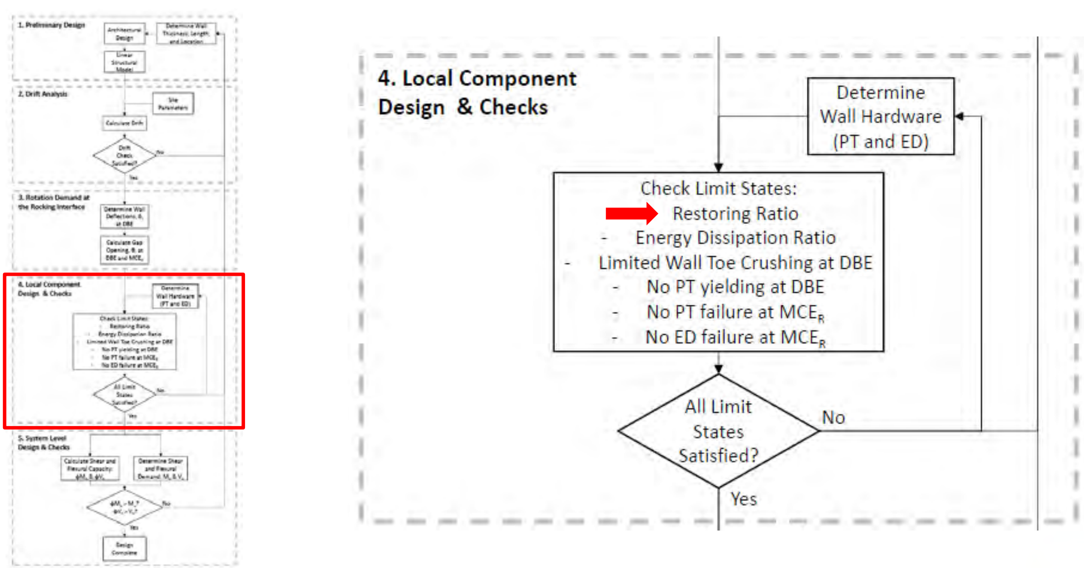


Figures from Busch et al (2022)

27

27

Prescriptive Design Procedure

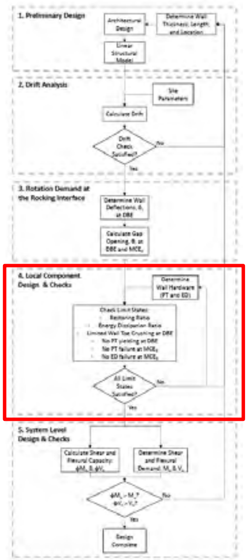


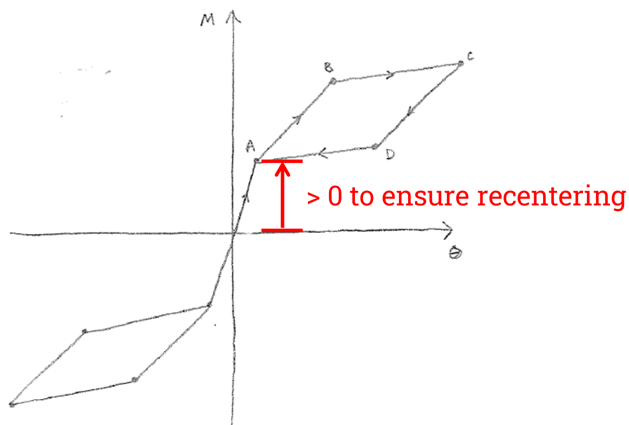
Figures from Busch et al (2022)

28

28

Prescriptive Design Procedure





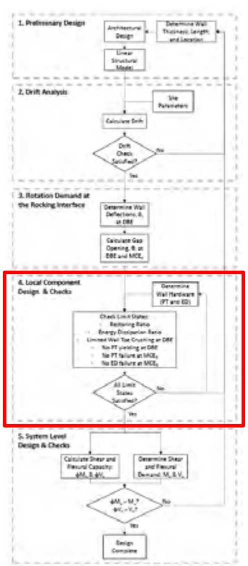
$$\sum_{j=1}^{n}\left[\left(F_{pti,j}+F_{grav,j}\right)*\frac{l_{w,j}}{2}\right]\geq\sum_{j=1}^{n}\left[F_{edu,wh,j}*l_{w,j}\right]$$

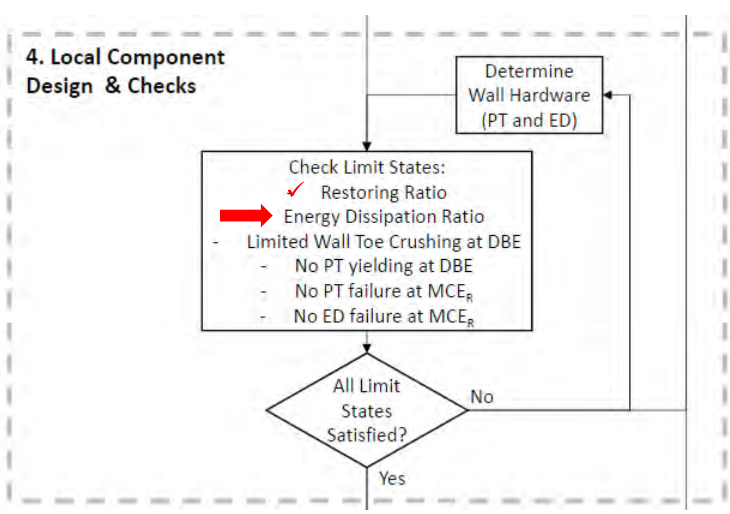
Left figure from Busch et al (2022)

29

29

Prescriptive Design Procedure



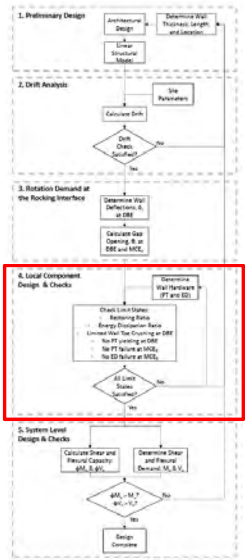


Figures from Busch et al (2022)

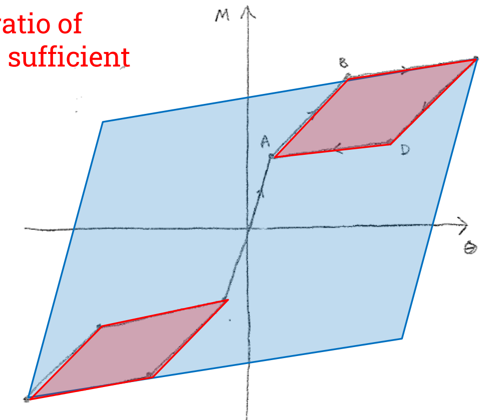
30

30

Prescriptive Design Procedure



Set minimum ratio of areas to ensure sufficient damping



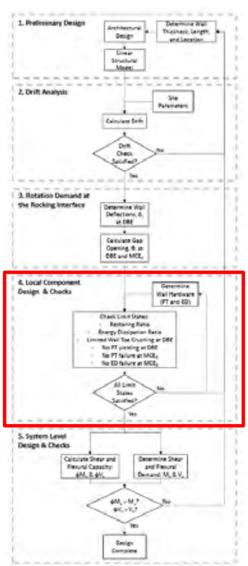
$$\sum_{j=1}^{n} M_{ed,rm,j} \geq \frac{1}{4} * \sum_{j=1}^{n} M_{n,rm,j}$$

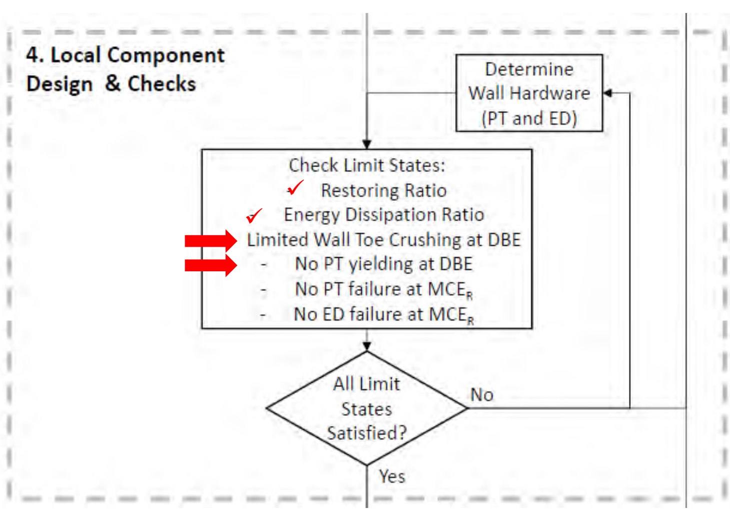
Left figure from Busch et al (2022)

31

31

Prescriptive Design Procedure



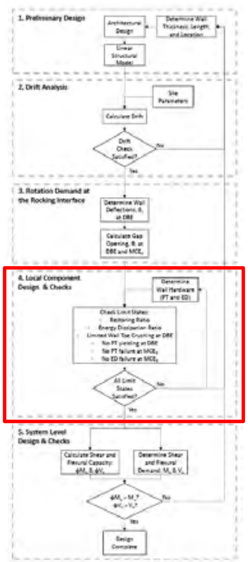


Figures from Busch et al (2022)

32

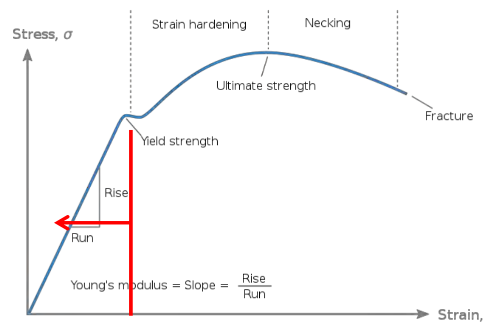
32

Prescriptive Design Procedure



Limit Wall Toe Crushing at DBE

No Post-Tensioning Yielding at DBE

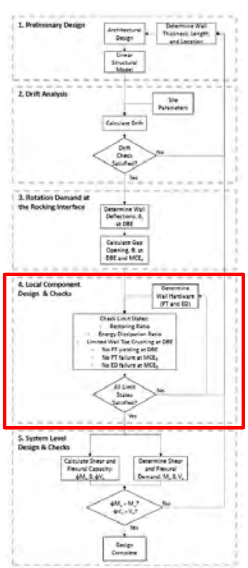


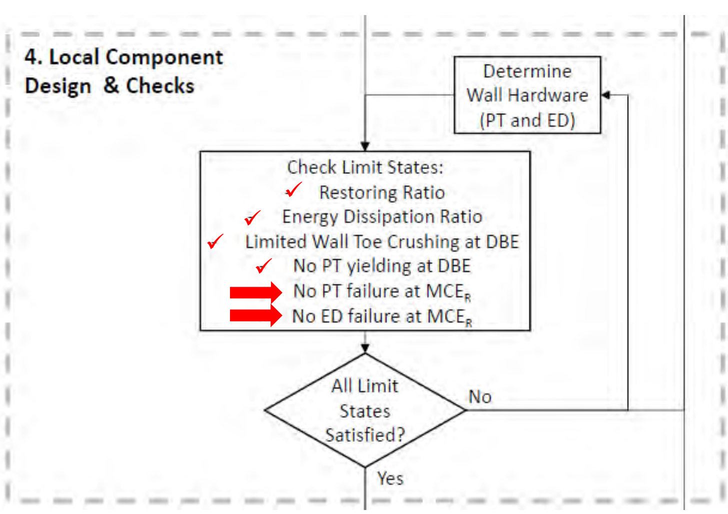
Left figure from Busch et al (2022)

33

33

Prescriptive Design Procedure



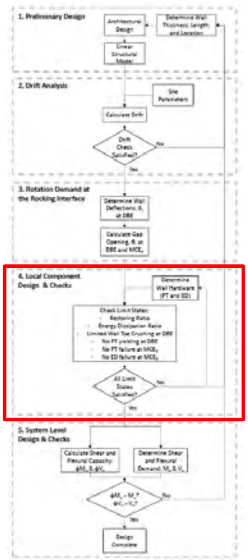


Figures from Busch et al (2022)

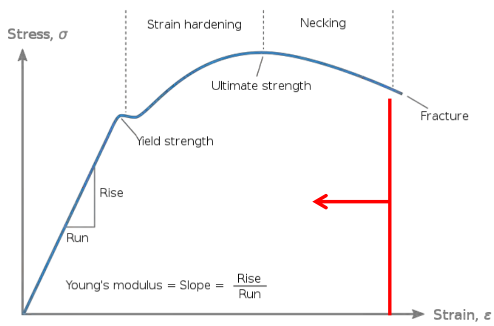
34

34

Prescriptive Design Procedure



No Post-Tensioning Failure at MCE_R



No Energy Dissipation Failure at MCE_R

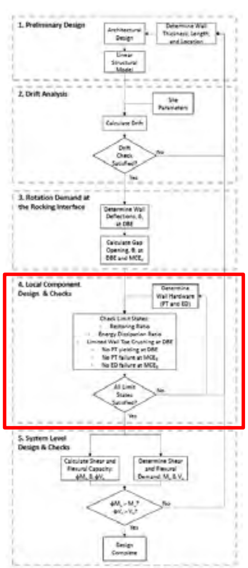
$$\theta_{max} = \frac{1.5 * C_1 * \delta_x - \delta_{xe,y}}{h_w}$$

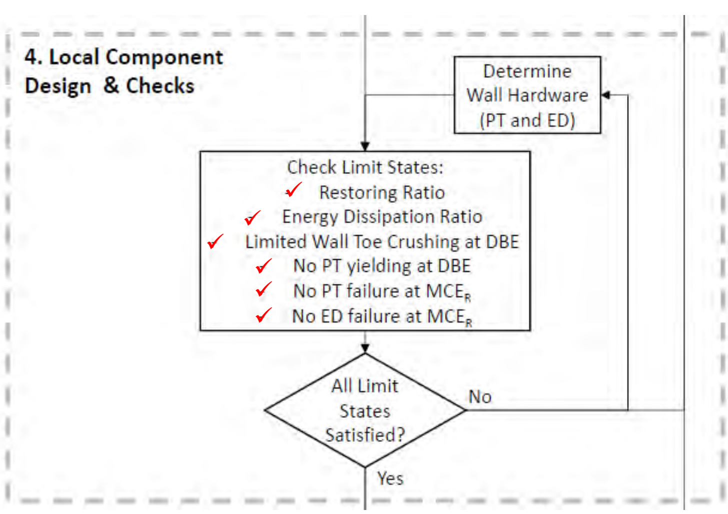
Left figure from Busch et al (2022)

35

35

Prescriptive Design Procedure



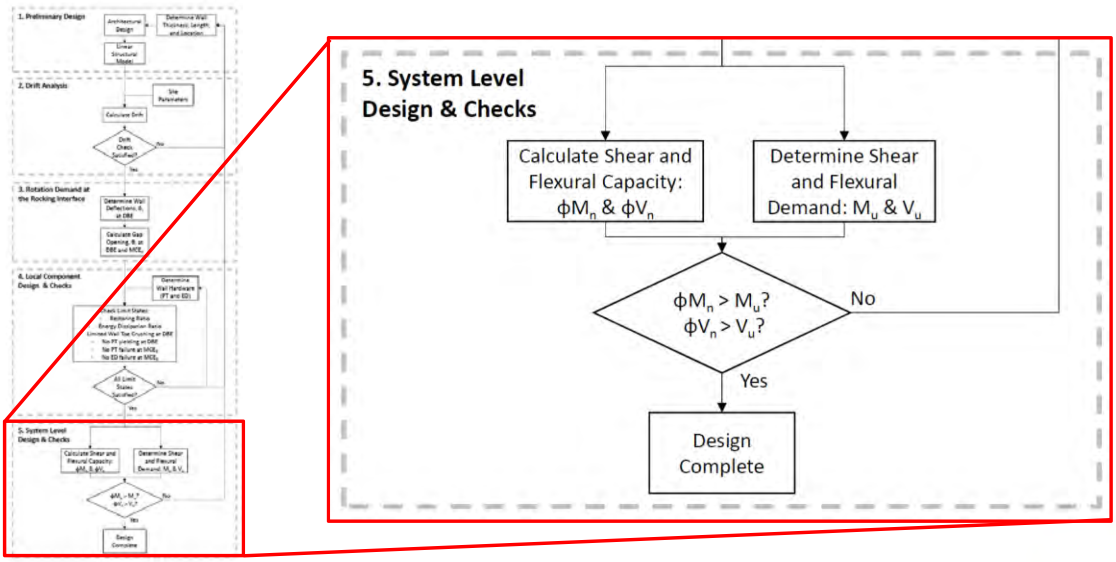


Figures from Busch et al (2022)

36

36

Prescriptive Design Procedure

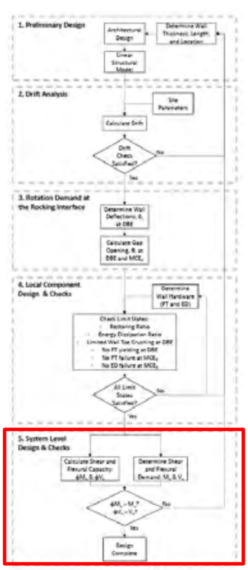


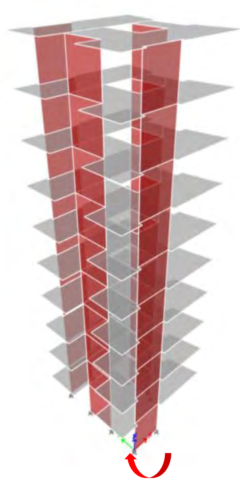
Figures from Busch et al (2022)

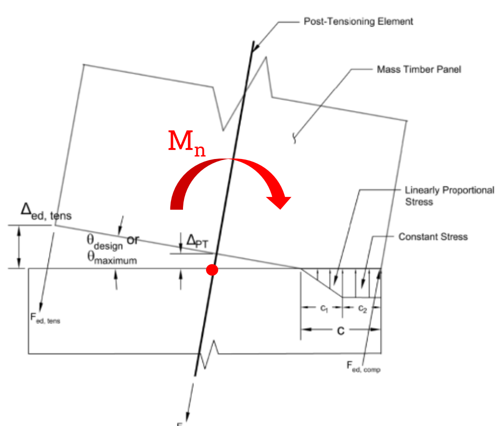
37

37

Prescriptive Design Procedure





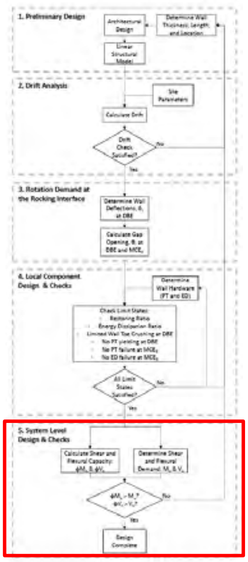


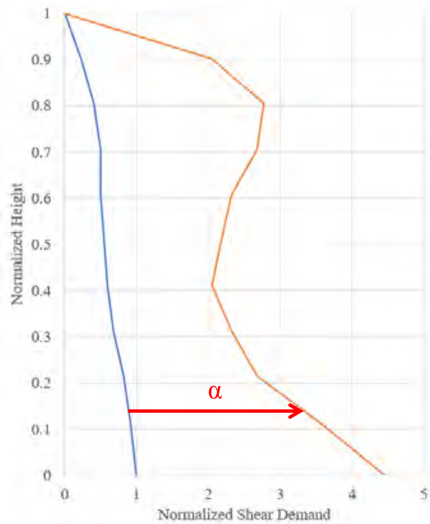
Left and right figure from Busch et al (2022)

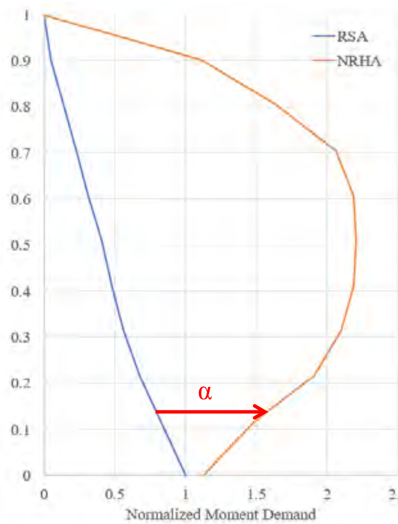
38

38

Prescriptive Design Procedure





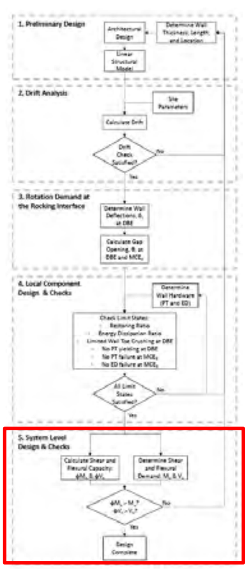


Left figure from Busch et al (2022)

39

39

Prescriptive Design Procedure



From ACI 318-19

18.10.3.1 The design shear force V_e shall be calculated by:

$$V_e = \Omega_v \omega_v V_u \leq 3V_u \quad (18.10.3.1)$$

where V_u, Ω_v, and ω_v are defined in 18.10.3.1.1, 18.10.3.1.2, and 18.10.3.1.3, respectively.

18.10.3.1.1 V_u is the shear force obtained from code lateral load analysis with factored load combinations.

18.10.3.1.2 Ω_v shall be in accordance with Table 18.10.3.1.2.

Table 18.10.3.1.2—Overstrength factor Ω_v at critical section

Condition	Ω_v	
$h_{wcs}/\ell_w > 1.5$	Greater of	M_{pr}/M_u[1]
		1.5[2]
$h_{wcs}/\ell_w \leq 1.5$	1.0	

[1] For the load combination producing the largest value of Ω_v.

[2] Unless a more detailed analysis demonstrated a smaller value, but not less than 1.0.

18.10.3.1.3 For walls with $h_{wcs}/\ell_w < 2.0$, ω_v shall be taken as 1.0. Otherwise, ω_v shall be calculated as:

$$\omega_v = 0.9 + \frac{n_s}{10} \quad n_s \leq 6$$
$$\omega_v = 1.3 + \frac{n_s}{30} \leq 1.8 \quad n_s > 6 \quad (18.10.3.1.3)$$

where n_s shall not be taken less than the quantity $0.007h_{wcs}$.

From prescriptive design provisions

C.6.3 Amplification of Forces and Moments

Amplification of forces and moments, α, in Sections C.6.4 through C.6.9 shall be determined using a rational analysis that accounts for flexural overstrength of the rocking mechanism in the fundamental mode and reduced nonlinearity in modes other than the fundamental mode. The value of α shall be calculated for each force or moment of interest and shall not be taken less than ω_b and need not be taken greater than 3.

$$\omega_b = \sum_{j=1}^{n} M_{pr,rm,j} \Big/ \sum_{j=1}^{n} M_{u,rm,j}$$

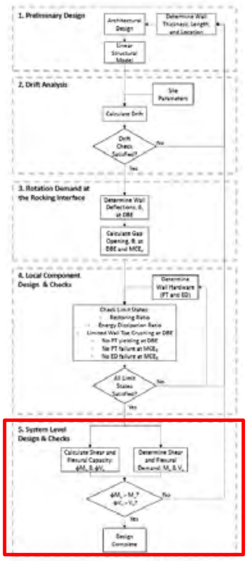
$$\alpha = \frac{\sqrt{\left(\frac{\omega_b * V/V_t * r_1}{R}\right)^2 + \Sigma_{i=2}^{n} r_i^2}}{\sqrt{\Sigma_{i=1}^{n}\left(\frac{V/V_t * r_i}{R}\right)^2}}$$

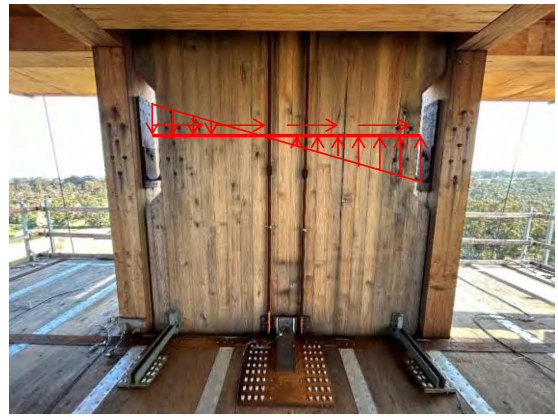
Left figure from Busch et al (2022)

40

40

Prescriptive Design Procedure





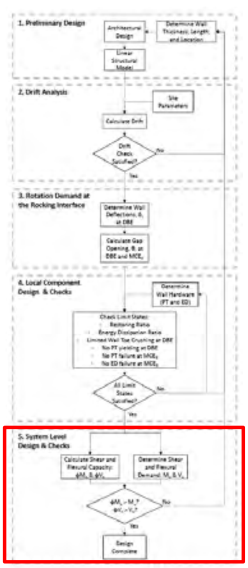
$$f_v = 1.5 * \alpha * \frac{V}{t_v * l_w}$$

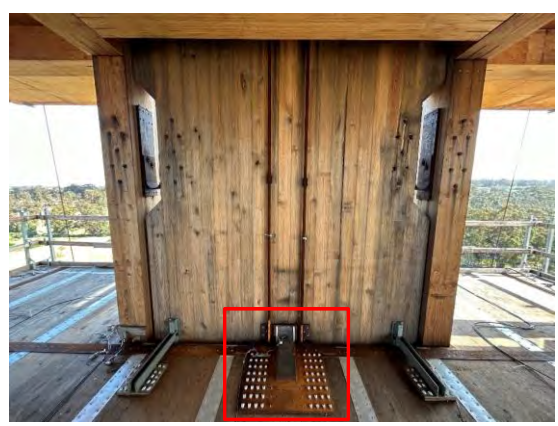
Left figure from Busch et al (2022)

41

41

Prescriptive Design Procedure





$$f_v = 1.5 * \alpha * \frac{V}{t_v * l_w}$$

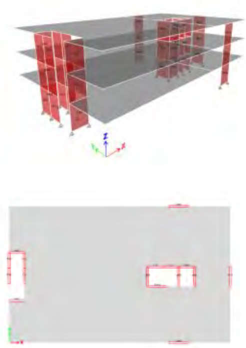
Left figure from Busch et al (2022)

42

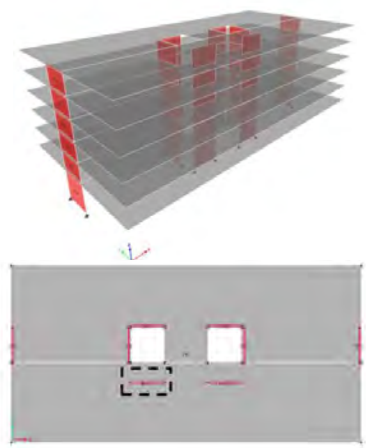
42

Design Examples

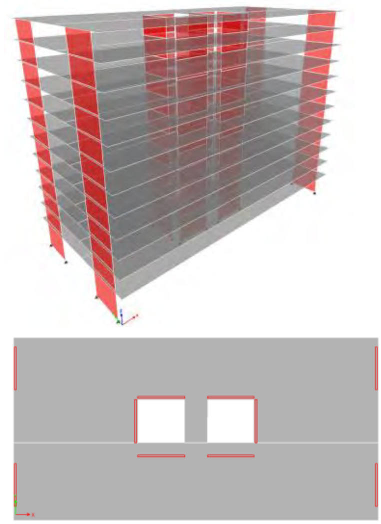
3-Story



6-Story



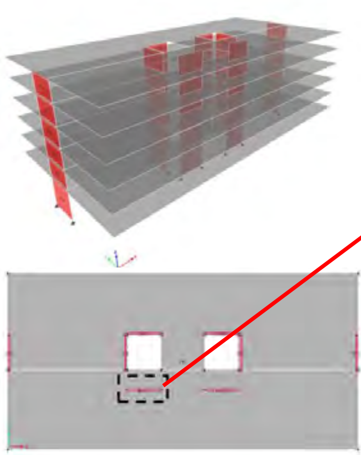
12-Story

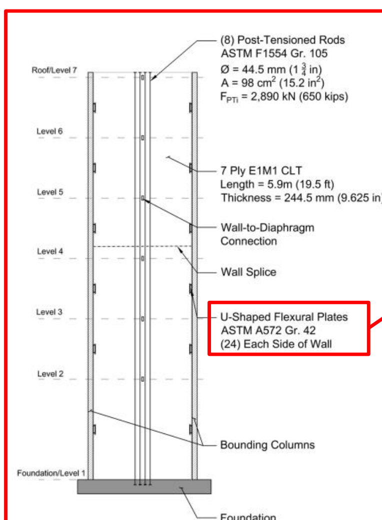


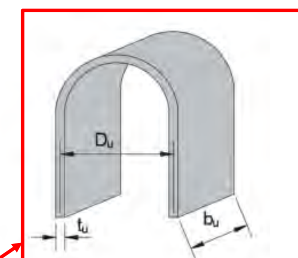
43

Design Examples

6-Story example for hypothetical site in Seattle, WA







b_u = 180 mm (7 in)

t_u = 13 mm (½ in)

D_u = 110 mm (4 ¼ in)

k_o = 10.9 kN/mm (62.4 kips/in)

F_u = 55.2 kN (12.4 kips)

F_y = 38.5 kN (8.65 kips)

Center and right figures from Busch et al (2022)

44

Performance-Based Design

Eleventh U.S. National Conference on Earthquake Engineering
Integrating Science, Engineering & Policy
June 25-29, 2018
Los Angeles, California

FRAMEWORK – INNOVATION IN RE-CENTERING MASS TIMBER WALL BUILDINGS

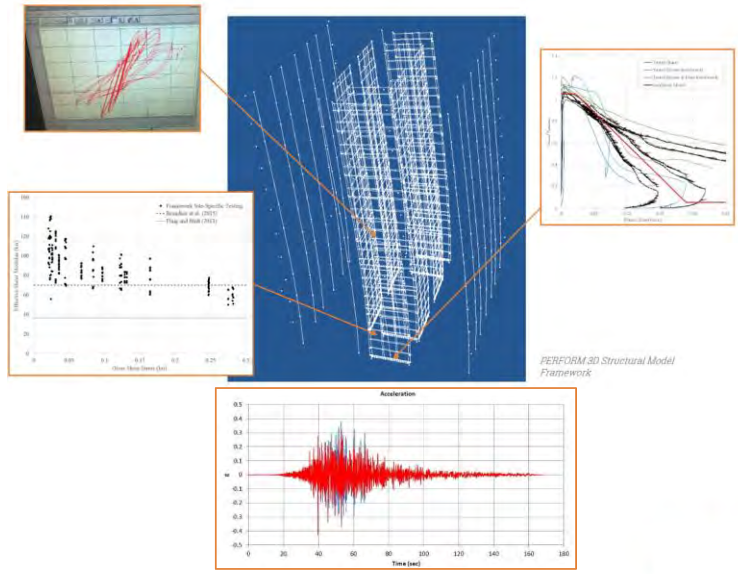
R.B. Zimmerman[1] and E. McDonnell[1]

ABSTRACT

Framework is a 12-story, 140ft (43m) tall mixed use building to be constructed almost entirely out of mass timber, including both the gravity and lateral force-resisting systems, in a region of high seismicity in the United States (Portland, Oregon). Design of the building's lateral force-resisting system has advanced the structural engineering profession's understanding of re-centering mass timber wall buildings. Critical issues identified during the design of Framework include appropriate designation of force- and deformation-controlled components in performance-based seismic design, especially as they relate to attaining seismically resilient/repairable buildings, effective linear modeling of cross-laminated timber, and conversion to mean properties for wood materials. Furthermore, adapting wood design equations such as axial-flexural interaction for application to mass timber required returning to first principles. Unique challenges in regard to splicing of wall panels also motivated envisioning new, innovative solutions. This paper intends to begin to establish a standard-of-practice for the design of re-centering mass timber wall buildings in the United States by learning from the design of Framework.

[1]Associate, KPFF Consulting Engineers, Portland, OR 97204 (email: reid.zimmerman@kpff.com)

Zimmerman R.B., McDonnell E. Framework – Innovation in Re-Centering Mass Timber Wall Buildings. *Proceedings of the 11th National Conference in Earthquake Engineering*, Earthquake Engineering Research Institute, Los Angeles, CA. 2018.



45

What Comes Next?

46

What Comes Next?

From ASCE 7-16

Table 12.2-1 Design Coefficients and Factors for Seismic Force-Resisting Systems

Seismic Force-Resisting System	ASCE 7 Section Where Detailing Requirements Are Specified	Response Modification Coefficient, R^a	Overstrength Factor, Ω_0^b	Deflection Amplification Factor, C_d^c	Structural System Limitations Including Structural Height, h_n (ft) Limits[d] — Seismic Design Category B	C	D[e]	E[e]	F[f]
B. BUILDING FRAME SYSTEMS									
1. Steel eccentrically braced frames	14.1	8	2	4	NL	NL	160	160	100
2. Steel special concentrically braced frames	14.1	6	2	5	NL	NL	160	160	100
3. Steel ordinary concentrically braced frames	14.1	3¼	2	3¼	NL	NL	35[j]	35[j]	NP[j]
4. Special reinforced concrete shear walls[g,h]	14.2	6	2½	5	NL	NL	160	160	100
5. Ordinary reinforced concrete shear walls[g]	14.2	5	2½	4½	NL	NL	NP	NP	NP
6. Detailed plain concrete shear walls[g]	14.2 and 14.2.2.7	2	2½	2	NL	NP	NP	NP	NP
Post-Tensioned Mass Timber Rocking Walls	14.1 and 14.5	6?	2½?	5?	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
14. Steel and concrete composite special shear walls	14.3	6	2½	5	NL	NL	160	160	100
15. Steel and concrete composite ordinary shear walls	14.3	5	2½	4½	NL	NL	NP	NP	NP
16. Special reinforced masonry shear walls	14.4	5½	2½	4	NL	NL	160	160	100
17. Intermediate reinforced masonry shear walls	14.4	4	2½	4	NL	NL	NP	NP	NP
18. Ordinary reinforced masonry shear walls	14.4	2	2½	2	NL	160	NP	NP	NP
19. Detailed plain masonry shear walls	14.4	2	2½	2	NL	NP	NP	NP	NP
20. Ordinary plain masonry shear walls	14.4	1½	2½	1¼	NL	NP	NP	NP	NP
21. Prestressed masonry shear walls	14.4	1½	2½	1¾	NL	NP	NP	NP	NP
22. Light-frame (wood) walls sheathed with wood structural panels rated for shear resistance	14.5	7	2½	4½	NL	NL	65	65	65
23. Light-frame (cold-formed steel) walls sheathed with wood structural panels rated for shear resistance or steel sheets	14.1	7	2½	4½	NL	NL	65	65	65
24. Light-frame walls with shear panels of all other materials	14.1 and 14.5	2½	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

47

Thank You

Reid Zimmerman, PE, SE
Technical Director | KPFF Portland
reid.zimmerman@kpff.com

kpff

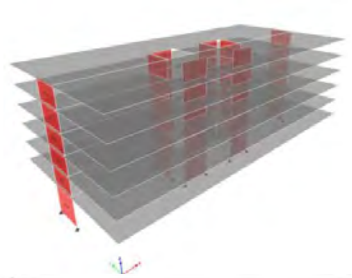
References

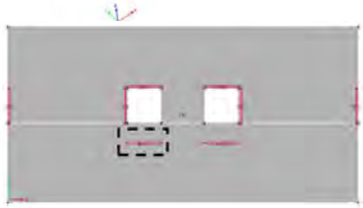
- Amer, A. (2019) “Multi-Directional Cyclic Testing of Cross-Laminated Timber Rocking Wall-Floor Diaphragm Sub-Assemblies”, Lehigh University.
- Busch et al. (2022) “Prescriptive Seismic Design Procedure for Post-Tensioned Mass Timber Rocking Walls”, *Journal of Structural Engineering*.
- Ganey, R., Berman, J., Akbas, T. and Loftus, S. (2017) “Experimental Investigation of Self-Centering Cross-Laminated Timber Walls”, *Journal of Structural Engineering*.
- Iqbal, A., Pampanin, S., Buchanan, A. H. and Palermo, A. (2015). “Performance and Design of LVL Walls Coupled with UFP Dissipaters”, *Journal of Earthquake Engineering*.
- Iqbal, A., Pampanin, S., Fragiacomo, M., Buchanan, A. H. and Palermo, A. (2016) “Seismic Design and Response of Post-Tensioned Shear Walls with Plywood Coupling”, *Journal of Structural Engineering*.
- Zimmerman, R.B. and McDonnell, E. (2018) “Innovation in Re-Centering Mass Timber Wall Buildings”, *Proceedings of the 11th U.S. National Conference on Earthquake Engineering*.

49

Design Examples

6-Story example for hypothetical site in Seattle, WA



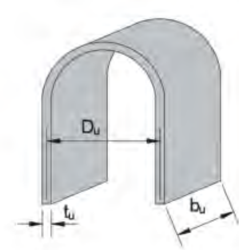


From prescriptive design provisions

$$k_{ufp} = \frac{16 * E_{ufp} * b_{ufp}}{27\pi}\left(\frac{t_{ufp}}{D_{ufp}}\right)^3$$

$$V_{n,ufp} = \frac{f_{ufpy} * b_{ufp} * t_{ufp}^2}{2 * D_{ufp}}$$

$$V_{pr,ufp} = \frac{R_t * f_{ufpt} * b_{ufp} * t_{ufp}^2}{2 * D_{ufp}}$$



b_u = 180 mm (7 in)

t_u = 13 mm (½ in)

D_u = 110 mm (4 ¼ in)

k_o = 10.9 kN/mm (62.4 kips/in)

F_u = 55.2 kN (12.4 kips)

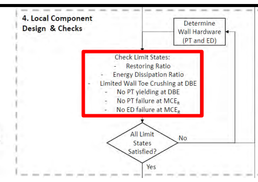
F_y = 38.5 kN (8.65 kips)

Center and right figures from Busch et al (2022)

50

50

Design Examples



- Restoring ratio [Eq. (3)]: No floor dead load acts on the wall due to the wall with Bounding Column Configuration, and the self-weight of the wall itself is conservatively neglected

$$\text{Restoring ratio} = \frac{2{,}890 \text{ kN} + 0.9 \times 0}{2 \times 55.2 \text{ kN} \times 24} = 1.10 > 1.0 \quad (SI \text{ units})$$

$$\text{Restoring ratio} = \frac{650 \text{ kips} + 0.9 \times 0}{2 \times 12.4 \text{ kips} \times 24} = 1.10 > 1.0 \quad (US \text{ units})$$

- Energy dissipation [Eqs. (4)–(10)]

$$\text{Energy-dissipation ratio} = \frac{(925 \text{ kN} + 925 \text{ kN}) \times 5.94 \text{ m}/2}{15{,}500 \text{ kN} \cdot \text{m}} = 0.35 > 0.25 \quad (SI \text{ units})$$

$$\text{Energy-dissipation ratio} = \frac{(208 \text{ kips} + 208 \text{ kips}) \times 234 \text{ in.}/2}{137{,}100 \text{ kip} \cdot \text{in.}} = 0.35 > 0.25 \quad (US \text{ units})$$

- Limited wall toe crushing at DBE [Eqs. (11)–(13)] with the plastic hinge height, H_p, taken as 61 cm (24 in.)

$$DCR_{\text{wood,compstrain}} = \frac{-0.0118}{-0.9 \times 0.015} = 0.87 \leq 1.0$$

- No PT yielding at DBE [Eqs. (14)–(17)]

$$DCR_{\text{PT,yieldforce}} = \frac{2{,}890 \text{ kN} + 1{,}090 \text{ kN}}{0.9 \times 724 \text{ MPa} \times 98 \text{ cm}^2} = 0.62 \leq 1.0 \quad (\text{SI units})$$

$$DCR_{\text{PT,yieldforce}} = \frac{650 \text{ kips} + 246 \text{ kips}}{0.9 \times 105 \text{ ksi} \times 15.2 \text{ sq in.}} = 0.62 \leq 1.0 \quad (\text{US units})$$

- No PT failure at MCE_R [Eqs. (18) and (19)]

$$DCR_{\text{PT,ultimatestrain}} = \frac{0.0024}{0.05} = 0.05 \leq 1.0$$

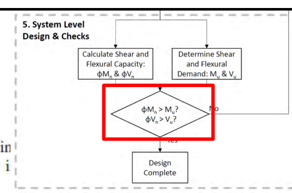
- No energy-dissipation failure at MCE_R. For UFPs, the critical design property is the distance from the start of the 180° bend and the nearest attachment point (e.g., weld or bolt) on the straight portion. Therefore, this length would need to exceed $\Delta_{ed,lens} = 62$ mm (2.45 in.) per Fig. 7(c) plus any elastic elongation of the tension side of the CLT wall panel at the location of the top story UFPs.

Figures from Busch et al (2022)

51

51

Design Examples



Demands:

The shear demand tributary to the wall of interest in this example is $V_u = 930$ kN (210 kips) based on the scaled MRSA from the ETABS analysis (which provides a shear demand for each wall). Alternatively, the shear demand for the wall of interest can be determined from the base shear and diaphragm analysis deemed appropriate for the building. As mentioned previously, further modifications were applied to the shear based on Eq. (32) in Step 5 to provide the final amplified shear demand to account for higher mode effects

$$V_{uA} = 1.5 \times 1.5 \times 1.25 \times 930 \text{ kN} = 2{,}620 \text{ kN} \quad (\text{SI units})$$

$$V_{uA} = 1.5 \times 1.5 \times 1.25 \times 210 \text{ kips} = 590 \text{ kips} \quad (\text{US units})$$

The moment demand of the wall was taken directly from the ETABS model under the scaled MRSA in the x-direction

$$M_u = 13{,}400 \text{ kN} \cdot \text{m}(118{,}300 \text{ kip} \cdot \text{in.})$$

Capacities:

The shear capacity mentioned in Eq. (34) in where the reference in-plane shear capacity i manufacturer's literature:

$$\phi V_n = 0.75 \times 2.88 \times 1.0 \times 5.94 \text{ m} \times 457 \text{ kN/m} = 5{,}870 \text{ kN} \quad (\text{SI units})$$

$$\phi V_n = 0.75 \times 2.88 \times 1.0 \times 19.5 \text{ ft} \times 31.3 \text{ kip/ft} = 1{,}320 \text{ kips} \quad (\text{US units})$$

The moment capacity mentioned in Eq. (22) in Step 5 will be as follows:

$$\phi M_{n,\text{design}} = 0.9 \times 15{,}500 \text{ kN} \cdot \text{m} = 13{,}900 \text{ kN} \cdot \text{m} \quad (\text{SI units})$$

$$\phi M_{n,\text{design}} = 0.9 \times 137{,}100 \text{ kip} \cdot \text{in.} = 123{,}400 \text{ kip} \cdot \text{in.} \quad (\text{US units})$$

Check:

$$V_{uA} = 2{,}620 \text{ kN} \leq 5{,}870 \text{ kN} = \phi V_{n,\text{design}} \quad (\text{SI units})$$

$$V_{uA} = 590 \text{ kips} \leq 1{,}320 \text{ kips} = \phi V_{n,\text{design}} \quad (\text{US units})$$

$$M_u = 13{,}400 \text{ kN} \cdot \text{m} \leq 13{,}900 \text{ kN} \cdot \text{m} = \phi M_{n,\text{design}} \quad (\text{SI units})$$

$$M_u = 118{,}300 \text{ kip} \cdot \text{in.} \leq 123{,}400 \text{ kip} \cdot \text{in.} = \phi M_{n,\text{design}} \quad (\text{US units})$$

Figures from Busch et al (2022)

52

52