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NHERI TallWood 10-story Test Nonstructural, Part 1 of 4: Project Overview and Curtain Wall Subassembly

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ABSTRACT

As the market for mass timber buildings have been growing in the US due to the need for more sustainable building designs, more research is required for tall mass timber buildings in high seismic regions. A 10-story mass timber building with both structural and non-structural components will be tested on the University of California San Diego (UCSD) large scale shake table to investigate the behavior and resilience of the building components. One of the key nonstructural systems included in the program is a 2-story, fire-rated curtain wall subassembly. The subassembly has been designed for both wind and seismic loading, wherein out-of-plane wind loading controls the connection forces. The test objectives for the curtain wall subassembly are to determine if the AAMA racking protocols accurately reflect the response in a real building, to evaluate the deformation response of the stiffer fire-rated curtain walls, and to correlate the observed damage states with the interstory drift.

Introduction

The market for mass timber building systems has been growing steadily in the United States. Mass timber provides a more innovative and sustainable option for building design compared to leading steel and concrete building systems. Tall mass timber building systems have become a possible solution to meet the demands for urban densification and environmental sustainability [1]. While the market for mass timber in the U.S. has been growing, research on mass timber buildings in seismic regions lags behind. The NHERI Tallwood project will develop and examine structurally resilient rocking wall lateral systems in a shake table test of a 10-story tall mass timber building. The rocking wall system is inherently flexible and expected to sustain large drift demands without much damage.

While the main project objective is to validate the seismic performance of the structural system, the resilience of the non-structural systems must also be considered. This investigation focuses on vertically distributed (connected floor-to-floor) drift sensitive components, and aims to validate state-of-the-art, but somewhat untested, deformation compatible details. The 1st of 4 related papers, this paper describes the 10-story testbed structure, outlines the design criteria for nonstructural walls, and presents details of a 2-story C-shaped curtain wall subassembly that wraps around one corner of the building. Subsequent papers will describe

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details of additional exterior wall subassemblies [2], interior walls [3], and stair systems [4] incorporated into the test program.

10-Story Mass Timber Testbed Structure

This project will utilize the large-scale outdoor shake table facility at the University of California, San Diego (UCSD). The 10-story test specimen, displayed in Fig. 1(a), will be the world's tallest full-scale mass timber building ever tested. The test building will utilize a variety of mass timber products for the floors, gravity frame, and rocking wall components in the building. As shown in Fig. 1(b), cross-laminated timber (CLT) and mass plywood panel (MPP) post-tensioned rocking walls with U-shaped flexural plate (UFP) dissipaters act as the lateral force resisting systems in the building in the east-west and north-south directions, respectively. Gravity framing is provided by veneer-laminated timber (VLT) beams and columns detailed with pinned connections in each direction. Several different mass timber components are utilized for floor diaphragms including CLT, VLT, glu-laminated timber (GLT), nail-laminated timber (NLT), and dowel-laminated timber (DLT). The floor panels are connected together by various strap and spline connections.

The structure has been designed following the methodology in [5]. Initial member sizes are developed using a force-based procedure with an assumed R factor, applying elastic analysis to an equivalent shear wall model of the system. Deflection amplification principles are applied to account for inelasticity, and used to compute design base rotation, and subsequently, demands to design post-tensioning and energy dissipation elements. Demands under service, design and MCE events are updated through response history analysis of a nonlinear 3D model of the structure using OpenSees. Refinement of the design to satisfy resilience objectives is envisioned as part of the design procedure, but practical constraints limit the ability to full execute this.

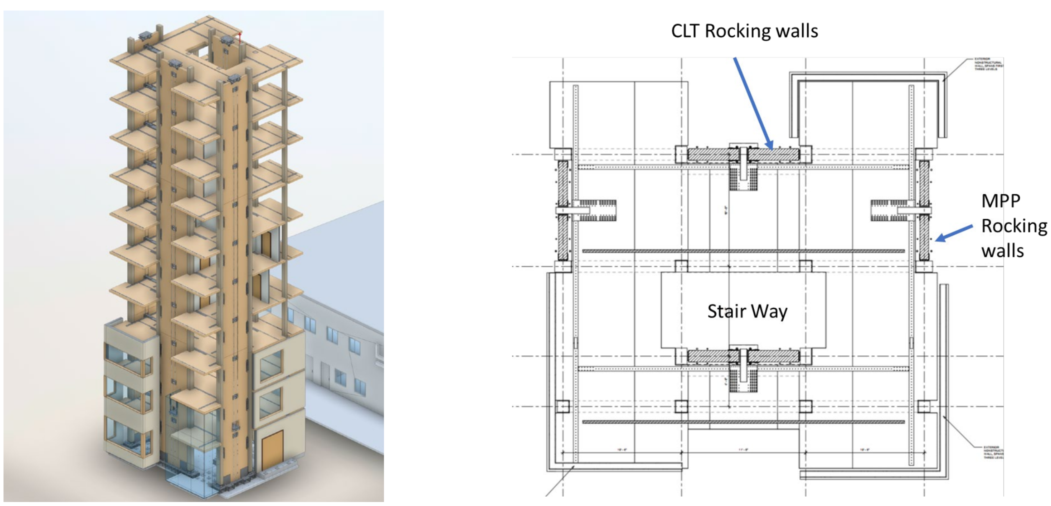

Figure 1. (a) 3D rendering and (b) floor plan of the 10-story test building at the large-scale outdoor shake table facility at UCSD.

Design Criteria

Common design criteria are applied to all nonstructural walls with consideration of dead, wind, and seismic loading. The key assumptions used to calculate the design loads are detailed in the subsections below, with the values tabulated in Table 1.

Wind Demand

The wind design pressures are based on the 2018 IBC [5] and ASCE 7-16 [6]. The parameters for the 3-second gust are 96 mph/Exposure C/Risk Category II/Mean Roof Height: 115'-0". The structure is classified as a partially open building, which leads to the same design pressures as a fully enclosed building. Per ASCE 7-16 Section 30.5, the leeward wind direction controls and wind pressure is constant over the height of the building and thus the same for all wall subassemblies (Table 1). With exception, interior walls are not normally designed for wind pressure. To avoid an outcome of unrealistically strong interior walls, the design wind velocity has been reduced by 0.8 and the design wind pressure by $0.8^2 = 0.64$, as allowed by ASCE 37 [7] for a temporary structure with a 6-month design life.

Seismic Demand

The seismic demands are based on a modified version of ASCE 7-16 Eq. 13.3-1 to achieve the performance-based design criteria:

$$F_p = \frac{0.4 a_p S_{MS} W_p}{\left(\frac{R_p}{I_p}\right)} \left(1 + 2\frac{z}{h}\right) \tag{1}$$

In this equation, F_p is the total lateral seismic force, a_p is the component amplification factor, S_{MS} is the MCE spectral coefficient at short periods, w_p is the unit weight of the component, R_p is the component response modification factor, I_p is the importance factor, z is the max elevation of the component within the building, and h is the total height of the building. To meet the performance-based design criteria, S_{MS} - computed as 1.654 - replaces S_{DS} in the ASCE 7-16 Eq. 13.3-1, and R_p/I_p is taken as 1.0 to design for elastic response.

The seismic demand is calculated independently for interior walls, exterior walls, and curtain wall subassemblies, controlled by the maximum elevation of each component. The max elevation of the interior walls, exterior walls, and curtain wall are 104.5', 36', and 24', respectively, relative to a total building height of 115'. While a_p is taken as 1 for most components, a_p = 1.25 and thus develops higher forces for the non-structural wall fasteners listed separately in Table 1. Some components were designed using Allowable Stress Design (ASD) assumptions, such that the demands in Table 1 can be multiplied by 0.6 for wind loading and 0.7 for seismic loading.

Table 1. Seismic and wind design loads for non-structural components in the test structure using Load and Resistance Factor Design (LRFD) philosophy.

	Interior Walls	**Exterior CFS Walls**	**Exterior Wall Connections**	**Curtain Walls**	**Curtain Wall Connections**
max z/h	0.91	0.31	0.31	0.21	0.21
F_p	$1.86W_p$	$1.07W_p$	$1.34W_p$	$0.94W_p$	$1.18W_p$
W_p	6 psf	10 psf	10 psf	16.5 psf	10 psf
Design Wind Pressure Zone 4: Typical	14 psf	28.6 psf	28.6 psf	28.6 psf	28.6 psf
Design Wind Pressure Zone 5: Corners	24.3 psf	52.5 psf	52.5 psf	52.5 psf	52.5 psf

Design of the Curtain Wall Subassembly

Curtain wall systems are used in multi-story buildings and are attached to the edge of the floor slabs. Curtain walls are framed floor to floor typically with heavy horizontal and vertical members called mullions that support the glass lites. Wind loading generally controls the out-of-plane behavior of the curtain wall system, with seismic loading as the primary influence on in-plane behavior [8]. Three variations on curtain wall system detailing can accommodate drift during seismic loading: (1) in unitized systems, horizontal slip joints allow horizontal movement between upper and lower units; (2) in stick-built systems, the framing racks or distorts to displace with the building; and (3) in hybrid systems, the framing slips both horizontally and vertically to allow rocking between adjacent units. This project utilizes a stick-built curtain wall system.

The curtain wall system may utilize one of two methods to secure the glass lite within the mullion glazing pocket: (1) structural sealant glazing (SSG) uses a silicone adhesive to secure the glass to the mullion, or (2) mechanically captured glazing uses a gasketed pressure plate mechanically secured to the mullions through the glazing pocket to hold the glass in place. The latter method is applied to the curtain wall system in this project. The curtain wall system must be designed to prevent glass fallout at the peak drift per ASCE 7-16 Section 13.5.9. The displacement at glass fallout is determined using AAMA 501.6 [9] racking protocol applied to a mock-up of the curtain wall system, or theoretical calculations for a mechanically captured system.

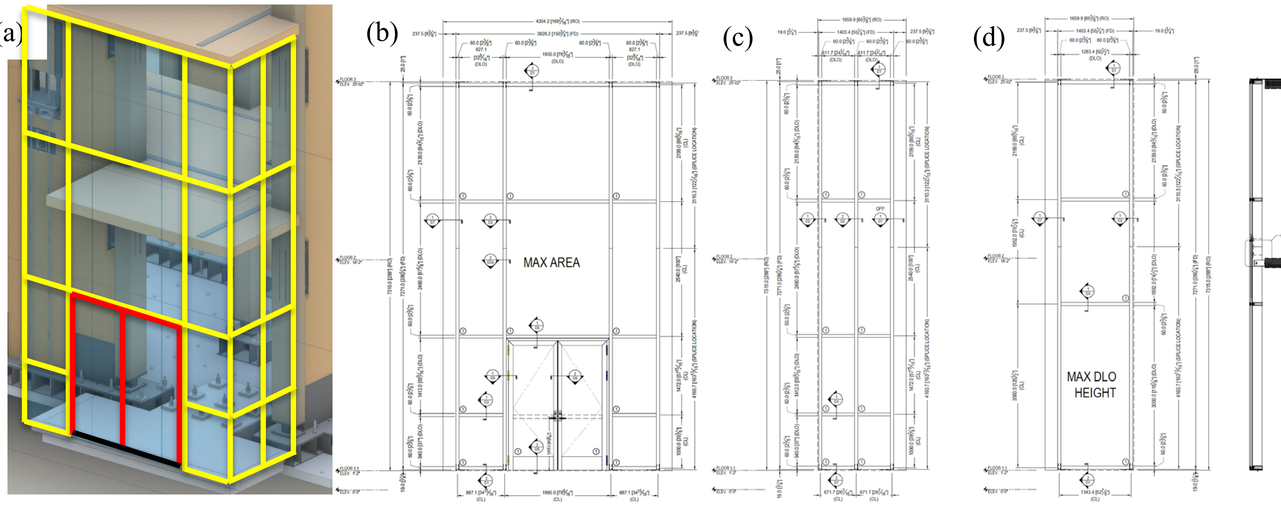

Figure 2. (a) 3D rendering of the curtain wall assembly and location of the floor connections and mullions; detailed (b) front view, (c) right side view, and (d) the left side view of the curtain wall assembly.

The 2-story curtain wall subassembly designed for this project is shown in Fig. 2. The front wall is 14'-1 7/16" long, and the side walls are 5'-5 3/8" long. The curtain wall system utilizes a fire-rated specialty product with 1-1/16" glazing. The glazing is a multilayer laminate composed of thin sheets of annealed glass with intumescent layers in between for fire protection. The area is utilized to create a glass panel with the largest allowable area in the front wall and another with the tallest allowable panel height on one side wall. Smaller panels are utilized on the other side wall of the subassembly as a control. This fire-rated assembly utilizes S235JR steel mullions (F_y = 25 ksi) instead of typical aluminum mullions characteristic of curtain walls.

Connection Details

The anchorage demands are calculated based on design criteria above, and are controlled by out-of-plane wind loading. The curtain wall system is connected to the base and the floor diaphragms at each of its vertical mullions: (1) at the sill, (2) at an intermediate location, and (3) at the head of the curtain wall subassembly, represented in Fig. 2. The sill of the curtain wall system is connected to the base slab steel ledger using two ½"-13 x 2" Nelson Weld Studs at each vertical mullion. Vertical mullions are connected to top the second-floor diaphragm using four ½" x 5 ½" fully threaded self-tapping wood screws. The head of the curtain wall system is fastened to the top of the third-floor diaphragm using eight ½" x 5 ½" fully threaded self-tapping wood screws. Each of the anchorage components are made of ASTM A36 steel (F_y = 36 ksi).

Conclusions

This paper discusses the need for innovative tall mass timber buildings as a solution to urban densification and environmental sustainability in seismic regions. More research is needed in this area, and therefore a 10-story shake table test of a 10-story mass timber building with post-tensioned rocking walls will be conducted at the NHERI@UCSD facility. The test specimen also serves as a realistic testbed to examine the response of less researched non-structural components. This test building will undergo a suite of ground motions of varying intensities to gain more information about the performance of nonstructural systems. For the curtain wall subassembly, the test objectives are: (1) determining if the AAMA racking protocols are representative of the response in a real building, (2) evaluating the deformation capacity of the stiffer fire-rated curtain walls compared to other systems, and (3) correlating the observed damage states with the interstory drift.

Acknowledgments

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