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SEISMIC AND ENERGY RETROFIT OF MULTIFAMILY LIGHT-FRAME TIMBER RESIDENTIAL BUILDINGS USING MASS PLY PANEL (MPP) WALL FAÇADE SYSTEM

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ABSTRACT: The most common type of construction in the US for single and multifamily housing is known as Light-Wood-Frame (LWF). This system has been widely used for decades, following prescriptive design rules and usually without an engineered-based analysis. Due to the evolution of knowledge on the seismic hazard in the Pacific Northwest, retrofitting these structures built before the 1990s has become a necessity. In addition, these buildings also lack the energy efficiency required to reduce our global carbon footprint. This paper presents a novel façade retrofit solution consisting of prefabricated mass ply panels (MPP) that can be rapidly applied on-site over existing building cladding to upgrade older LWF one- to three-story buildings. The structural design of an existing prototype two-story LWF building in Portland, OR is presented. Custom steel connections resist design forces following the Equivalent Lateral Force (ELF) method from ASCE 7-16. Nonlinear displacement validation is performed using analytical models (AM) and finite element models (FEM), obtaining lateral drifts of 0.50% for the design earthquake (DE) reducing the expected damage in the building. Over-strength factors are calculated, considering the contribution of existing LWF walls and steel connections. A follow-up experimental setup is described for future validation of the initial assumptions.

KEYWORDS: Façade, Light-Wood-Frame, Mass Timber, Retrofitting, Seismic Vulnerability.

1 INTRODUCTION

In the US, the majority of low-rise residential buildings consist of Light-Wood-Frame (LWF) lateral force-resisting system (LFRS). The LWF structural system makes use of dimensional lumber elements equally spaced forming walls, floor diaphragms, and roofs. These elements are generally joined together using nails and panelised sheeting, such as plywood, oriented-strand board (OSB), or gypsum wallboard (GWB). Even though this solution has been prevalent and overall well-designed buildings tend to perform well under seismic loading, the 1994 Northridge Earthquake resulted in many residential buildings sustaining damage due to excessive lateral drifts [1,2]. In addition, in the Pacific Northwest, the Cascadia Subduction Zone, which drives the high seismic risk of existing structures in the region [3] was only recognized in the early 1990s, and buildings in the region constructed prior to this date were designed to sustain relatively low levels of seismic loading.

Besides the inadequate seismic resistance of many existing buildings constructed before the 1990s, many US buildings were constructed before the advent of building energy codes. In 1975, the American Society of Heating, Refrigerating, and Air-Conditioning Engineers (ASHRAE) promulgated Standard 90-75, which is widely recognized as the first US residential energy code. Since that time, housing energy efficiency has significantly improved in many states. However, pre-code housing remains a significant fraction of the nation's housing stock. According to the Northwest Energy Efficiency Alliance (NEEA) Residential Building Stock Assessment, 88% of this housing stock in the Pacific Northwest is one- to three-story light-wood-frame structures and were constructed between 1960-1994 with very low wall insulation levels (64% had R8-R12 wall insulation) [4]. In addition, the majority of the 88% of the housing stock was constructed with 2x4 studs (38 mm x 76 mm) at 406 mm (16”) on-centre [4].

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Such outdated buildings can benefit from a retrofit solution that minimizes impact on the interior of the units, thus minimizing downtime, combining increase energy and seismic performance. The solution proposed in this study involves performing most, if not all, interventions on the exterior of the building through the development of a façade retrofit solution. While other successful energy façade retrofit programs exist in Europe, such as the *Energiesprong* [5] method that provides many lessons for the US, energy façade retrofits on the Pacific Northwest must also contend with seismic upgrades, which are often automatically triggered with the mass of an existing building increases by 10% relative to the mass of the existing building.

In the development of a novel façade retrofit solution, this study adopts a recent product known as mass ply panel (MPP) [6] in the development of structural and non-structural retrofit façade panels that enable seismic and energy retrofit of LWF buildings. Specifically, this paper proposes a structural design methodology for the structural façade panels, considering custom-designed connectors that attach the façade retrofit solution to the existing concrete foundation and existing LWF structure. In addition, considerations needed for the design of the non-structural façade panels are also presented.

1.1 EXAMPLE CASE STUDY

An existing two-story LWF structure built in 1971 in the city of Portland, Oregon (45.51179, -122.67563) is selected as an application example to demonstrate the seismic design retrofit procedures and dimensioning of structural elements of the retrofit façade system and considerations for non-structural elements.

Figure 1: *Visualization of case study multifamily building used for façade retrofit study, pre-retrofit [7].*

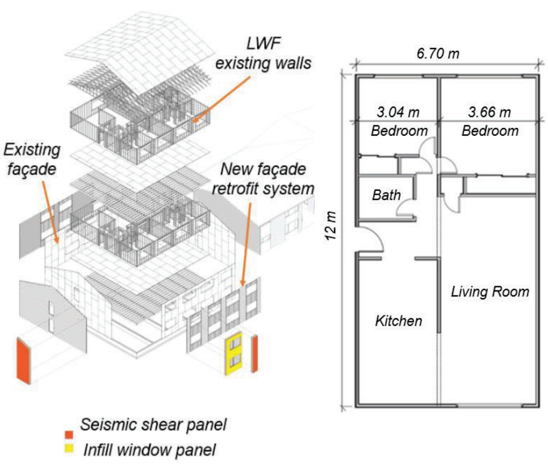
The two-story building shown in Figure 1 consists of multiple two-unit modules illustrated in Figure 2a and b. For the existing building, based on the NEEA study [4], it is assumed that there are two types of walls, including (1) exterior walls consisting of 8x8 plywood sheathed wood structural panels (WSP) with 6d nails spaced a 150 mm and (2) interior walls consisting of gypsum sheathed walls.

1.2 FAÇADE RETROFIT SOLUTION

A façade system is proposed, formed by two different types of MT panels. First, the non-structural MPP with slotted-hole connections resisting out-of-plane loads only (highlighted in yellow in Figure 2 and Figure 3), and second the structural walls that resist lateral forces (highlighted in red in Figure 2 and Figure 3).

The MPP structural walls have increased over-turning capacity due to the steel plate hold-downs that transfer overturning tension and compression forces caused by lateral loads to the foundation. A shear steel plate is installed at the base of the MPP transferring the base shear reaction to an upgraded foundation. Fasteners distribute the tension forces from the steel plates to the MPP, and expansion bolts connect the steel plates to the concrete foundation. The MPP walls are connected to MPP transfer joists that transfer the load from the existing LWF to the retrofit system (see Figure 4a).

Insulation layers are attached to both structural and non-structural MPP panels, including wood fibre insulation infill and siding. Windows, doors, field-installed foundation insulation with vertical siding elements used to conceal the hold-downs and other panel-to-panel joints on the completed façade.



(a) (b)

Figure 2: *(a) 3D exploded view of a two-unit module of the case study multifamily building and example retrofit panels, with structural panels shown in orange and non-structural in-fill panels shown in yellow, (b) Individual unit in case study building.*

This paper focuses on structural design aspects of the study. For more on architectural design considerations, energy retrofit design, construction details, and a mock-up of the proposed solutions, we direct the interested reader to a separate paper [7].

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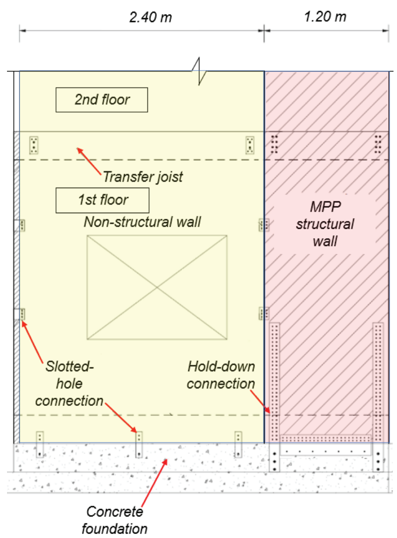
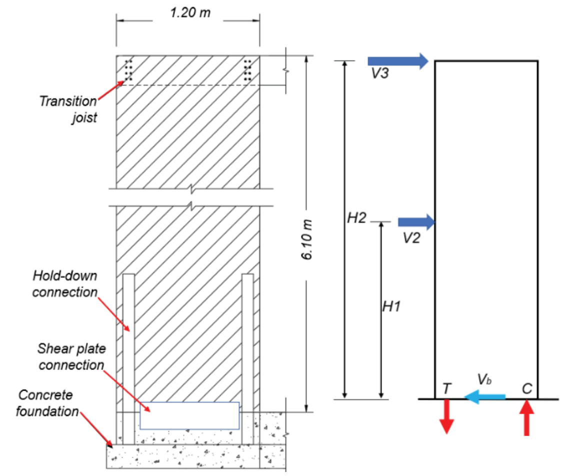


Figure 3: MPP non-structural and structural wall panels with their various connections. The non-structural panel is highlighted in yellow, and the structural panel is shown in red.



(a) (b)

***Figure 4.** (a) Single wall components, (b) Reaction forces generated at the base of the MPP.*

2 STRUCTURAL DESIGN PROCEDURES

The design of the new lateral force resisting system (LFRS) follows a capacity-based design approach, where hold-downs are designed as structural fuse elements, while shear transferring elements are designed to remain essentially elastic by considering an over-strength factor (Ω), precluding brittle failures and allowing the wall to develop large deformations without loss of structural capacity.

The design of the structural wall panel followed existing standards, including ASCE 41-17 [8] and ASCE 7-16 [9], and consisted of determining: (1) the tributary seismic weight to be resisted by the structural wall panel, (2) the design seismic base shear, (3) design of the components of the structural wall and connections, (4) a drift analysis, and (5) capacity-based design of elements that are designed to remain essentially elastic. For the non-structural walls, the design consisted of displacement compatibility checks, namely through the design of slotted connections in panel-to-panel, panel-to-transfer joist, and panel-to-foundation connections. The existing and upgraded concrete foundation requires the design of epoxied dowels, shear transfer between the upgraded foundation and the existing foundation, and geotechnical foundation stability checks.

2.1 SINGLE WALL ANALYSIS

In a single structural wall, lateral design forces are calculated and distributed assuming a triangular distribution of forces (see Figure 4b).

Tension and compression reaction forces generated at the base of the wall, T and C, respectively, are determined using the following expression:

$$T = C = \frac{(V_3 \times H_2) + (V_2 \times H_1)}{L_1 + \frac{2}{3}a} \qquad (1)$$

where V_3 is the lateral load at floor level 3, H_2 the height at floor level 3, V_2 is the lateral load at floor level 2, H_1 is the height at floor level 2, L_1 is the length between the center of the hold-down connection and the initial point where the wall starts lifting, and a is the length of the compression zone. Additional details on these dimensions are shown in Figure 6a.

The foundation will most probably need to be retrofitted since typical stem walls in existing buildings will not have the bearing area needed to support the new façade panels. Also, due to the addition of extra walls on the façade, the existing diaphragm needs to be evaluated to verify if additional retrofit elements are needed to resist added capacity provided by the façade panels. Upon determination of the diaphragm forces, it may be necessary to provide additional floor sheathing and/or steel coiled straps that may work as collectors or diaphragm tension chords. The need for these elements will vary from project to project, and are therefore not detailed in this paper. Example cross-sections on how the MPP is attached to the existing façade and additional non-structural insulation materials are shown in Figure 5a and b for foundation and roof heave connections, respectively. Additional details on other necessary considerations are provided elsewhere [7].

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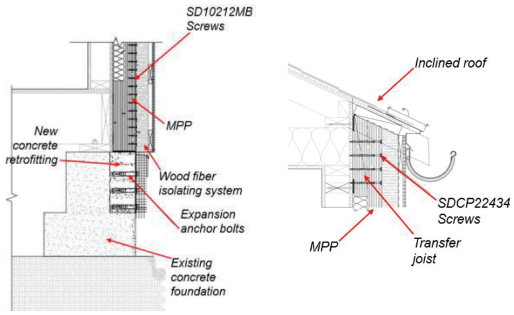


(a) (b)

Figure 5. *Section cut of (a) MPP to the concrete foundation, (b) MPP to transfer joist connection.*

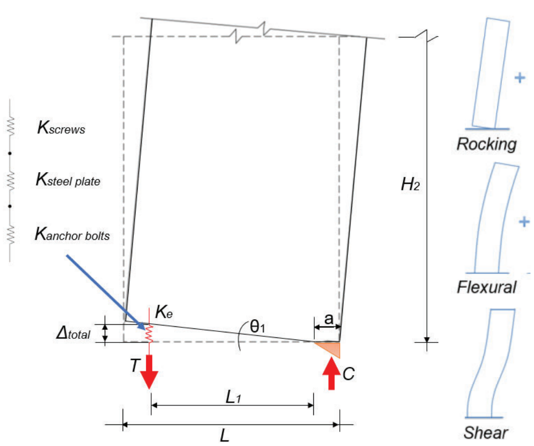
2.2 LATERAL DEFORMATION ANALYSIS=

Lateral deformations of the new LFRS can be verified using a nonlinear analytical model, that considers three types of deformation types in the wall: (1) lateral deformation due to the rocking of the panel, (2) lateral deformation due to the flexural deformation of the panel, (3) lateral deformation due to the shear deformations of the panel (see Figure 6b).

Figure 6a illustrated a simplified model to capture the effects induced by the rocking of the wall. As the wall rocks about one end, the opposite end tends to uplift resulting in compression and tension of either hold-down. For the hold-down that goes into tension an equivalent stiffness K_e for the hold-down connection is given by:

$$\frac{1}{K_e} = \frac{1}{K_{sc}} + \frac{1}{K_{sp}} + \frac{1}{K_{ab}} \tag{2}$$

where $\boldsymbol{K_{sc}}$ is the stiffness provided by the screws, $\boldsymbol{K_{sp}}$ is the stiffness of the steel plate, and $\boldsymbol{K_{ab}}$ is the stiffness provided by the anchor bolts.



(a) (b)

Figure 6. *(a) Simplified model for rocking deformation analysis, (b) Types of lateral deformations in the wall.*

The lateral displacement at the top of an MPP wall D_{total} is given by:

$$D_{total} = D_{hold-down} + D_{flexure} + D_{shear} \tag{3}$$

$$D_{hold-down} = H \times \theta_1 \tag{4}$$

$$\theta_1 = 90^o - \cos^{-1}\left(\frac{\Delta_{total}}{L_1}\right) \tag{5}$$

$$D_{flexure} = \frac{V_2 {H_1}^3}{3E_{app}I} + \frac{V_3 {H_2}^3}{3E_{app}I} \tag{6}$$

$$D_{shear} = \frac{V_2 H_1}{G_{app}A} + \frac{V_3 H_2}{G_{app}A} \tag{7}$$

where $D_{hold-down}$ is the lateral displacement due to hold-down deformation, $D_{flexure}$ is the displacement due to wall flexure, and D_{shear} is the displacement due to wall shear; θ_1 is the rotation at the base of the wall; Δ_{total} is the opening at the base of the wall; H is the height of the wall, $E_{app}I$ is the apparent wall's flexural stiffness (I = 1/12 $t_w L^3$), and $G_{app}A$ is the wall's apparent shear stiffness (A = $t_w L$).

2.3 NONLINEAR CAPACITY OF LWF WALLS

The capacity-based design methodology requires the calculation of any source of over-strength that can affect the new LFRS. In the present application, the main contributor to over-strength in the system is the existing LWF structure. To obtain the lateral stiffness of these walls, simplified backbone curves can be calculated from experimental tests, considering different types of sheathing and length of walls [10]. Structural walls are considered the ones with proper sheathing, with WSP. Infill walls are initially not considered in the lateral resistance of the exiting LWF, and are only considered for over-strength factor calculation. Nevertheless, depending on the contribution of the nonstructural walls, they may be considered structural for structural assessment.

3 EXAMPLE BUILDING APPLICATION

The loads considered to act on the structure are: (1) Dead loads, including self-weight and over-imposed loads of 1.05 kN/m^2 and 0.72 kN/m^2 for intermediate floors and roof, respectively, and (2) live loads of 1.91 kN/m^2 and 0.96 kN/m^2 for intermediate floors and roof, respectively. The performance level for existing structures defined in ASCE 41-17 (BSE-1E) requires an earthquake with a 20% of probability of return in 50 years. For existing structures, ASCE 41-17 and local design codes [11] allow for the use of a design limit of 75% of the seismic demand used for new buildings (BSE-1N), or 10% in 50 years, also known as Design Earthquake (DE).

The design approach included the following steps. First, the two-unit structure shown in Figure 2a was assumed to

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be retrofitted by six MPP structural wall segments for the LFRS, for the direction of the building with more openings. The other direction of the building is not presented in this paper, since the more critical direction is analysed. Each MPP structural wall segment has dimensions: 1.20 m wide, 6.10 m height, and 76.2 mm thick. Second, a seismic modification factor (R-factor) of 3 is assumed for the MPP walls [12]. The building is a risk category II structure, therefore a base shear (V_b) of 22% of the seismic weight is determined for the BSE-1N design level earthquake (DE). Applying the limit of 75% the BSE-1E design base shear of 16.8% is obtained. Note that this level is higher than what would be needed for the existing structure with light-frame-wood walls, which would be allowed to be designed for a corresponding R-factor value of 6.5. This would result in a BSE-1N force value of 10.1% of the buildings seismic weight and a corresponding BSE-1E force value of 7.6% of the seismic weight.

Note that this increased level of design force for the façade retrofit solution compared to a retrofit solution of existing LWF walls using additional and stronger sheathing, the façade retrofit solution enhances building performance by reducing drifts and mitigates soft-story failure modes, thus allowing for an enhanced resilient design. Nonetheless, the solution may raise questions from structural engineers and some building owners since it does require that the existing structure be retrofitted to a higher seismic force level.

For the example building, a seismic weight of 400 kN was determined, considering the dead load and 25% of live load. Following allowable stress design (ASD) [9] combinations (with 0.70 factor for seismic forces), a horizontal force of 7.87 kN is obtained for each of the MPP walls, and tension and compression forces of 36.61 kN are obtained for the hold-downs and MPP structural wall panel, respectively.

3.1 HOLD-DOWN CONNECTION DESIGN

The number of screws needed to resist the tension forces is determined following the NDS yielding modes [13], considering a modifying factor $C_d = 1.6$, a group factor $C_g = 0.86$, and additional factors equal to 1.0. The steel plate is sized following the requirements in AISC 360-16 [14], including tension and shear checks. The design checks for the expansion bolts follow the design manual provided by the manufacturer [15]. Shear and compression stresses on the MPP structural wall are checked to verify the thickness of the panel. The ASTM A572-50 [16] steel plate hold-downs obtained are 3.2 mm thick, 101 mm wide, and 1460 mm high, and require a total of 60 screws type SD10212MB [17], and three carbon steel expansion bolts with a diameter of 16 mm (see Figure 7).

To determine the hold-down equivalent stiffness (K_e in Figure 6), the stiffness used for an individual fastener is obtained from shear tests of angle bracket connections using nails with similar diameters [18]. For the steel plate, an axial stiffness relationship, E_sA/L, is used, where E_s is the steel Young's modulus, A is the cross-section area, and L is the effective length of the steel plate. Elastic and plastic shear stiffness and yield forces for an anchor bolt are assumed following engineering first principles. Backbone values per component are shown in Table 1.

Table 1: *Backbone parameters for the hold-down connection.*

Element	Yield force (kN)	Yield disp. (mm)	$K_{elastic}$, (kN/mm)	$K_{plastic}$, (kN/mm)
Bolts	345	38	9	12
Steel plate	122	2	52	3
Screws	96	9	11	4

3.2 SHEAR TRANSFER CONNECTIONS

For this example, an overstrength factor, Ω, of 4 is initially considered. Based on this assumption, the ASTM 572 shear plate connection is 3.2 mm thick, 171 mm wide, 889 mm long, and requiring a total of 46 SD10212MB screws, and seven carbon steel expansion bolts with a diameter of 9.5 mm.

For the transfer joists, 16 screws type SDCP22434 [17] are needed. Note that these are designed with slotted hole connections that allow the wall to rock relative to the transfer joist without inducing uplift forces on the joist. Details are shown in Figure 7.

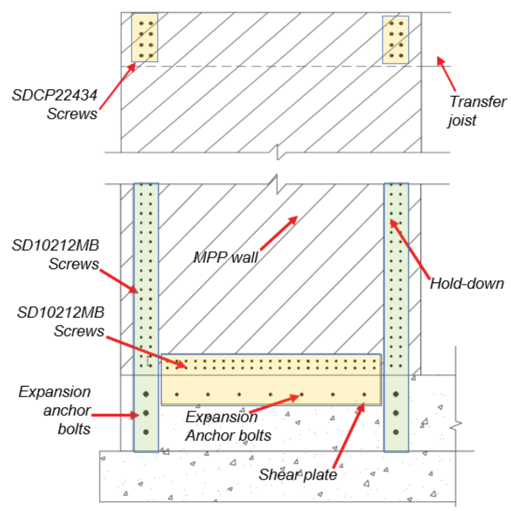

Figure 7. *Hold-down, shear plate connections, and transfer joist details.*

3.3 LWF CONTRIBUTION TO STRENGTH AND STIFFNESS

The exterior and interior walls of the existing structure contribute to the performance of the building by increasing its apparent strength and stiffness. For each MPP structural wall, a tributary length of existing exterior and interior walls and type of sheathing are considered. For the interior gypsum board walls, a tributary length of 3 m is assumed. For the exterior walls, which are assumed to be wood structural panels with 6d nails spaced at 150 mm on-center (8x8 WSP 6d @ 150 mm), a length of 1.20

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m is assumed. The contribution of the existing exterior walls with window or door openings was neglected. The resulting backbone curves based on the considerations described in this paragraph are shown in Figure 8 based on experimental data available in [10].

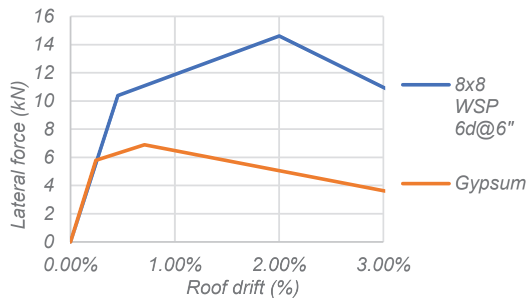

Figure 8. *Nonlinear backbone capacity of existing LWF walls based on their tributary length assigned to an MPP structural wall panel for this façade retrofit example.*

4 FINITE ELEMENT MODEL

Figure 9 illustrates a detailed Finite Element Model (FEM) that was developed to predict the response of the retrofitted building and validate the analytical methodology described in the previous sections. The FEM software used is ETABS [19]. The MPP structural walls are modelled using elastic shell elements for the panel and nonlinear fiber shell elements for the steel plate, both with a mesh size of 50 mm x 50 mm. Orthotropic material properties for the MPP are assumed, based on experimental data [20], using 12362 MPa for Ex, 2979 MPa for E_y, 206 MPa for Ez, 861 MPa for Gxy, 820 MPa for Gyz, and 145 MPa for Gzx. Multilinear elastic links are used to model the nonlinear behavior of screws and bolts. Compression-only nonlinear elastoplastic contact springs are assigned to the FEM to simulate the uplift/contact behavior of the panel with the foundation, considering a yield stress for MPP of 42.95 MPa, a corresponding yield strain of 0.0032, and ultimate strain of 0.007, with the tributary area for each spring and a plastic hinge length of twice the panel width (152.4 mm) [21].

The existing LWF structure is modelled using X-braced frames that can deform axially only and capture the horizontal story force-displacement response shown in Figure 8. Compression and tension plastic hinges are assigned for two braced frames acting in parallel to capture the shear response of the exterior and interior walls. The vertical and horizontal elements are assumed to be nearly rigid elements with moment releases at the ends.

A nonlinear static analysis is performed, using a displacement-controlled analysis, until reaching a target displacement at the top of the MPP wall of 244 mm, which corresponds to 4% roof drift ratio. The nonlinear parameters used in the software are: Newton-Raphson for positive iterations, Constant-Stiffness for negative iterations, solution scheme is Event-to-Event Only, event lumping tolerance (relative) of 0.001, maximum events per step of 200, minimum event step size of 0.001, and the maximum number of null events per step of 5.

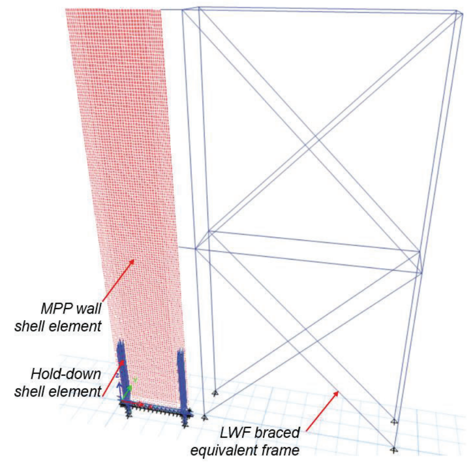

Figure 9. *Two-dimensional FEM developed in ETABS. The MPP structural wall is modelled as shell elements. Steel connections are discretized with shell elements with nonlinear materials. Screws, bolts, and contact elements are modelled as nonlinear multilinear springs. The existing LWF exterior and interior walls are modelled using equivalent braced frames.*

5 RESULTS AND DISCUSSION

5.1 CAPACITY CURVES AND OVER-STRENGTH FACTORS

Figure 10 shows the response for different models. The black line shows the FEM pushover analysis response of a model consisting of only the existing LWF structural walls (WSP walls). The yield force of this system is reached at approximately 11.9% of the seismic weight. After the system yields, damage concentrates on the first floor of the existing building, creating a soft-story mechanism. For reference, the ASCE 41-17 BSE-1E required design force (7.79%) of the LWF walls, shown as a green dash-dot line, is slightly smaller than the yield force of the existing structural wall. Using a typical NDS safety factor of 3.0, the yield point of the capacity curve should be equal to 23.4%. It can be seen in the figure that the LWF structural capacity curve is below this value, therefore the existing LWF structure requires retrofit.

Note that the nonstructural components such as the interior walls should be classified as structural elements if stiffness or strength exceeds 10% of the exterior structural walls [8]. In this case study, including the contribution of the interior walls, the yield force increases 82% from just considering exterior walls to 21.7%. Therefore, even accounting for the nonstructural elements, this LWF structure requires retrofit.

Additionally, Figure 10 shows three different FEM results. The FEM-1 curve is the capacity curve for the MPP structural wall where only the contact springs and hold-down steel plate and screw connections are modelled explicitly, or in other words, where the horizontal shear steel plate is not modelled. FEM-1 shows a higher ductility than the WSP walls, reaching the first yield point at 1.50% drift. The FEM-2 results show the capacity curve of the model with the shear steel plate and thus illustrate the effect of adding the shear plate at the base of the MPP relative to FEM-1. The addition of the shear steel plate at the base of the wall increases the lateral stiffness and strength of the system. Finally, FEM-3 corresponds to the model shown in Figure 9. The results indicate the increase in capacity required in terms of base shear (over-strength) when the contribution of the existing exterior and interior LWF, WSP, and gypsum walls, respectively, are also added to the model.

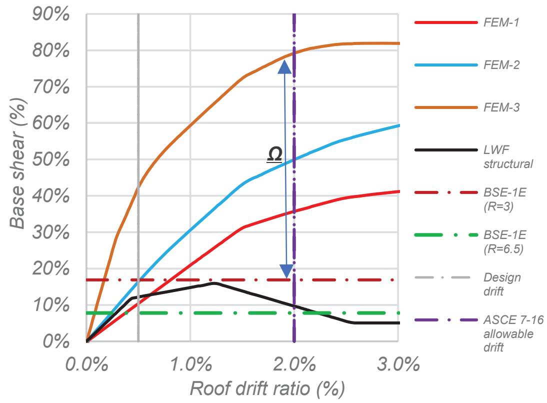

Figure 10: *Capacity curves for the following systems: WSP structural capacity (black), MPP structural wall (red), MPP structural wall with shear plate connection (blue), MPP structural wall with shear plate connection, and existing LWF (orange). The horizontal dashed-dot lines correspond to the BSE-1E force levels.*

The analysis results presented in Figure 10, illustrate that the over-strength in the system originates from two main sources. First, through the engagement of the shear plate connection at the base of the MPP, and second through the lateral stiffness provided by the existing LWF exterior and interior walls. At 2% drift, the over-strength from the shear plate contribution is approximately 143%, while the over-strength originating from the existing LWF walls is 476%.

5.2 STRESS ANALYSIS OF MPP AND STEEL PLATES

MPP walls need to resist the stresses induced by the lateral loads at higher drifts, allowing the connections to develop their nonlinear capacity. Using MPP allowable stresses: tension F_{t0} = 8.96 MPa, bending F_{b0} =13.1 MPa, compression F_{c0} =16.55 MPa, and shear F_{v0} =1.76 MPa [6]. The stress distribution in the MPP and steel plates, at the design force level (7.87 kN), is shown in Figure 11. At this level, the steel plates and the MPP show elastic properties, satisfying one of the initial assumptions in the design process. Nevertheless, the FEM shows shear stress concentration at the base of the wall at higher deformations, shown in Figure 12 for 2% drift. A follow-up experimental test will provide further validation of these results.

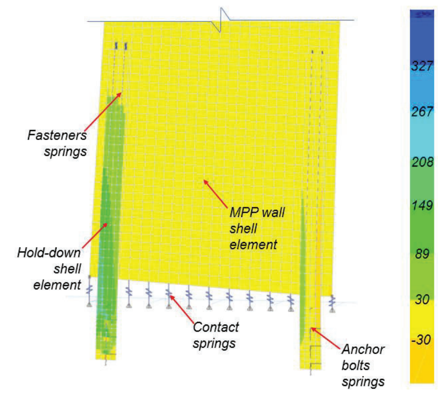

Figure 11: *Axial stresses (F22) for design level (MPa), for hold-down and MPP shell elements. Non-linear springs representing the fasteners and rocking effect at the base of the wall.*

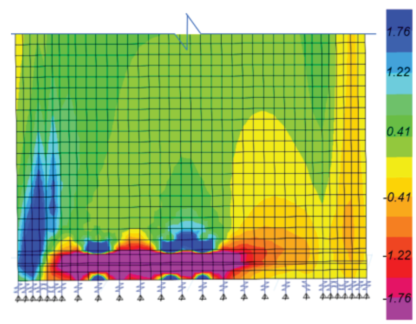

Figure 12: *Shear stress (F12) concentration in MPP at 2% drift ratio.*

5.3 ANALYTICAL MODEL AND FEM COMPARED.

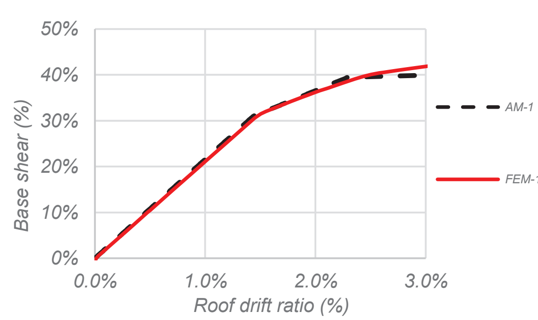

Figure 13. *Capacity curves for AM and FEM for MPP wall with hold-down connections only.*

After applying Equations 3 to 9, the analytical model describes the capacity curve of the LFRS and the yielding points of the components. Compared with the detailed FEM, the results were practically identical, obtaining the yielding points for fasteners and steel plates (see Figure 13).

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6 CONCLUSIONS

The implementation of a combined energy and seismic façade retrofit system using MPP provides a practical solution for existing LWF buildings.

A force-based lateral load calculation (ELF) from ASCE 7-16 can be used to size steel connections in a retrofit system. In addition, the lateral stiffness of the existing LWF is an important factor to consider when developing over-strength factors. The lateral transfer system from the existing structure to the new LFRS needs to be designed following capacity-based design demands, increasing the ductility of the system.

A simplified analytical model can be used to obtain the nonlinear capacity curve of the LFRS, following the geometric and mechanical properties of the wall, which was later validated by a detailed finite element model. An experimental test is needed to validate the assumptions made in the design and the modelling of the LRFS.

7 FUTURE WORK AND EXPERIMENTAL TEST

The analytical model presented in this document presents the equations for the MPP wall only, a future implementation of the existing LWF structure is planned, with the development of the stiffness matrix describing the behavior of 2- and 3-story buildings.

This project has been accompanied initially by the construction of a mock-up, real-scale specimen, built to verify the connections matched with the panels and the foundation. A follow-up experimental test is being developed in parallel, where a real-scale MPP with steel connections attached to a concrete beam will be subjected to a quasistatic lateral protocol load. From the experiment, a validation of the stiffness of the connection is expected, also showing the probable damage at the base of the MPP.

Based on ASCE 41-17, further performance assessment of the system is planned, determining performance points and implementing the methodology from this standard. Response history analysis will be also implemented to determine ductility factors and dynamic effects in the behaviour of the LFRS.

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