Influence of Nonstructural Components and Systems (NCS) on the Dynamic Behavior of Buildings

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Abstract

This research aims to address the magnitude of which the installation of nonstructural elements influences the dynamic behavior of a building. To evaluate the impact of architectural finishes on a structure’s dynamic behavior, a building located at 1908 Shattuck Avenue in Berkeley, California was assessed experimentally and theoretically by means of forced vibration testing (FVT) and computational modeling, respectively. Unique to the tested building is two stories of timber superstructure framed over a post-tensioned elevated slab which spans over six concrete-infilled columns, each of which are spliced with a base isolator at two thirds of their height. Nonstructural contributions to dynamic behavior were captured using FVT procedures at multiple installation phases of nonstructural groups. Specific dynamic changes between phases of the construction were captured using recordings of twelve accelerometers as the structure underwent forced vibration. After a normalization of the peak responses, these degrees of freedom represented a series of mode shapes of the as-built structure. The normalized mode shapes at each testing phase were then used to calibrate a suite of theoretical, computational models which in turn served to compare the building including or excluding stiffness contributions from architectural finishes. Results from the theoretical comparisons indicated error in mode shapes and frequencies when excluding nonstructural stiffness contributions in the elastic analysis of the tested structure. Additionally, theoretical modeling procedures outlined in this research serve as a precedent for modeling techniques and assumptions to account for when modeling specific components relevant to this study.

Section 1 Introduction

1.1 Overview

This study is intended to determine via experimentation and theory the distribution of mass and stiffness that various structural and architectural elements contribute to a building’s dynamic response at elastic loading levels. Assessment of structural and nonstructural elements will be performed using a combination of forced vibration analysis and computational modeling across several phases of a building’s construction schedule to isolate dynamic influence of the unique building alterations between phases.

1.2 Previous Research

Findings provided in this study were founded upon procedures, data and other previously-performed research that were adjusted or extrapolated to meet the application requirements for this study.

The initial experimental testing procedure presented in this study was predicated upon a series of forced-vibration tests of full-scale structures. This research was performed by faculty and graduate students of the Architectural Engineering Department of California Polytechnic State University, San Luis Obispo, and represents a collection of iterations to the procedure of collecting and processing the experimental modal response of a specimen structure. Forced-vibration procedures presented in Structural Damage Detection by Comparison of Experimental and Theoretical Mode Shapes (Rosenblatt, 2016) and Structural Damage Detection Utilizing Experimental Mode Shapes (Gerbo, 2014) were adhered to for the experimental assessment of this study’s specimen structure.

The theoretical modeling procedure used in this study was largely built upon common elastic theory and practical elastic stiffness derivations. A lack of cited research in this portion of the report is due to the preference for a model built upon common and widely-accepted modeling techniques, in an effort to bolster the practical relevance of the findings presented by this study. Modeling conditions that were based upon specific research included meshing densities of shell and membrane elements using Determination of the Modal Parameters of a Five Story Reinforced Concrete Structure Using Ultra-Low Level Excitation and Computational Analysis (Rendon, 2012), and an inclusion of vertical mass.
within the computational model with adherence to Implications of Vertical Mass Modeling Errors on 2D Dynamic Structural Analysis (Whalen, Archer, & Bhatia, 2004). Additionally, timber and gypsum material data used in the final suite of theoretical models was based on the practical values provided taken from the Special Design Provisions for Wind and Seismic (American Wood Council, 2008).

Section 2 Test Structure

2.1 Building Description

The specimen structure assessed for this study is a multi-use, three-story building located at 1908 Shattuck Avenue in Berkeley, California. The designed program consists of two floors of office occupancy over a ground floor commercial space. Approximate plan dimensions of the building are 90 feet from east to west and 35 feet from north to south. Floor heights for the second, third and roof levels are approximately 16, 10 and 12 feet, respectively. For the purposes of this report, naming conventions for the second and third levels are the podium and mezzanine levels, respectively.

Structural gravity systems used at the two upper floors of office program include timber framing with supplemental steel gravity frames. Beneath this timber superstructure is a 14-inch thick post-tensioned reinforced concrete podium slab which is supported by six cantilevered 24-inch diameter pipes with concrete infill. To complete the gravity system, the cantilevered columns bear upon a network of six-foot wide by three-foot deep post-tensioned concrete grade beams. Timber shear walls serve as the lateral system for the timber superstructure which is laterally supported by the cantilevered columns beneath the podium. Additionally, the cantilevered columns are spliced at approximately two thirds of their height with triple friction pendulum base isolators. It should be noted that these isolators are not activated during the experimental testing of the structure, as vibration levels within the superstructure are too low to overcome static friction resistance within the isolators.

Nonstructural elements that contribute most to the mass and stiffness distribution within the timber superstructure include exterior standing seam metal cladding, interior gypsum drywall, and two-inch thick topping slabs at the podium and mezzanine floors.

2.2 Testing Phase Description

Figure 2-1 depicts the four stages within the construction schedule during which experimental testing was conducted. These phases were selected to isolate influence on global dynamic behavior of various elements for calibration and comparison with computational models.

The first phase of testing occurred on May 30th, 2015. At this time in the construction schedule the building consisted of merely the concrete podium, concrete-infilled cantilevered steel columns and grade beams beneath a slab on grade. The isolators during this phase of the testing were constrained by four bolted plates at each isolator for construction purposes.

The phase two assessment was conducted on August 8th, 2015, as the timber superstructure was completed and consisted of all wall, floor, and roof structural membrane elements. The purpose of this phase was to capture a base response of the timber structure for comparison to future tests involving the addition of nonstructural elements.

By February 2nd, 2016, all cladding, roofing, and interior drywall elements had been installed. Phase three was performed with the intent of observing the effects of exterior cladding, interior finishes, and roof finishes, and their effect on the mass and stiffness distributions of the system.

The last dynamic response assessment was conducted on April 4th, 2016. At this time in the construction schedule, two inch-thick normal-weight concrete topping slabs at the podium and mezzanine levels had been poured, and the building’s mass and stiffness distributions could be considered very close to their final state. This final phase of testing served as an additional investigation of the structure’s change in modal behavior due to a relatively significant increase in uniformly distributed mass at the podium and mezzanine levels.

Section 3 Theory and Methodology

This section is intended to provide a summary of the relevant structural analysis theories and methods that are most fundamental to the development of this research.

3.1 Response Amplification
At resonant response during harmonic excitation, a dynamic body exhibits an amplification of displacements, velocities, and accelerations relative to the same excitation load applied statically. This effect is known as dynamic amplification due to harmonic loading and is summarized by the ratio of amplified peak dynamic response to the static response. This characteristic of harmonic dynamic behavior allows the ultra-low forced vibration testing procedure to distinguish modal response at resonant frequency from ambient vibrations. As shown in Figure 3-1, the response amplification factors at resonance – or a frequency ratio of one – for acceleration, velocity, and displacements can act as significant multipliers on physical accelerations depending on the damping properties of the structure.

3.2 Mass-Weighted Modal Assurance Criterion

To evaluate similarity or lack thereof among a given set of mode shapes, use of the mass-weighted modal assurance criterion (MWMAC) is implemented. The MWMAC is used in two applications for this study. The first function of the MWMAC is within the experimental results assessment of this report provided in the experimental testing procedure, where the MWMAC quantifies a level of contamination between measured mode shapes. The second MWMAC application is in the comparison of theoretical and experimental results, as the MWMAC is used to evaluate levels of correlation among experimental and theoretical mode shapes. Formulation of the MWMAC is provided in Equation 3-1.

\[
MWMAC_{ij} = \frac{(\Phi_i^T M \Phi_j)^2}{(\Phi_i^T M \Phi_i) (\Phi_j^T M \Phi_j)}
\]

Equation 3-1

Where,

- \(\Phi_i\) = \(i^{th}\) mode shape vector
- \(\Phi_j\) = \(j^{th}\) mode shape vector
- \(M\) = system mass matrix

The system mass matrix of Equation 3-1 consists of a diagonal matrix with dimensions equating to the number of degrees of freedom recorded. Based on the placement of each accelerometer during testing, each degree of freedom has a tributary area and mass associated with its readings. For the specimen structure used in this study, 12 accelerometers were placed throughout the structure with tributary areas summarized in Figure 3-2.

Using the tributary areas depicted in Figure 3-2, a series of mass matrices are created based on the materials that are present during each phase of testing. Weights and masses associated with each degree of freedom are provided in Table 3-1. Degrees of freedom oriented in the east to west direction are indicated by the bold columns in Table 3-1 and are representative of the total weight and mass at each floor. It
Section 4 Experimental Testing Procedure

For each phase of forced vibration testing (FVT), equipment was typically setup on the Friday prior to testing after construction teams had left, and the FVT data acquisition procedure would then occur the following Saturday and last for approximately nine hours. This section covers the testing setup and data acquisition procedure to take place for each phase.

### 4.1 Forced Vibration Testing

The FVT procedure consists of a series of steps performed for capture of modal behavior of the physical structure. For a given mode, behavior includes natural frequency, damping, and mode shape, each of which are used to evaluate the dynamic response of the structure. This section outlines the steps in which the FVT process was conducted.

#### 4.1.1 Equipment Setup and Layout

The FVT procedure instrumentation and layout can be divided into data acquisition and forcing equipment based on the FVT function served. In addition to the equipment that is summarized in Table 4-1, wiring, mounting equipment, a power source, and tools are also provided during the testing procedure.

### Table 3-1 Degree of Freedom Weights & Masses

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<td>35</td>
<td>27</td>
<td>20</td>
<td>99</td>
<td>31</td>
<td>43</td>
<td>25</td>
</tr>
</tbody>
</table>

### Table 3-1 Degree of Freedom Weights & Masses

<table>
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<td>20.92</td>
<td>16257</td>
<td>3.85</td>
<td>1.30</td>
<td>1.10</td>
<td>0.83</td>
<td>0.62</td>
<td>3.08</td>
<td>0.97</td>
<td>1.33</td>
<td>0.78</td>
</tr>
</tbody>
</table>

**Placement of equipment was governed by the desired degrees of freedom to be captured. For the assessed structure, three accelerometers were used at the podium level, five were placed just beneath the roof. These locations and quantities were based on the quantity of immediately available accelerometers and the structural composition of each floor.**

At the podium level, rigid plate dynamic behavior was expected, therefore, two accelerometers at the geometric center of the podium and a third placed eccentrically to the center were enough to capture translation and rotation of the podium slab. At the mezzanine level, a greater quantity of accelerometers were needed to capture stiffness distribution of the flexible diaphragm, which, for analytical purposes, was divided in half based on the diaphragm’s component aspect.
Due to the slenderness in plan of the smaller portion of the mezzanine diaphragm, significant flexible behavior was expected in north to south dynamic response. This portion of the diaphragm was discretized into two segments by an accelerometer placed at the mid-span of the smaller portion of the mezzanine diaphragm. An accelerometer was also located at the east and west edge of each half of the mezzanine, leaving the last accelerometer at the mezzanine’s re-entrant corner to capture the east to west translation of the mezzanine level; the use of only one accelerometer in this direction was once again justified by the aspect ratio of the mezzanine level diaphragm proportions. At the roof level, a uniform distribution of diaphragm stiffness was expected given the regular rectangular shape, so the roof diaphragm was discretized into two halves with a ceiling-mounted accelerometer at mid-span. As with the mezzanine level, one accelerometer was used to capture east to west response due to the stoutness of the roof diaphragm in this direction.

4.1.2 Ambient Vibration Assessment
With all equipment mounted, connected, and powered, the FVT process would begin with an ambient vibration assessment. The structure’s ambient response served to predict modal frequencies and establish estimates for upper and lower cutoffs for bandpass filtering of the recorded signals.

4.1.3 Frequency Sweeping
Following the ambient vibration assessment, a low-resolution, wide bandwidth-filtered sweep was performed near peaks provided by the ambient frequency power spectrum. These frequency sweeps were intended to give a rough estimate for the frequency domain of the modal response of interest. Once plots were formed that capture peak response at one hertz above and below the rough natural frequency, a high-resolution, small-bandwidth-filtered frequency sweep at the peak of the wide sweep was conducted to provide a more precise modal natural frequency. For each frequency sweep plot provided in the experimental results, both narrow and wide sweeps were combined for each modal response for ease of observation. An example of the combined frequency sweep can be observed in Figure 4-1, complete with a shaded region indicating the magnitude of damping.

4.1.4 Mode Shape Capture
At the frequency associated with the peak modal response as determined by the frequency sweeps, a synchronized readout of the accelerometers was recorded. By capturing the synchronized output of all accelerometers over a time interval, the response can be analyzed for phase lag or verification of the expected mode shape behavior.

4.2 Experimental Conclusions
Each FVT assessment involved varying levels of difficulty in locating the first three resonant modal responses of the structure. The primary issue exhibited in all testing phases involved proximity of the peak responses, which in turn led dynamic responses that were visually impure and contaminated with undesired modal behavior. A primary explanation for the proximity of the peak responses was the regular, concentric layout of the podium slab lateral system. The near-identical translating stiffness in both the transverse and longitudinal directions coupled with the lack of eccentricity in column placement causes the mass-dominant podium slab to exhibit three primary modes with relatively similar mass to stiffness ratios.

To observe changes in frequency sweeps across all phases, Figure 4-2 through Figure 4-5 provide normalized comparisons of targeted modes.
Figure 4-2 Normalized Transverse Frequency Sweep, All Phases

Figure 4-3 Normalized Longitudinal Frequency Sweeps, All Phases

Figure 4-4 Normalized First Rotational Frequency Sweeps, All Phases

Figure 4-5 Normalized Second Rotational Frequency Sweeps, All Phases
The following two figures, Figure 4-6 and Figure 4-7, summarize the detected modal frequencies and damping ratios, respectively, for all frequency sweeps. Damping ratios, however, do not exhibit prominent trends, thus indicating inconclusive experimentally-derived damping ratio results.

Table 4-2 Modal Natural Frequency Summary

<table>
<thead>
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<th>Phase</th>
<th>Mode Frequency [Hz]</th>
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<tr>
<td>I</td>
<td>4.47 4.06 6.30</td>
</tr>
<tr>
<td>II</td>
<td>3.89 4.62 8.23</td>
</tr>
<tr>
<td>III</td>
<td>3.46 3.73 3.83 8.99</td>
</tr>
<tr>
<td>IV</td>
<td>3.89 4.13 4.68 9.71</td>
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Table 4-3 Damping Ratio Summary

<table>
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<th>Phase</th>
<th>Mode Damping Ratio [-]</th>
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<tr>
<td>I</td>
<td>0.028 0.008 0.047</td>
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<tr>
<td>II</td>
<td>0.013 0.017 0.016</td>
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<tr>
<td>III</td>
<td>0.014 0.015 0.011 0.028</td>
</tr>
<tr>
<td>IV</td>
<td>0.013 0.021 0.031 0.026</td>
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Section 5 Theoretical Modeling Procedure

For comparison of experimental results with structural dynamics theory, the full structure was modeled in CSI’s ETABS structural analysis software (CSI 2015); ETABS was the program of choice for this study due to its wide-spread use in practice. Unique to this study’s modeling process was the adjustment of specific modeling parameters within practical reason in order to calibrate the computational model dynamic behavior against mode shapes and frequencies acquired through experimental testing. Computational model geometry, section sizes, and detailing were based on the as-built structural drawings obtained from Tipping Structural Engineers.

5.1 Stiffness Derivations

With adherence to structural drawings throughout the modeling procedure, it was assumed that the structure’s geometry and section sizes were relatively identical between the as-built structure and computational model. Therefore, material property accuracy became a variable of significant importance in distributing stiffness throughout the structure. While material properties for steel are relatively reliable, properties of concrete and timber elements in particular require significant consideration due to time-dependent and orthotropic attributes, respectively.
Concrete Stiffness

Concrete mix designs are typically specified for a minimum compressive strength after 28 days of curing; however, the actual strength of concrete was dependent on the duration in which it has been allowed to cure (Wight & MacGregor, 2012). To account for the varying stiffness of concrete associated with the testing date of each phase, the compressive strength was derived using Equation 5-1.

\[
f'_{c(t)} = f'_{c(28)} \left(4 + 0.85t\right)
\]

Equation 5-1
(Wight & MacGregor, 2012)

Where,
\[f'_{c(28)} = \text{compressive strength [psi] at 28 days}\]
\[t = \text{duration [days] between construction and testing}\]

Once compressive strength had been determined for a given duration of curing, the modulus of elasticity, \(E_c\), was determined per ACI 318-11 Section 8.5 with \(f'_{c(t)}\) used in lieu of \(f'_{c}\). Concrete strength and modulus of elasticity values for this study are summarized in Table 5-1; phases one and four are divided into two durations given the construction time required between foundation, podium, and topping slab concrete pours.

5.1.2 Structural Sheathing and End Post Stiffness

Sheathing stiffness parameters are provided by Table C4.2.2A of the AWC SPDWS-2008. Shear stiffness values pertinent to this study are summarized in Table 5-2.

Table 5-1 Phase-Dependent Concrete Stiffness Values

<table>
<thead>
<tr>
<th>Phase</th>
<th>(t) [days]</th>
<th>(f'_{c(t)}) [psi]</th>
<th>(E_c) [psi]</th>
<th>(f'_{c(t)}) [psi]</th>
<th>(E_c) [psi]</th>
<th>(f'_{c(t)}) [psi]</th>
<th>(E_c) [psi]</th>
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<td>6938</td>
<td>4747762</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>V</td>
<td>&gt;270</td>
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<td>6938</td>
<td>4747762</td>
<td>2883</td>
<td>3060690</td>
</tr>
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</table>

5.1.3 Nonstructural Cladding and Finish Stiffness

The building for this study was finished with 5/8" Gypsum board throughout the majority of interior walls and ceilings. Per Table C4.2.2B of the AWC SPDWS-2008, the shear modulus for Gypsum board panels is 40,000 lbs/in². The building’s exterior walls were clad with 22 gauge, or 1/32 inch thick ASTM B370 cold-rolled copper manufactured standing seam panels (Fernau & Hartman Architects, Inc., 2014). Copper’s modulus of elasticity is 17,000,000 lbs/in²; when combined with a Poisson’s ratio of 0.33, the derived shear modulus is approximately 6,390,000 lbs/in². Implementation techniques of these material stiffness values within ETABS are described in depth in Section 5.2.5.
5.2 Modeling Assumptions

5.2.1 Modal Analysis and Mass Application Parameters

Theoretical mode shapes and frequencies were produced via modal analysis of the structural model. For this study, mass sourcing parameters for the modal analysis were modified to account for more realistic experimental conditions involving the inclusion of vertical mass and avoiding traditional lumped-mass techniques. By including vertical mass in modal analysis, primary natural frequencies were reduced by approximately 0.2 hertz or 5.1 percent, and brought closer to experimental frequencies.

Mass application to the model was performed using a combination of self-weight and additional area mass assignments for frame and shell elements, respectively. Upon assigning mass to wall and floor elements, the mass was distributed to the nodes of each mesh element within ETABS, thus capturing mass distribution much more accurately than concentration of mass at discrete nodes. By distributing mass throughout the system in this manner, mode shapes and natural periods of vibration were free of localized stress and displacement errors caused by lumped-mass principles (Whalen, Archer, & Bhatia, 2004). To verify total mass quantities applied to the system at each phase, a vertical acceleration load case was routinely checked and verified with mass summaries of Table 3-1.

5.2.2 Meshing Density and Element Continuity

As depicted in Figure 5-1, mesh density of shell and membrane elements consisted of analysis mesh edge dimensions no larger than 24 inches. This discretized mesh was required for convergence of modal frequencies by reducing inaccurate stress distributions associated with larger meshes (Rendon, 2012). To create an interface between frame and meshed elements at the podium level, additional members are provided to distribute large concentrated forces from the column frame elements to a series of locations within the slab mesh as shown in Figure 5-2.

5.2.3 Foundation Springs and Increased Stiffness at Joint Zones

With an initial assumption of fully restrained column bases at grade level, the phase one computational model exhibited a longitudinal frequency that was 0.25 hertz greater than experimental results. Given high-confidence in experimental phase one longitudinal results, stronger correlation of theoretical and experimental results was desired prior to
adding the superstructure components of phases two through four.  
To incorporate more flexibility into the phase one computational model and effectively reduce natural frequencies, grade beams were modeled with line springs as vertical support to represent soil stiffness. From the available geotechnical boring data, soil conditions included stiff to hard silty and sandy clays and dense to very dense clayey and gravelly sands at the proposed grade beam elevation (Fugro Consultants, Inc., 2012). Young’s modulus values associated with these soil compositions range from 1825 to 4500 pounds per square inch in stiffness (Geotechdata.info, 2013); when combined with a grade beam bearing width of 72 inches, line spring stiffness ranges from 130 to 320 kip/in per inch of beam. A soil stiffness of 1825 psi was chosen based on frequency results, resulting in a line spring stiffness of 131.4 kips/in/in. To properly apply foundation springs at the bearing face of the foundation, grade beam insertion points were assigned to the bottom center of the cross section. This geometric feature of the model also allowed the column frame section to extend into the grade beam joint section to reflect as-built conditions. Additionally, the as-built five-inch thick slab on grade was investigated; however, the grade beam effective flange width per Section 8.12 of ACI 318-11 in combination with a rigid diaphragm at grade level provided negligible contributions to the overall system stiffness. The slab on grade at the ground level was therefore ignored for modeling simplicity.

5.2.4 Wall Modeling

Timber walls were modeled using shear and flexure derivations provided by equation C4.3.2-1 of the AWC SPDWS-2008. This equation is a four-part deflection function that includes non-linear nail slip and post anchorage components, both of which were neglected given the low amplitude of structural excitation.

\[ \delta_{SW} = \frac{\gamma v h^2}{E A b} + \frac{v h}{G_{sv}} + 0.75 h e_n + \frac{h}{b} \Delta_a \]

**Equation 5-2**

(American Wood Council, 2008)

Where,

- \( v \) = induced unit shear [plf]
- \( h \) = shear wall height [ft]
- \( E \) = modulus of elasticity of end posts [psi]
- \( A \) = area of end posts cross – section [in\(^2\)]
- \( b \) = shear wall length [ft]
- \( G_{sv} \) = shear stiffness [lbs/in per depth of panel per AWC SPDWS 2008 Table C4.2.2D]
- \( \Delta_a \) = vert. elongation of wall anchors [in] at load \( v \)
- \( e_n \) = nail slip [in] AWC SPDWS 2008 Table C4.2.2D

Using the bending and shear components of this equation, wall deflections were calculated for a single wall with end posts and compared to results of a similar computational model prior to carrying out full superstructure modeling. Wall shear behavior was represented by a membrane element with in-plane shear stiffness equal to the derived shear modulus of wall sheathing per Table 5-2 and a minimum analysis mesh edge dimension of 24 inches. Bending stiffness in the model was captured by an end post at each end of the shear wall with an assigned stiffness from Table 5-3, and top and bottom rotational restraints released or, “pinned.” These post elements are visible in Figure 5-4, with additional posts provided to control out-of-plane wall behavior as discussed in Section 5.2.5.
Computationally modeled walls that adhered to the stated design techniques exhibit displacements within five percent of the calculated displacement per equation C4.3.2-1 of the AWC SPDWS-2008.

5.2.5 Miscellaneous Modeling Adjustments

In addition to the stated modeling parameters implemented within the theoretical model, a series of supplemental adjustments were applied to more accurately represent experimental mode shapes and frequencies. Given the chosen method for mass application to the theoretical model on a tributary area basis, walls and floors were required to support out-of-plane loading to prevent primary mode shapes from becoming dominated by wall and floor out-of-plane flexibility. To provide out-of-plane stiffness to the membrane elements without contributing undesired global stiffness, plate elements were incorporated into the layered wall element. This plate element contributes only to the out-of-plane stiffness of the layered wall and was assigned a material and thickness that roughly reflected the out-of-plane flexural stiffness of the as-built wall or floor. This rough order of magnitude in out-of-plane stiffness was permitted over a more detailed stiffness derivation as the out-of-plane stiffness was merely required to distribute out-of-plane mass to the wall supports without dominating primary mode shapes with wall flexibility. End posts within the walls were also assigned magnified flexural stiffness properties to resolve out-of-plane loading from wall elements into lateral loading at floor and roof membrane elements. As verification that these added out-of-plane elements do not influence the structure’s global stiffness, changes in frequencies with the out-of-plane elements toggled on and off were routinely checked. This was done by executing the model without out-of-plane stiffness elements applied, and then observing the first several modes with high mass participation ratios to ensure that their frequencies were within three percent of the same model’s results with out-of-plane elements applied.

Additionally, for this study, two modeling techniques typically adhered to in practice were not implemented in the theoretical model. The first is the use of rigid diaphragm features, which had negligible effect on the rigidity of the podium slab and would have inhibited flexible behavior if applied at the roof and mezzanine levels. The additional practical technique avoided was a reduction in the moment of inertia for sections composed of concrete. Given that all primarily concrete elements created in the theoretical model were post-tensioned, it is assumed that the concrete sections would not exhibit cracking that would otherwise result in a reduced moment of inertia.

Section 6 Theoretical Validation and Nonstructural Study

This section addresses the reliability of the computational results and evaluated whether or not the model was adequate for a study of the nonstructural component influences. Once proven valid, the computational model was then used to extract modal frequencies for select phases with and without stiffness contributions included.

6.1 Theoretical Accuracy

Use of the practical material stiffness values and modeling assumptions set forth in Section 5 resulted in reasonable to excellent correlation with experimental results. To rapidly compare results and avoid an over-abundance of MWMAC data, experimental and theoretical modal frequency results are provided in Table 6-1 and Figure 6-1.

<table>
<thead>
<tr>
<th>Table 6-1 Theoretical and Experimental Frequency Summary</th>
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<tr>
<td>Phase</td>
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<td></td>
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<tr>
<td>Transv.</td>
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<td>Longit.</td>
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<td>1st Rot.</td>
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From the frequency summary, there are a number of observations to be made regarding points of considerable correlation and sources of error prior to declaring the computational models fit for further theoretical study. Data points of excellent correlation begin with the experimental and theoretical transverse frequencies, which maintain the most consistent agreement across all phases with a peak frequency deviation of 0.15 hertz, or 4.3 percent error at phase three. This data appears to be the best captured and most accurately modeled from a frequency standpoint of the first three modes. The next point of significance is in the agreement of the experimental and theoretical translating modes across phases one and two, as the ordering of the transverse and longitudinal modes does in fact switch. Phase two frequencies, however, indicate sources of error present in the experimentally captured frequencies. Proximity of the experimental, longitudinal and theoretical, rotational frequencies of phase two suggests that the experimental longitudinal frequency is closer to the experimental transverse frequency, and the experimentally recorded longitudinal mode is in fact the experimental first rotational frequency. Figure 6-2 provides additional frequency correlation regarding phases capable of yielding a second rotational mode. Frequency trends across phases two through four display similarity between experimental and theoretical second
rotational frequencies despite the increased frequency range at which second rotational modes were recorded. Regions of inter-phase study will occur between phases two to three and three to four and have been highlighted in Figure 6-1 and Figure 6-2.

6.2 Nonstructural Influence
Critical to this study is the magnitude of mass and stiffness contribution of nonstructural elements applied to the structure at various construction phases. While the experimental results displayed a combination of the mass and stiffness variables through each phase, the theoretical model –validated with the experimental results – provided insights into the magnitudes in which these variables influence the structural response.
To conduct the nonstructural study within the computational model, stiffness properties of nonstructural elements were assigned a value of zero to isolate the influence of other elements that retained their stiffness. The nonstructural element’s mass, however, remained constant through each inter-phase study, thus simulating a practical scenario of including the mass of a nonstructural finish without incorporating its stiffness.

Results of the nonstructural stiffness study are provided in Figure 6-3. Evidently, the nonstructural components fail to significantly influence the primary modes due to the location of the applied nonstructural elements; given the mass-dominant dynamic characteristics of the podium slab, the nonstructural elements would require application at the podium level and below to influence the first modes to a greater degree. To mitigate the influence of the podium slab mass and target the super structure, it becomes necessary to observe nonstructural influence on the second torsional modes of phases one through four. Figure 6-4, provides the results for all modes of the nonstructural study.

Table 6-2 and Table 6-3 quantify the level of error in frequency resulting from omitting stiffness contributions of nonstructural elements. These tables are intended to simulate practical design scenarios of neglecting stiffness contributions of nonstructural elements.
As indicated by Table 6-2 and Table 6-3, significant percent errors are associated with the second rotational mode in both inter-phase studies as the influence of the nonstructural elements is exemplified. By virtue of the magnitude of these errors, the nonstructural study results suggest the importance of including nonstructural stiffness for accurate modeling of as-built conditions. Similarly, accuracy of the modeling parameters and stiffness applications used in Section 5 is supported by the second torsional frequency data of Figure 6-4; as the various nonstructural element stiffness contributions are included, an evident convergence in frequencies is observed in both inter-phase studies.

Section 7 Conclusion

Given the complexity involved in correlating experimental and theoretical modal behavior for a full scale structure, the procedure outlined in this study proved effective for evaluation of the nonstructural component influence on the building’s modal response. This evaluation, consisting of extensive experimental testing and theoretical modeling, yielded results
in mode shapes and natural frequencies that displayed an evident change in stiffness as a result of the installation of various nonstructural elements.

7.1 Experimental Summary

The experimental results of this research served as an attempt to capture the most unique aspect of this study – the evolving dynamic behavior of the specimen structure through stages of its construction. Despite a few data point outliers of the experimental results, primarily associated with phase two, the experimental procedure was generally successful in demonstrating global trends in the change of frequency and mode shapes across each phase of testing. The derived damping ratios failed to exhibit any trends from phase to phase, however, their magnitudes fell within a range of three to half a percent. This range of damping ratios is roughly similar to the expected elastic damping ratios for elements within the podium structure, the portion of the structure which dominated the primary mode shapes.

The most significant shortcoming of the experimental procedure of this study was that the secondary transverse, longitudinal and rotational mode shapes were not captured. As discussed in the experimental and theoretical conclusions, the secondary rotational mode exhibited an increase in stiffness from phase three to four of 0.76 hertz, or 8.45 percent. This increase in frequency is in stark contrast to the decrease of the primary mode frequencies which were dominated by the podium mass, and indicates that the second rotational mode was able to isolate dynamic behavior of the timber superstructure. By this reasoning, secondary transverse and longitudinal modes would have similarly isolated influence of the timber superstructure mass and exhibited an increase in frequency from phases two to three.

The theoretical model was then evaluated against the high-confidence portion of experimental data, and a reliance upon practical assumptions and derived stiffness parameters was used for constructing the model. Additionally, the computational model was used for further exploration of the structure’s change in dynamic behavior through construction phases and performed a nonstructural study that would otherwise not be possible solely through experimental analysis.

7.2 Theoretical Summary

Modeling procedures applied to the theoretical portion of this study were intended to deviate minimally from those adopted by practitioners in order to maximize relevance of the study. Despite these intentions of minimal deviation, this study exhibits a series of extensively modeled characteristics including but not limited to mass distribution techniques, foundation springs and timber shear wall implementation that are not characteristic of ordinary modeling procedures. However, these unique elements and their chosen methods of application were predicated upon a number of justified assumptions such as calibration with experimental modal behavior, empirically-derived material parameters and theoretical extrapolations of elastic behavior, all of which have been thoroughly documented in this report.

As shown in Section 6, the computational model generally matched experimental dynamic behavior. Comparative MWMAC values were as high as 0.9991 for the podium assembly, 0.9394 for the final structure, and averaged 0.9252 without phase two included; experimental mode shape results for phase two were susceptible of significant error and therefore resulted in poor correlation with phase two theoretical mode shape results. Computational frequency results yielded increased margins of error relative to the comparative MWMAC results, but the transverse experimental and theoretical frequencies, a mode that appeared moderately accurate across all phases, exhibited a peak frequency difference of merely 0.15 hertz or a 4.2 percent error. From the results of Section 6.1, error between the theoretical and experimental modal results was relatively low for high-confidence data comparisons. Therefore, theoretical modeling procedures used in this study proved valid for capture of the localized mass and elastic stiffness increases that resulted from added nonstructural components to the specimen structure.

7.3 Practical Relevance

Ultimately, the purpose of this study is to evaluate the effect of nonstructural elements on the specimen structure’s modal behavior. Modeling procedures and results provided in this study may be applied to structural designs in order to produce a more efficient structure by including the elastic stiffness contributions of nonstructural elements for resistance of lateral loads.

As a sample of the magnitude of increase in localized elastic stiffness contributed by a nonstructural finish, the gypsum board finish may be evaluated with stiffness values provided by the Special Design Provisions for Wind and Seismic (American Wood Council, 2008). For the structure used in this study, an added layer of 5/8th-inch thick gypsum board increased a wall membrane element’s total shear modulus by 40,000 lbs/ft², or a 45-95% increase depending on the wall’s structural sheathing layer thickness and quantity. It should be noted that these stiffness increases are only valid under elastic loading. Additionally, this stiffness increase is not directly proportional to an increase in frequency as the non-structural elements also contribute mass to the system.

A global stiffness increase resulting from the inclusion of nonstructural elements was then evaluated. The structure’s relative stiffness is measured by a change in modal frequency as stiffness contribution from various nonstructural elements are toggled on and off. From the theoretical nonstructural study for primary modes as summarized by Figure 6-3, frequency errors resulting from the absence of nonstructural stiffness peak at a mere 0.31 hertz, or an 8.59% error. The second torsional mode, however, exhibited a much more significant
frequency difference of 2.55 hertz, or 27.3% error as the mass-dominant characteristic of the podium is diminished. These larger percent errors shown by the second torsional mode more adequately align with the significant magnitude of local stiffness increase caused by the gypsum finish alone and therefore more appropriately capture the scale in which nonstructural components contribute to the global stiffness of the specimen structure.

Section 8 References


