Lake Oswego City Hall
Seismic Assessment

Prepared January 2017
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1. Introduction
1. INTRODUCTION

PROJECT TEAM

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EXECUTIVE SUMMARY

The primary purpose of this report is in support of Mackenzie’s efforts to help the City of Lake Oswego make some informed decisions about the re-cladding of the existing City Hall building. During preliminary discussions, the question of what alternative veneer/cladding materials could be used on the building came up. Changing the material of the skin system of a building alters the seismic mass of a building and can in some cases trigger mandatory seismic upgrades. The primary purpose of this study was to determine what material types and wall assemblies may be possible without triggering mandatory seismic upgrades. The second purpose was to conduct an ASCE 41-13 seismic evaluation of the building and develop a seismic retrofit scheme which would allow for the use of heavier cladding materials.

First, an analysis of the building’s current mass was conducted to determine how much additional mass could potentially be added to the exterior skin of the building on both a global and localized basis before triggering seismic upgrades. While this task was completed, its results were nullified by the following seismic evaluation.

Following the veneer analysis, an ASCE 41-13 Tier 1 seismic evaluation of the existing City Hall building was conducted. As part of the review, a site visit was conducted on November 22th, 2016. The City Hall building was evaluated at a Life Safety Performance Level rather than an Immediate Occupancy Performance Level as it is understood the Police Department will soon be removed from the building, resulting in a change in the required performance level.

During the seismic evaluation, it became apparent that the building suffers from both global and localized seismic deficiencies. While some of these deficiencies would not have been classified as such at the time of the building’s construction (due to changes in codes and the level of seismic hazard here in Oregon over the years), it would appear that some of them would not have been permitted by the governing building code in force at that time, the 1982 Uniform
1. INTRODUCTION

Building Code (UBC). This last fact, that portions of the building may not have been in compliance with the building code at the time of its construction, likely nullifies the option to slightly increase the mass of the exterior cladding of the building without requiring seismic upgrades.

The rest of this report will: detail the veneer study performed; address the proposed approach for modifying the building’s skin system without requiring seismic upgrades; present the findings of the seismic evaluation including both global and local deficiencies; and identify one set of possible seismic retrofits. This report will then be used to help prepare a cost estimate of the seismic retrofits required to re-clad the building with something other than its existing skin system. That cost estimate is presented in a separate report.
As noted above, one of the original intents of this report was to determine what exterior wall assemblies might be permitted without triggering mandatory seismic upgrades. To do so, Mackenzie anticipated using provisions of Chapter 34, Existing Buildings and Structures, of the current building code, the 2014 Oregon Structural Special Code (OSSC). Section 3401.1.2 permits the replacement of existing building materials with “like materials” provided the replacement or repair of such materials does not constitute structural repairs or alterations. Mackenzie interprets this section of the building code as allowing the City Hall to be re-clad with a similar or “like” system to that which is already in place: exterior insulating finish system (EIFS).

In order to utilize another material, Section 3404.4 of the OSSC permits alterations which impact “existing structural elements carrying lateral load” provided such impacts do not constitute more than a 10% increase in the demand-to-capacity ratio of the elements. Mackenzie intended to use this provision to present the City of Lake Oswego with alternate cladding materials and wall assemblies which may have weighed slightly more than the existing EIFS wall assembly. However, the use of this provision relies on several key assumptions. First, the authority having jurisdiction (AHJ) and its building official must agree with the interpretation. In Mackenzie’s experience, there is some variability amongst building officials in how this provision is interpreted and the magnitude of changes they will permit. Typically, their interpretation is based on a second assumption: that the building was in compliance with the building code when it was constructed. The purpose of this Section 3404.4 is to allow building owners to make slight modifications without overly penalizing them for changes in the building codes over time. However, buildings which were never in compliance have been identified by building officials as being ineligible to use this section.

Based on the findings of the seismic evaluation (presented later in the report), Mackenzie does not believe the building was in compliance with the 1982 UBC at the time of construction and thus would not be eligible to utilize Section 3404.4 to justify the replacement of the existing veneer system with a heavier version.

A seismic mass takeoff and vertical force distribution were completed for the building in its current configuration. These analyses were completed both considering and excluding the first-floor podium. The hollow core planks, steel framing, and concrete slabs of the first-floor podium weigh almost as much as the rest of the upper building combined and are braced on two sides by surrounding soil. The consideration of the podium has a large impact on the vertical distribution of seismic forces through the structure. However, the allowable change in wall assembly mass was relatively consistent.
2. VENEER STUDY

It was found that increasing the exterior wall assembly weight from its current value of ~8.3psf up to ~17psf would limit the increase in seismic forces to less than 10% at any level. These values include everything from the inside finish material through to the exterior cladding. While the original intent was to find the largest increase in wall assembly weight that would not trigger a mandatory seismic upgrade, due to the findings of the seismic evaluation and the associated interpretation of section 3404.4, Mackenzie does not anticipate such an increase in wall assembly weight would be permitted by the building official and does not recommend that this approach be pursued.
3. Seismic Assessment
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3. SEISMIC ASSESSMENT

ASCE 41-13 ANALYSIS BACKGROUND

The seismic evaluation was conducted using ASCE 41-13 Seismic Evaluation and Retrofit of Existing Buildings. This document is not a code, but a nationally-recognized standard used by engineers to evaluate and retrofit existing buildings. Previously, there were two separate documents for the evaluation and retrofit of existing buildings: ASCE 31 and ASCE 41, respectively. Recently, these documents were combined into the updated version, ASCE 41-13, to help alleviate some of the inconsistencies that occurred when a building made the transition from seismic evaluation to the retrofit/upgrade process. New building codes include many provisions that require or encourage design and detailing practices that improve the seismic performance of a building, including regular building configuration, ductile detailing, and high quality materials. Most existing buildings will not meet these criteria that new construction would be designed and detailed for; however, it is recognized that these existing structural systems still have capacity that the new code doesn’t recognize. The ASCE 41-13 includes guidelines and methods for evaluating the capacities of existing structural elements that might otherwise be insufficient when analyzed using the new building code provisions.

Within the ASCE 41-13 there are four building Performance Levels (lower to higher performance): Collapse Prevention (5-E), Life Safety (3-C), Immediate Occupancy (1-B), and Operational (1-A). Unless otherwise required by code (i.e., emergency response facilities, prisons, or other essential facilities), the majority of buildings are designed for the Performance Level of Life Safety (LS). The LS performance level is meant to ensure the safety of building occupants; however, buildings with this performance level will likely experience significant damage that may or may not be repaired or occupied after the earthquake. For critical facilities that need to retain full function immediately post-earthquake to provide emergency response to the community, the building is evaluated to the higher standard of Operational. It should be noted that for structural evaluation, the Operational and Immediate Occupancy criteria are the same. The difference in the two levels is that the support systems and equipment are operational; see Figure 1. Figure 2 includes a brief summary of

![Building performance levels](image-url)
3. SEISMIC ASSESSMENT

Figure 2
Damage Control and Building Performance Labels

<table>
<thead>
<tr>
<th>Overall damage</th>
<th>Structure components</th>
<th>Nonstructural components</th>
<th>Comparison with performance intended for typical buildings designed to codes or standards for new buildings, for the design earthquake</th>
</tr>
</thead>
<tbody>
<tr>
<td>Severe</td>
<td>Severe</td>
<td>Extensive damage. Infills and unbraced parapets failed or at incipient failure.</td>
<td>Significantly more damage and greater life safety risk.</td>
</tr>
<tr>
<td>Moderate</td>
<td>Moderate</td>
<td>Falling hazards, such as parapets, mitigated, but small arches, mechanical, and electrical systems are damaged.</td>
<td>Somewhat more damage and slightly higher life safety risk.</td>
</tr>
<tr>
<td>Light</td>
<td>Light</td>
<td>Equipment and contents are generally secure but might not operate due to mechanical failure or lack of utilities. Some cracking of facades, partitions, and ceilings as well as structural elements. Elevators can be restarted. Fire protection operable.</td>
<td>Less damage and low life safety risk.</td>
</tr>
<tr>
<td>Very light</td>
<td>Very light</td>
<td>Negligible damage occurs. Power and other utilities are available, possibly from standby sources.</td>
<td>Much less damage and very low life safety risk.</td>
</tr>
</tbody>
</table>


each performance level and the anticipated damage for a building designed to each performance level. With the removal of the Police Station and emergency call center from the existing City Hall building, its required Performance Level would fall from Immediate Occupancy to Life Safety.

ASCE 41-13 incorporates a multi-tier methodology for evaluating existing structures. Tier 1, which was chosen for this analysis, is a preliminary screening phase which utilizes a checklist approach to identify potential seismic hazards. It should be noted that at this stage, any identified risks are preliminary and may or may not be justifiable using a higher tier analysis. Tier 2 and Tier 3 are the evaluation and detailed evaluation phases, respectively, which were not conducted at this time. If a deficiency is identified in the Tier 1 screening phase, further Tier 2 or Tier 3 analysis can be used to show the specific item is acceptable. After the seismic evaluation is completed, ASCE 41-13 may be used to complete a seismic retrofit design to address issues identified in the evaluation stage. As a part of the Tier 1 screening phases, various analyses or “Quick Checks” are to be performed where specifically required. Not all items that pass the quick check will necessarily meet more detailed checks nor are they guaranteed to meet current code requirements.

The Tier 1 analysis consists of a visual survey, which was conducted on November 22, 2016. For each of the Tier 1 checklist items, an
3. SEISMIC ASSESSMENT

evaluation of Compliant (C), Non-compliant (NC), Not Applicable (N/A), or Unknown (U) is marked. NC does not necessarily mean that the issue cannot be justified with a higher tier evaluation phase; rather, only that it does not pass the Tier 1 screening criteria.

SCOPE AND LIMITATIONS

This Tier 1 analysis is based on site observations of only readily visible items and evaluation of available drawing documents listed herein. It should be noted that other deficiencies might exist that have not been identified by this screening phase and quick checks. In addition, no material or other testing was performed at this time for review. The Tier 1 quick check calculations have been performed and a more in-depth detailed analysis may be performed, though it is likely to have minimal impact on the results of this evaluation.

EXISTING BUILDING DESCRIPTION

The existing city hall building is located at 380 “A” Avenue, Lake Oswego, Oregon. The original building was built in 1986 and currently houses the Lake Oswego Police Department. The building consists of light framed walls and three cores made of reinforced concrete and masonry. The exterior walls consist of light gauge metal studs clad with Exterior Insulation and Finish System (EIFS). The roof is framed with metal-web wood trusses supported by steel beams and columns. Typical floor framing includes both metal-web wood trusses and wood I-joists supporting wood structural panels and a 1 ½” layer of gypcrete. These joists sit on steel wide flange beams that are connected to HSS columns. The first floor of the city hall is framed with concrete on metal deck supported by wide flange beams and concrete columns as seen in Figure 7. The building also includes a parking structure made of precast concrete planks, concrete walls, and concrete columns as seen in Figures 8 and 9. This first floor and the adjacent heavy parking structure act much like a podium or rigid platform for the rest of the building sitting on it. The primary lateral system of the building consists of two levels of CMU elevator/stair core walls stacked on top of concrete core walls as seen in Figures 10 and 11.

The original building documents prepared by Architect Barrentine Bates Lee and Structural Engineer VanDomelen, Looijenga & Associates, dated August 1985, were available for review along with several other building evaluations done throughout the years. Notably, a seismic assessment was completed by KPFF Consulting Engineers in September of 2006. A geotechnical report was unavailable for review.

The seismic provisions in the current Oregon Structural Specialty Code have changed significantly since the facility was originally designed, and the purpose of this report is to identify potential seismic deficiencies in the structural system.
GLOBAL DEFICIENCIES

The existing City Hall building suffers from several key, global deficiencies in its lateral force resisting system. The building utilizes wood structural panel diaphragms with a gypcrete topping. As the gypcrete is thin and unreinforced, it can only be considered a finish and is not expected to contribute to the rigidity or strength of the floor/roof diaphragms. As a result, the diaphragms must be considered flexible when compared with the relative rigidity of concrete/masonry shear walls/cores.

The only vertical force resisting elements above the first floor are the stair cores located in the center of the building. While at the first floor the concrete on metal deck or concrete on hollow-core plank diaphragms are supported on three or four sides by concrete shear walls, the upper level diaphragms have no such support at the exterior wall. This arrangement of the vertical force resisting elements results in a highly deficient building as the flexible wood diaphragms are not expected to be capable of resisting lateral forces via rotation or torsion of the diaphragms. In fact, this mechanism was explicitly prohibited in the 1982 UBC in force at the time of construction. The worst case scenario would be north-south oriented shaking in which the western edge (and all edges) of the building is laterally unsupported allowing the diaphragm to essentially deflect until failure. While wood diaphragms have been shown to present semi-rigid characteristics in certain arrangements, this building does not have such an arrangement and significant damage could be expected under a code level seismic event.

At the first floor level, there is neither a vertical load resisting element nor a clear shear transfer load path at the transition between the elevated parking deck and the concrete on metal deck floor diaphragm of the building. This lack of continuity or support for these heavy decks may result in out of phase behavior at the first floor or the loss of gravity support for the hollow core planks at the transition. Either behavior would contribute to the poor performance of the building during a seismic event.

LOCAL DEFICIENCIES

There are limited dragstruts present in the building. Even if the wood diaphragms provided an unusually high level of torsional resistance, the dragstruts in the building are inadequate to deliver the resulting seismic forces to the building’s cores. In addition, the building lacks any clearly defined chords at the perimeter of the diaphragms. The lack of such chords combined with large expected deflections could result in significant failure of the floor and roof diaphragms in a seismic event.

Additionally, the lack of continuous cross ties between steel girders or between wood trusses is another deficiency of the building that may contribute to diaphragm failure in a seismic event. While not
as critical as the absence of dragstruts or chords (due to alternate and secondary force paths), this deficiency may magnify the effects of other building deficiencies as ties are a significant contributor to diaphragm integrity.

### 3. SEISMIC ASSESSMENT

#### MAIN BUILDING EVALUATION

**Evaluation Criteria**

This building was evaluated for a seismic event with a probability of exceedance of 10% in 50 years or a 500-year event (BSE-1) for a Performance Level of Life Safety. This is the same design earthquake ground motion hazard to which new buildings are designed. The level of seismicity was determined at the site and compared to the ASCE 41-13 level definitions; see Figure 12. For this building, $S_D=0.659$ and $S_D=0.387$; therefore, the site is in an area of high seismicity.

Based on this seismicity definition and a Life Safety performance objective, the required checklists can be determined, as seen in Figure 13. The Basic Configuration, Life Safety Structural Checklists, and Nonstructural Checklists are required. The Nonstructural Checklist will not be covered in this evaluation, and only structural elements will be discussed in the report.

ASCE 41-13 has different checklists depending on the building construction type. This building type is classified as RM1, Reinforced Masonry Bearing Walls, and C2A, Concrete Shear Walls.

Figure 11
CMU Walls Stacked On Concrete

Figure 12
Level of Seismicity Definitions

<table>
<thead>
<tr>
<th>Level of Seismicity</th>
<th>$S_{DL}$</th>
<th>$S_{DN}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very low</td>
<td>&gt;0.067 g</td>
<td>&gt;0.067 g</td>
</tr>
<tr>
<td>Low</td>
<td>≥0.167 g</td>
<td>&gt;0.067 g</td>
</tr>
<tr>
<td>Moderate</td>
<td>≥0.33 g</td>
<td>&gt;0.133 g</td>
</tr>
<tr>
<td>High</td>
<td>≥0.50 g</td>
<td>&gt;0.20 g</td>
</tr>
</tbody>
</table>

*The higher level of seismicity defined by $S_{DL}$ or $S_{DN}$ shall govern.*

**Source:** Table 2-5, page 49; ASCE Standard – ASCE/SEI 41-13: American Society of Civil Engineers – Seismic Evaluation and Retrofit of Existing Buildings

**Figure 17**
Checklists Required for a Tier 1 Screening

Table 4-7. Checklists Required for a Tier 1 Screening

<table>
<thead>
<tr>
<th>Level of Seismicity</th>
<th>Level of Building Performance</th>
<th>Very Low Seismicity Checklist (Sec. 16.1.1)</th>
<th>Basic Configuration Checklist (Sec. 16.1.2)</th>
<th>Life Safety Checklist (Sec. 16.2LS through 16.15LS)</th>
<th>Immediate Occupancy Checklist (Sec. 16.2O through 16.15O)</th>
<th>Life Safety Nonstructural Checklist (Sec. 16.17)</th>
<th>Position Retention Nonstructural Checklist (Sec. 16.17)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very low</td>
<td>LS</td>
<td>X</td>
<td></td>
<td></td>
<td></td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Very low</td>
<td>IO</td>
<td>X</td>
<td>X</td>
<td></td>
<td></td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Low</td>
<td>LS</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Low</td>
<td>IO</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Moderate</td>
<td>LS</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Moderate</td>
<td>IO</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>High</td>
<td>LS</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
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<td>X</td>
</tr>
<tr>
<td>High</td>
<td>IO</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
</tbody>
</table>

*An X designates the checklist that must be completed for a Tier 1 screening as a function of the level of seismicity and level of performance.

*Defined in Section 2.5.

**Source:** Table 4-7, page 67; ASCE Standard – ASCE/SEI 41-13: American Society of Civil Engineers – Seismic Evaluation and Retrofit of Existing Buildings
3. SEISMIC ASSESSMENT

Summary of ASCE 41-13 Tier 1 Evaluation
The Tier 1 screening phase identified numerous structural items as non-compliant. Non-compliant issues require further evaluation in order to determine their full impact on the seismic performance of the building, but these issues are a relatively good indicator of potential performance issues. A summary of some non-compliant issues is presented below organized by each checklist. Copies of the Tier 1 checklists and calculations are included in this report in Appendices A and B.

Life Safety Basic Configuration Checklist

- **Load Path** – A clear lateral load path to transfer seismic forces from the walls, into the roof and floor diaphragms, into the main lateral force resisting system, and then out into the foundations is required for compliance. The lateral force resisting system appears to be deficient. There is a lack of chords and/or dragstruts at the exterior of the building to resist moments caused by seismic forces. This may result in diaphragm failure during a seismic event. With the lateral force resisting system located at the stair and elevator core at the center of the building, there are large cantilevers of the flexible wood diaphragms of up to 65 feet. These were not been permitted even in the UBC 1982 that was used in the design of this building. These large cantilevers will cause extreme torsion at the diaphragm level during a seismic event which the wood diaphragms are not capable of sustaining. At the attached parking garage on the south side, there is no clear path to transfer shear from the hollow core planks to the concrete deck which will cause the parking garage to act independently of the rest of the city hall during an earthquake.

- **Mass** – There cannot be a change in effective mass of more than 50% from one story to the next. The mass of the parking garage at the first floor represents a mass greater than 50% of the second story. Because the first floor essentially functions as a podium and is surrounded on at least two sides by soil, this mass will not be considered a detriment to the seismic performance of the building.

- **Torsion** – For the building to comply for torsion, the center of mass and the center of rigidity at each floor cannot be more than 20% of the width of the building. Because of the cantilevered flexible diaphragm at all sides of the building, localized torsion will be experienced during a seismic event.

- **Liquefaction** – Liquefaction-susceptible soils cannot exist in the foundation soils at depths within 50ft under the building. In the absence of a site-specific geotechnical report, the Oregon Statewide Geohazards map prepared by DOGAMI is often used which indicates the station sits in an area with high liquefaction hazard. However, preliminary information from GeoDesign (preparing a site-specific geotechnical report for the future police station on the same site) indicates that liquefaction is not a hazard.

- **Ties Between Foundation Elements** – The foundation needs to have ties to resist seismic forces where footings are not restrained by
beams, slabs, or stiff soil. Since there are no ties from the footings to the slab or between spread footings, this building is not in compliance. In addition, since there is no geotechnical report, the soils on the site cannot be confirmed to be Class D.

**Life Safety Structural Checklist for Building Types C2A and RM1**

- **Complete Frames** – Where concrete walls provide both gravity support and lateral resistance, a concrete frame within the assembly must be specifically designed to provide vertical support in the event the wall is damaged by lateral forces. The concrete walls in this building may become damaged during a seismic event and must still be able to support gravity loads. The concrete cores in the City Hall building likely have not been adequately designed to carry both vertical and seismic loads.

- **Reinforcing Steel** – The spacing of reinforcing steel in reinforced masonry shear walls needs to be less than 48". The horizontal reinforcement in the reinforced masonry shear walls are spaced at 48”.

- **Shear Stress Check** – The ASCE 41 provides quick checks for the maximum shear stress in a shear wall. For a Tier 1 analysis, both the concrete and reinforced masonry shear walls failed the quick checks. Shear stress in the concrete walls was calculated to be greater than $2\sqrt{(f' c)}$. Shear stress in the reinforced masonry wall was calculated to be greater than 70lb/in².

- **Wood Ledgers** – The connections of wood ledgers to the concrete or masonry wall panels are not permitted to induce cross-grain bending or tension perpendicular to the grain in a wood member. This can cause non-ductile failure of the ledger. The wood ledgers at the shear wall cores induce cross-grain bending in the ledgers. See Figure 14.

- **Transfer to Shear Walls** – Diaphragms need to be connected to shear walls to transfer seismic forces, and the connections need to be strong enough to carry the load. The wood drag struts located at the corners of the shear wall cores were not found to have sufficient capacity to transfer seismic loads from the diaphragm to the shear walls. See Figure 15.

- **Coupling Beams** – Coupling beams connect two separate wall panels across a means of egress. They drag force from one wall to another, so they do not act independently during a seismic event. Shear stirrups are required in coupling beams above a means of egress. Since the plans do not show the necessary stirrups above means of egress, the concrete segments above means of egress are determined to be deficient.

- **Confinement Reinforcement** – All boundary elements of shear walls need to be confined with spirals or ties. The confinement reinforcement is insufficiently spaced at distances greater than 8db (3” for #3 ties). The plans call out for 6 ties spaced at 4” top and bottom and ties at 8” to fill the rest of the wall weight.
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- **Openings at Shear Walls** – Large openings in the diaphragms at shear walls limit the capability to transfer seismic forces from the diaphragms to the shear walls. At the north shear wall core, there is both a mechanical shaft and elevator shaft directly adjacent to the shear wall. These openings occur throughout the whole length of the shear wall; therefore, this check item is noncompliant.

- **Plan Irregularities** – Diaphragms need to have tensile capacity to resist seismic forces at reentrant corners or other plan irregularities. The Lake Oswego City Hall has many instances of reentrant corners that do not have any sort of strapping or ties to transfer tension forces across the diaphragm. This will cause shear failures at the corners during a seismic event.

- **Cross Ties** – Cross ties are required to make diaphragm chords act together during a seismic event. In the east to west direction, there are steel beams that tie one end of the building to the other, but there are no diaphragm chords to capture forces. It is unknown whether the connections between the steel beams can transfer seismic forces from one side of the building to the other. In the north to south direction, there is no strapping detailed where the I-joists attach to the steel beams.

- **Diagonally Sheathed and Unblocked Diaphragms** – Diagonally sheathed and unblocked diaphragms cannot span more than 30 feet or have aspect ratios greater than 3 to 1. Since the diaphragms in the City Hall building are unblocked and span distances much greater than 30 feet, the diaphragms are noncompliant.

- **Stiffness of Wall Anchors** – Anchors that connect wood structural elements to structural walls need to be installed taut or stiff, so there is very little relative movement between the wall and the diaphragm. It is unknown whether the anchors throughout the building have been installed as such.

**Additional Concerns from Site Visit**

- EIFS is clearly failing throughout the exterior of the building. There is visible water damage in some areas. The staff commented that mold and moisture seepage were prevalent in the exterior walls. This water damage is beyond the scope of this report.

- Many wide flange beams in the basement level were rusted and corroded. See Figures 16 and 17. This can cause the connections to fail as they further deteriorate.

- Some concrete beams at the basement level have experienced cracking. See Figure 18.

- The shear wall cores have some cracks forming. See Figure 19. It is unclear if these cracks are superficial surface level cracks caused by the installation of stair anchors, or if they are the result of building movement. Building staff members indicated that these cracks have been present since the building opened.
4. RECOMMENDATIONS
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4. RECOMMENDATIONS

RETROFIT RECOMMENDATIONS

Prior to completing a full or partial seismic retrofit design, material testing of key structural elements would be recommended. While not explicitly required by ASCE 41-13 for a building with a Life Safety performance level, the benefits of such testing typically outweigh the costs.

The Tier 1 structural deficiencies listed above will require further evaluation (ASCE 41-13 Tier 2 or 3 analyses) for the design of the seismic retrofits listed below. For Lake Oswego City Hall to meet the Life Safety Performance Level, each of these items will need to be further evaluated and brought up to current code requirements. The following narrative describes the approximate scope of one possible upgrade scheme to address the identified deficiencies. While the scope of the retrofits is listed here, and plans showing the extents of the retrofits are provided in Appendix D, the associated details and narrative are addressed in more detail in a separate cost estimating package.

STRUCTURAL RETROITS

1. Addition of a braced frame along gridline A (east wall of the building) continuing to a thickened shear wall below and new/expanded foundations at the parking level.
2. Addition of braced frames along gridline E (west wall of the building) continuing down to new foundations at the street level.
3. Addition of braced frames along gridlines 1 and 9 (north and south walls of the building) continuing to thickened retaining wall or braces and new/expanded foundations at the parking level.
4. Provide a new braced frame and foundations at the first level along gridline 9 to support the elevated parking deck.
5. Strengthening of the existing stair and elevator cores with the addition of shotcrete or FRP and anchors to the foundations.
6. Provision of new dragstruts in line with core walls along gridlines 3 and 7 and potential strengthening of those along gridlines D and F. These new steel dragstruts would replace existing wood dragstruts or include strengthening existing steel girders as required.
7. Provide positive connections at girder transitions capable of transferring seismic forces (as at grid intersections H2, H3, H5, H7, K2, K3, K7, 1C, 1G, 9E, 9I, etc.) at the roof and floor levels 2 and 3.
8. Provide continuous cross ties in the north-south direction between wood trusses across girders. This would require demolition and replacement of portions of the existing gypcrete and installation of straps at ~4'-0” O.C.
4. RECOMMENDATIONS

9. Strengthening of shearwall segments supporting the elevated parking deck with the addition of shotcrete

CONCLUSIONS

The ASCE 41 Tier 1 analysis and assessment of the building demonstrates that it has several significant seismic deficiencies which can be expected to result in very poor seismic performance. While it was found that the veneer weight of the building could be increased slightly without altering the seismic mass of the building so much that a full seismic upgrade would be required explicitly by code, the deficiencies of this building are such that even a small change in the mass of the building is likely to violate the spirit and intent of the building code provisions intended to allow modification of a building. In order to justify increasing the mass of the building with a veneer change, the building would likely need to be brought into compliance with the current building code. Compliance with the current building code would require a full seismic retrofit of both the local and global seismic deficiencies of the building's lateral force resisting system. If such a retrofit were completed, the building would certainly improve its structural capabilities and performance and could be brought up to a Life Safety performance level.

In order to design such a retrofit, a complete Tier 3 analysis of the building and seismic force resisting system would need to be conducted to fully understand all the issues that would require repair or retrofit to bring the building into conformance with Life Safety performance level requirements. The complete analysis and design development for those repairs is an effort that is beyond the scope of this investigation. Depending on the results of this additional analysis/investigation, there may be changes to the list of repairs above.
A. VENEER ANALYSIS
Load Takeoffs

**Roof Load Takeoff**

<table>
<thead>
<tr>
<th>Dead Loads (Roof)</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Roofing (psf)</td>
<td>1</td>
</tr>
<tr>
<td>Sheathing (psf)</td>
<td>2.4</td>
</tr>
<tr>
<td>Ceiling (psf)</td>
<td>3</td>
</tr>
<tr>
<td>Lights (psf)</td>
<td>1</td>
</tr>
<tr>
<td>Mechanical (psf)</td>
<td>2</td>
</tr>
<tr>
<td>Miscellaneous (psf)</td>
<td>1.5</td>
</tr>
<tr>
<td>Misc. Wood (psf)</td>
<td>1.5</td>
</tr>
<tr>
<td>Steel Beams (psf)</td>
<td>1.8</td>
</tr>
<tr>
<td>Joists (psf)</td>
<td>1.6</td>
</tr>
<tr>
<td>Concrete (psf)</td>
<td>0.9</td>
</tr>
<tr>
<td><strong>Total PSF:</strong></td>
<td>16.7</td>
</tr>
<tr>
<td><strong>Area (sf):</strong></td>
<td>11600</td>
</tr>
<tr>
<td><strong>Weight (k):</strong></td>
<td>194</td>
</tr>
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</table>

**Walls Loads (Roof):**

<table>
<thead>
<tr>
<th>Shearwall Core (3):</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>8” CMU Grout @ Reinf. (psf)</td>
<td>51</td>
</tr>
<tr>
<td>Tributary wall height (ft)</td>
<td>6</td>
</tr>
<tr>
<td>Length of wall (ft)</td>
<td>52</td>
</tr>
<tr>
<td><strong>Weight (k):</strong></td>
<td>47.74</td>
</tr>
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</table>

**Exterior:**

<table>
<thead>
<tr>
<th>6” Steel studs 43 mil (psf)</th>
<th>1.83</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gypsum (psf)</td>
<td>4</td>
</tr>
<tr>
<td>EIFS (psf)</td>
<td>1.5</td>
</tr>
<tr>
<td>Insulation (psf)</td>
<td>1</td>
</tr>
<tr>
<td><strong>WALL WEIGHT (psf):</strong></td>
<td>8.33</td>
</tr>
<tr>
<td>Tributary wall height (ft)</td>
<td>6</td>
</tr>
<tr>
<td>Length of wall (ft)</td>
<td>481</td>
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<td><strong>Weight (k):</strong></td>
<td>24.04</td>
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**3rd Floor Load Takeoff**

<table>
<thead>
<tr>
<th>Dead Loads (Third)</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>1 1/2” Gypcrete (psf)</td>
<td>13</td>
</tr>
<tr>
<td>Sheathing (psf)</td>
<td>2.4</td>
</tr>
<tr>
<td>Ceiling (psf)</td>
<td>3</td>
</tr>
<tr>
<td>Lights (psf)</td>
<td>1</td>
</tr>
<tr>
<td>Miscellaneous (psf)</td>
<td>1.5</td>
</tr>
<tr>
<td>Misc. Wood (psf)</td>
<td>1.5</td>
</tr>
<tr>
<td>Steel Beams (psf)</td>
<td>3.4</td>
</tr>
<tr>
<td>Joists (psf)</td>
<td>3.2</td>
</tr>
<tr>
<td><strong>Total PSF:</strong></td>
<td>29.0</td>
</tr>
<tr>
<td><strong>Area (sf):</strong></td>
<td>13400</td>
</tr>
<tr>
<td><strong>Weight (k):</strong></td>
<td>388.6</td>
</tr>
</tbody>
</table>

**Walls Loads (Third):**

<table>
<thead>
<tr>
<th>Shearwall Core (3):</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>8” CMU Grout @ Reinf. (psf)</td>
<td>51</td>
</tr>
<tr>
<td>Tributary wall height (ft)</td>
<td>12</td>
</tr>
<tr>
<td>Length of wall (ft)</td>
<td>52</td>
</tr>
<tr>
<td><strong>Weight (k):</strong></td>
<td>95.47</td>
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**Exterior:**

<table>
<thead>
<tr>
<th>6” Steel studs 43 mil (psf)</th>
<th>1.83</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gypsum (psf)</td>
<td>4</td>
</tr>
<tr>
<td>EIFS (psf)</td>
<td>1.5</td>
</tr>
<tr>
<td>Insulation (psf)</td>
<td>1</td>
</tr>
<tr>
<td><strong>WALL WEIGHT (psf):</strong></td>
<td>8.33</td>
</tr>
<tr>
<td>Tributary wall height (ft)</td>
<td>12</td>
</tr>
<tr>
<td>Length of wall (ft)</td>
<td>504</td>
</tr>
<tr>
<td><strong>Weight (k):</strong></td>
<td>50.38</td>
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</table>
## Load Takeoffs

### 2nd Floor Load Takeoff

<table>
<thead>
<tr>
<th>DEAD LOADS (SECOND)</th>
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</thead>
<tbody>
<tr>
<td>1 1/2&quot; Gypcrete (psf)</td>
<td>13</td>
</tr>
<tr>
<td>Sheathing (psf)</td>
<td>2.4</td>
</tr>
<tr>
<td>Ceiling (psf)</td>
<td>3</td>
</tr>
<tr>
<td>Lights (psf)</td>
<td>1</td>
</tr>
<tr>
<td>Miscellaneous (psf)</td>
<td>1.5</td>
</tr>
<tr>
<td>Misc. Wood (psf)</td>
<td>1.5</td>
</tr>
<tr>
<td>Steel Beams (psf)</td>
<td>3.4</td>
</tr>
<tr>
<td>Joists (psf)</td>
<td>3.2</td>
</tr>
<tr>
<td><strong>TOTAL PSF:</strong></td>
<td><strong>29.0</strong></td>
</tr>
<tr>
<td>Area (sf)</td>
<td>13400</td>
</tr>
<tr>
<td><strong>WEIGHT (k):</strong></td>
<td><strong>388.6</strong></td>
</tr>
</tbody>
</table>

### WALL LOADS (SECOND)

**Shearwall Core (3):**

| 8" CMU Grout @ Reinf. (psf)         | 51 |
| Tributary wall height (ft)          | 6  |
| Length of wall (ft)                 | 52 |
| 8" Concrete (psf)                   | 100|
| Tributary wall height (ft)          | 7.5|
| Length of wall (ft)                 | 52 |
| **WEIGHT (k):**                     | **164.74**|

**Exterior:**

- 6" Steel studs 43 mil (psf)          | 1.83|
- Gypsum (psf)                        | 4 |
- EIFS (psf)                          | 1.5|
- Insulation (psf)                    | 1 |
| **WALL WEIGHT (psf):**               | **8.33**|
| Tributary wall height (ft)           | 13.5|
| Length of wall (ft)                  | 504|
| **WEIGHT (k):**                      | **57**|
# Load Takeoffs

## 1st Floor Load Takeoff

<table>
<thead>
<tr>
<th>Component</th>
<th>PSF</th>
<th>Weight (k)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>DEAD LOADS (FIRST)</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Floor and decking (psf)</td>
<td>38.2</td>
<td></td>
</tr>
<tr>
<td>Ceiling (psf)</td>
<td>3</td>
<td></td>
</tr>
<tr>
<td>Lights (psf)</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>Miscellaneous (psf)</td>
<td>1.5</td>
<td></td>
</tr>
<tr>
<td>Misc. Wood (psf)</td>
<td>1.5</td>
<td></td>
</tr>
<tr>
<td>Steel Beams (psf)</td>
<td>7.1</td>
<td></td>
</tr>
<tr>
<td><strong>TOTAL PSF:</strong></td>
<td>52.3</td>
<td>622.37</td>
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<tr>
<td>Area (sf)</td>
<td>11900</td>
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<table>
<thead>
<tr>
<th>Component</th>
<th>PSF</th>
<th>Weight (k)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>PARKING LOT (FIRST)</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>10&quot; Hollowcore (psf)</td>
<td>67</td>
<td>1095</td>
</tr>
<tr>
<td>Flooring (psf)</td>
<td>31</td>
<td></td>
</tr>
<tr>
<td>Miscellaneous (psf)</td>
<td>1.5</td>
<td></td>
</tr>
<tr>
<td>Concrete Beams (psf)</td>
<td>10</td>
<td></td>
</tr>
<tr>
<td><strong>TOTAL PSF:</strong></td>
<td>109.5</td>
<td></td>
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<tr>
<td>Area (sf)</td>
<td>10000</td>
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<table>
<thead>
<tr>
<th>Component</th>
<th>PSF</th>
<th>Weight (k)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>WALL LOADS (FIRST)</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shearwall Core (3):</td>
<td></td>
<td></td>
</tr>
<tr>
<td>8&quot; Concrete (psf)</td>
<td>100</td>
<td>234</td>
</tr>
<tr>
<td>Tributary wall height (ft)</td>
<td>15</td>
<td></td>
</tr>
<tr>
<td>Length of wall (ft)</td>
<td>52</td>
<td></td>
</tr>
<tr>
<td><strong>WEIGHT (k):</strong></td>
<td>234</td>
<td></td>
</tr>
<tr>
<td>Basement Walls:</td>
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<td></td>
</tr>
<tr>
<td>8&quot; Concrete (psf)</td>
<td>100</td>
<td>240</td>
</tr>
<tr>
<td>Tributary wall height (ft)</td>
<td>7.5</td>
<td></td>
</tr>
<tr>
<td>Length of wall (ft)</td>
<td>320</td>
<td></td>
</tr>
<tr>
<td><strong>WEIGHT (k):</strong></td>
<td>240</td>
<td></td>
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</table>

<table>
<thead>
<tr>
<th>Component</th>
<th>PSF</th>
<th>Weight (k)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>PARKING LOT WALLS (FIRST)</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>8&quot; CMU Full Grout (psf)</td>
<td>81</td>
<td>127.82</td>
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<tr>
<td>Tributary wall height (ft)</td>
<td>6</td>
<td></td>
</tr>
<tr>
<td>Length of wall (ft)</td>
<td>263</td>
<td></td>
</tr>
<tr>
<td><strong>WEIGHT (k):</strong></td>
<td>127.82</td>
<td></td>
</tr>
<tr>
<td>8&quot; Concrete (psf)</td>
<td>100</td>
<td>301.5</td>
</tr>
<tr>
<td>Tributary wall height (ft)</td>
<td>7.5</td>
<td></td>
</tr>
<tr>
<td>Length of wall (ft)</td>
<td>402</td>
<td></td>
</tr>
<tr>
<td><strong>WEIGHT (k):</strong></td>
<td>301.5</td>
<td></td>
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</table>

<table>
<thead>
<tr>
<th>Component</th>
<th>PSF</th>
<th>Weight (k)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Exterior:</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6&quot; Steel studs 43 mil (psf)</td>
<td>1.83</td>
<td>8.33</td>
</tr>
<tr>
<td>Gypsum (psf)</td>
<td>4</td>
<td></td>
</tr>
<tr>
<td>EIFS (psf)</td>
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<td></td>
</tr>
<tr>
<td>Insulation (psf)</td>
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<td></td>
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<tr>
<td><strong>WALL WEIGHT (psf):</strong></td>
<td>8.33</td>
<td>27.80</td>
</tr>
<tr>
<td>Tributary wall height (ft)</td>
<td>7.5</td>
<td></td>
</tr>
<tr>
<td>Length of wall (ft)</td>
<td>445</td>
<td></td>
</tr>
<tr>
<td><strong>WEIGHT (k):</strong></td>
<td>27.80</td>
<td></td>
</tr>
</tbody>
</table>
### A. VENEER ANALYSIS

#### Distribution of Seismic Forces

**With Podium Included**

#### BASE SHEAR

- Importance factor $I = 1.0$
- Redundancy factor $\rho = 1.3$
- Ordinary Reinforced Masonry Shearwall

**$\rho I/R = 0.650$**

**$S_{ds} = 0.659$ g**

**Cs = 0.330**

**Total building weight $W = 4058$ k**

**Equivalent Lateral Force base shear $(Cs*W) = 1337$ k**

#### Vertical Distribution of Seismic Forces

<table>
<thead>
<tr>
<th>Level</th>
<th>SDL (psf)</th>
<th>Area (ft²)</th>
<th>ht (ft)</th>
<th>$w_{x,fr}$ (k)</th>
<th>$w_{x,wall}$ (k)</th>
<th>$w_{x,total}$ (k)</th>
<th>$w_{x}*ht$ (k-ft)</th>
<th>$C_{vx}$ (%)</th>
<th>$F_x$ (k)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Upper 3rd/Roof</td>
<td>16.7</td>
<td>11600</td>
<td>54</td>
<td>194</td>
<td>72</td>
<td>265</td>
<td>14337</td>
<td>15.1%</td>
<td>202</td>
</tr>
<tr>
<td>3rd Floor</td>
<td>29</td>
<td>13400</td>
<td>42</td>
<td>389</td>
<td>146</td>
<td>534</td>
<td>22447</td>
<td>23.7%</td>
<td>317</td>
</tr>
<tr>
<td>2nd Floor</td>
<td>29</td>
<td>13400</td>
<td>30</td>
<td>389</td>
<td>221</td>
<td>610</td>
<td>18300</td>
<td>19.3%</td>
<td>258</td>
</tr>
<tr>
<td>1st Floor</td>
<td>-</td>
<td>-</td>
<td>15</td>
<td>1717</td>
<td>931</td>
<td>2648</td>
<td>39727</td>
<td>41.9%</td>
<td>560</td>
</tr>
<tr>
<td><strong>TOTAL</strong></td>
<td>38400</td>
<td>2688</td>
<td>1370</td>
<td>4058</td>
<td>94812</td>
<td>100%</td>
<td>1337</td>
<td></td>
<td></td>
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</tbody>
</table>

#### Diaphragm Design (Inertial) Forces (omit $\rho$)

<table>
<thead>
<tr>
<th>Level</th>
<th>$w_{x,total}$ (k)</th>
<th>$\Sigma w_i$ (k)</th>
<th>$F_x$ (k)</th>
<th>$\Sigma F_i$ (k)</th>
<th>$F_{x,min}$ (k)</th>
<th>$F_{x,max}$ (k)</th>
<th>$F_{p_{xx}}$ (k)</th>
<th>$F_{x,\text{design}}$ (k)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Upper 3rd/Roof</td>
<td>265</td>
<td>265</td>
<td>202</td>
<td>202</td>
<td>35</td>
<td>69</td>
<td>202</td>
<td>202</td>
</tr>
<tr>
<td>3rd Floor</td>
<td>534</td>
<td>800</td>
<td>317</td>
<td>519</td>
<td>69</td>
<td>139</td>
<td>347</td>
<td>317</td>
</tr>
<tr>
<td>2nd Floor</td>
<td>610</td>
<td>1410</td>
<td>258</td>
<td>777</td>
<td>79</td>
<td>159</td>
<td>336</td>
<td>258</td>
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<tr>
<td>1st Floor</td>
<td>2648</td>
<td>4058</td>
<td>560</td>
<td>1337</td>
<td>344</td>
<td>689</td>
<td>873</td>
<td>689</td>
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<td><strong>TOTAL</strong></td>
<td>4058</td>
<td>1337</td>
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<td></td>
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</tbody>
</table>

Lake Oswego City Hall

January 27, 2017

A-4
**NEW WALL LOADS (Total Wall Weight):**

Upper 3rd/Roof:
- Brick Veneer: 17 psf

3rd Floor
- Brick Veneer: 17 psf

2nd Floor
- Brick Veneer: 17 psf

1st Floor
- Brick Veneer: 17 psf

<table>
<thead>
<tr>
<th>Level</th>
<th>$W_{x\text{wall\ new}}$ (k)</th>
<th>$W_{x\text{total\ new}}$ (k)</th>
<th>$W_{x\text{new*ht}}$ (k-ft)</th>
<th>$C_{vx\ new}$ (%)</th>
<th>$F_{x\ new}$ (k)</th>
<th>Weight Change (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Upper 3rd/Roof</td>
<td>97</td>
<td>291</td>
<td>15688</td>
<td>15.6%</td>
<td>217</td>
<td>9.4%</td>
</tr>
<tr>
<td>3rd Floor</td>
<td>198</td>
<td>587</td>
<td>24649</td>
<td>24.5%</td>
<td>341</td>
<td>9.8%</td>
</tr>
<tr>
<td>2nd Floor</td>
<td>280</td>
<td>669</td>
<td>20070</td>
<td>20.0%</td>
<td>278</td>
<td>9.7%</td>
</tr>
<tr>
<td>1st Floor</td>
<td>960</td>
<td>2677</td>
<td>40161</td>
<td>39.9%</td>
<td>556</td>
<td>1.1%</td>
</tr>
<tr>
<td>TOTAL</td>
<td>1536</td>
<td>4224</td>
<td>100569</td>
<td>100%</td>
<td>1392</td>
<td>4.1%</td>
</tr>
</tbody>
</table>

**Analysis**

- New Total Building Weight: $V_{\text{new}} = 1392$ k
- Force base shear ($C_s \times W$): $F_{\text{design}} = (12.8 - 2)$

<table>
<thead>
<tr>
<th>Level</th>
<th>$F_{x\ new*ht}$ (k-ft)</th>
<th>$C_{vx\ new}$ (%)</th>
<th>$F_{x\ new}$ (k)</th>
<th>$F_{x\ new\ max}$ (k)</th>
<th>$F_{px\ new}$ (k)</th>
<th>$F_{x\ design}$ (k)</th>
<th>Change (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Upper 3rd/Roof</td>
<td>217</td>
<td>7.4%</td>
<td>217</td>
<td>76</td>
<td>1392</td>
<td>217</td>
<td>7.4%</td>
</tr>
<tr>
<td>3rd Floor</td>
<td>341</td>
<td>7.7%</td>
<td>341</td>
<td>153</td>
<td>362</td>
<td>341</td>
<td>7.7%</td>
</tr>
<tr>
<td>2nd Floor</td>
<td>278</td>
<td>7.6%</td>
<td>278</td>
<td>174</td>
<td>882</td>
<td>278</td>
<td>7.6%</td>
</tr>
<tr>
<td>1st Floor</td>
<td>696</td>
<td>1.1%</td>
<td>696</td>
<td>696</td>
<td></td>
<td>696</td>
<td>1.1%</td>
</tr>
<tr>
<td>TOTAL</td>
<td>1392</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Seismic Assessment
2160377.00
A-5
### Distribution of Seismic Forces

**With Podium Excluded**

**BASE SHEAR**

\[(12.8.1)\]

<table>
<thead>
<tr>
<th>Importance factor</th>
<th>Redundancy factor</th>
<th>Ordinary Reinforced Masonry Shearwall</th>
</tr>
</thead>
<tbody>
<tr>
<td>( I )</td>
<td>( \rho )</td>
<td>( R )</td>
</tr>
<tr>
<td>1.0</td>
<td>1.3</td>
<td>2.0</td>
</tr>
</tbody>
</table>

\[\rho \times I/R = 0.650\]

\[S_{ds} = 0.659 \text{ g}\]

\[C_{s} = 0.330\]

**REDUNDANCY FACTOR NOT INCLUDED!!!**

**W** Total building weight

\[V, \text{ ew} = 465 \text{ k} \]

**Equivalent Lateral Force base shear**

\[Cs \times W = 1410 \text{ k} \]

### Vertical Distribution of Seismic Forces

\[(12.8.3)\]

<table>
<thead>
<tr>
<th>Level</th>
<th>SDL (psf)</th>
<th>Area (ft²)</th>
<th>ht (ft)</th>
<th>( w_{x, \text{ thr}} ) (k)</th>
<th>( w_{x, \text{ wall}} ) (k)</th>
<th>( w_{x, \text{ total}} ) (k)</th>
<th>( w_{x, \text{ total}} \times \text{ht} ) (k-ft)</th>
<th>( C_{sx} ) (%)</th>
<th>( F_{x} ) (k)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Upper 3rd/Roof</td>
<td>16.7</td>
<td>11600</td>
<td>39</td>
<td>194</td>
<td>72</td>
<td>265</td>
<td>10354</td>
<td>30.5%</td>
<td>142</td>
</tr>
<tr>
<td>3rd Floor</td>
<td>29</td>
<td>13400</td>
<td>27</td>
<td>389</td>
<td>146</td>
<td>534</td>
<td>14430</td>
<td>42.5%</td>
<td>198</td>
</tr>
<tr>
<td>2nd Floor</td>
<td>29</td>
<td>13400</td>
<td>15</td>
<td>389</td>
<td>221</td>
<td>610</td>
<td>9150</td>
<td>27.0%</td>
<td>125</td>
</tr>
<tr>
<td><strong>TOTAL</strong></td>
<td>38400</td>
<td>971</td>
<td>439</td>
<td>1410</td>
<td>33935</td>
<td>100%</td>
<td>465</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Diaphragm Design (Inertial) Forces (omit \( \rho \))

\[(12.10.1)\]

<table>
<thead>
<tr>
<th>Level</th>
<th>( w_{x, \text{ total}} ) (k)</th>
<th>( \Sigma w_{i} ) (k)</th>
<th>( F_{s} ) (k)</th>
<th>( \Sigma F_{i} ) (k)</th>
<th>( F_{x, \text{ min}} ) (k)</th>
<th>( F_{x, \text{ max}} ) (k)</th>
<th>( F_{px} ) (k)</th>
<th>( F_{x, \text{ design}} ) (k)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Upper 3rd/Roof</td>
<td>265</td>
<td>265</td>
<td>142</td>
<td>142</td>
<td>35</td>
<td>69</td>
<td>142</td>
<td>142</td>
</tr>
<tr>
<td>3rd Floor</td>
<td>534</td>
<td>800</td>
<td>198</td>
<td>339</td>
<td>69</td>
<td>139</td>
<td>227</td>
<td>198</td>
</tr>
<tr>
<td>2nd Floor</td>
<td>610</td>
<td>1410</td>
<td>125</td>
<td>465</td>
<td>79</td>
<td>159</td>
<td>201</td>
<td>159</td>
</tr>
<tr>
<td><strong>TOTAL</strong></td>
<td>1410</td>
<td>465</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
# NEW WALL LOADS (Total Wall Weight):

**Upper 3rd/ Roof:**
- Brick Veneer: 17 psf

**3rd Floor**
- Brick Veneer: 17 psf

**2nd Floor**
- Brick Veneer: 17 psf

**1st Floor**
- Brick Veneer: 17 psf

<table>
<thead>
<tr>
<th>Level</th>
<th>$W_{new}$ (k)</th>
<th>$V_{new}$ (k)</th>
<th>New Total Building Weight</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1546</td>
<td>510</td>
<td></td>
</tr>
</tbody>
</table>

**Force base shear ($C_s\times W$) (12.8-2)**

<table>
<thead>
<tr>
<th>Level</th>
<th>$W_{total\ new}$ (k)</th>
<th>$\Sigma w_i$ (k)</th>
<th>$F_{x\ new}$ (k)</th>
<th>$\Sigma F_i$ (k)</th>
<th>$F_{x\ min\ new}$ (k)</th>
<th>$F_{x\ max\ new}$ (k)</th>
<th>$F_{px\ new}$ (k)</th>
<th>$F_{x\ design}$ (k)</th>
<th>Change (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Upper 3rd/ Roof</td>
<td>291</td>
<td>291</td>
<td>155</td>
<td>155</td>
<td>38</td>
<td>76</td>
<td>155</td>
<td>155</td>
<td>9.4%</td>
</tr>
<tr>
<td>3rd Floor</td>
<td>587</td>
<td>877</td>
<td>217</td>
<td>372</td>
<td>153</td>
<td>249</td>
<td>174</td>
<td>217</td>
<td>9.8%</td>
</tr>
<tr>
<td>2nd Floor</td>
<td>669</td>
<td>1546</td>
<td>137</td>
<td>510</td>
<td>87</td>
<td>174</td>
<td>174</td>
<td>174</td>
<td>9.7%</td>
</tr>
</tbody>
</table>

**TOTAL**

<table>
<thead>
<tr>
<th>$W_{total\ new}$ (k)</th>
<th>$\Sigma w_i$ (k)</th>
<th>$F_{x\ new}$ (k)</th>
<th>$\Sigma F_i$ (k)</th>
<th>$F_{x\ min\ new}$ (k)</th>
<th>$F_{x\ max\ new}$ (k)</th>
<th>$F_{px\ new}$ (k)</th>
<th>$F_{x\ design}$ (k)</th>
<th>Change (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1546</td>
<td>510</td>
<td>155</td>
<td>155</td>
<td>38</td>
<td>76</td>
<td>155</td>
<td>155</td>
<td>9.4%</td>
</tr>
<tr>
<td>217</td>
<td>249</td>
<td>174</td>
<td>174</td>
<td>9.8%</td>
<td>9.7%</td>
<td>9.7%</td>
<td>9.7%</td>
<td>9.7%</td>
</tr>
</tbody>
</table>
B. ASCE 41-13 CHECKLISTS
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**B. ASCE 41-13 CHECKLISTS**

**FIG. 4-1. Tier 1 Evaluation Process**

**Required Information:**
- Level of Performance
- Level of Seismicity
- General Bldg. Description

Chapters 2 & 3

**Benchmark Building?**
- Yes
  - Selection of Checklists
    - Section 4.4
  - Complete the Basic Configuration Checklist
    - Sections 16.1
  - Complete the Building System Structural Checklist
    - Sections 16.2 – 16.16
  - Complete the Nonstructural Checklist
    - Section 16.17
  - Summarize Deficiencies
  - Further Evaluation Required?
    - Yes
      - Complete the Level of Very Low Seismicity Checklist
    - No

**No**
- Very Low Level of Seismicity & Life-Safety Level of Performance?
  - Yes
    - Complete the Basic Configuration Checklist
    - Quick Checks
  - No
    - Complete the Building System Structural Checklist
    - Quick Checks

- Very Low Seismicity IO or Low, Moderate, or High Seismicity (LS/IO)?
  - Yes
    - Complete the Basic Configuration Checklist
    - Quick Checks
  - No

- Selection of Checklists
  - Section 4.4

- Complete the Level of Very Low Seismicity Checklist
  - Section 16.1.1

- Further Evaluation Required?
  - Yes
    - Complete the Level of Very Low Seismicity Checklist
  - No
## APPENDIX C
### SUMMARY DATA SHEET

**BUILDING DATA**

- **Building Name:** Lake Oswego City Hall
- **Date:** January 2017
- **Building Address:** 380 "A" Avenue Lake Oswego, Oregon 97034
- **Latitude:** 45.419399
- **Longitude:** -122.667684
- **Year Built:** 1986
- **Area (ft²):** 34,000 (+21,000 parking)
- **No. of Stories:** 3
- **Original Design Code:** UBC 1982

**USE**
- [ ] Industrial
- [x] Office
- [ ] Warehouse
- [ ] Hospital
- [ ] Residential
- [ ] Educational
- [ ] Other:

**CONSTRUCTION DATA**

- **Gravity Load Structural System:** Steel beams and columns with TJLX and TJI joists
- **Exterior Transverse Walls:** Non-structural metal studs with EIFS exterior
- **Exterior Longitudinal Walls:** Non-structural metal studs with EIFS exterior
- **Roof Materials/Framing:** 3/4" tongue and groove plywood decking with built up roof
- **Intermediate Floors/Framing:** HSS columns, TJLX’s, TJLX, wood sheathing, 1 1/2" Gypcrete, and WF girders
- **Ground Floor:** WF and concrete beams, metal deck with concrete topping slab
- **Columns:** HSS steel
- **Foundation:** Spread ftgs, Grade bms
- **General Condition of Structure:** Poor. Exterior cladding is failing, noticeable beam damage at ground floor
- **Levels Below Grade:** Concrete columns, walls, beams, WF beams. Hollowcore planks at parking
- **Special Features and Comments:** Large cantilevers of flexible wood diaphragm

**LATERAL-FORCE-RESISTING SYSTEM**

<table>
<thead>
<tr>
<th>System</th>
<th>Longitudinal</th>
<th>Transverse</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vertical Elements:</td>
<td>CMU and Concrete shear walls</td>
<td>Same</td>
</tr>
<tr>
<td>Diaphragms:</td>
<td>Wood sheathing</td>
<td>Same</td>
</tr>
<tr>
<td>Connections:</td>
<td>Poor</td>
<td>Same</td>
</tr>
<tr>
<td>Load bearing shear walls (cores):</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**EVALUATION DATA**

- **BSE-IN Spectral Response Accelerations:** $S_D = 0.593$, $S_{so} = 0.659$, $S_{sl} = 0.349$, $S_{s1} = 0.185$
- **Soil Factors:** $F_s = 1.2$, $F_{so} = 1.691$
- **Level of Seismicity**: High, Performance Level: Life Safety
- **Building Period:** $T = 0.312$
- **Spectral Acceleration:** $S_s = 0.593$
- **Modification Factor:** $C_aC_lC_1 = 1.1$, Building Weight: $W = 1400k$
- **Pseudo Lateral Force:** $V = 830k$, $914k$

**BUILDING CLASSIFICATION:**

- **REOUIRED TIER 1 CHECKLISTS**
  - [x] Basic Configuration Checklist
  - [x] Building Type RM1,C2A Structural Checklist
  - [ ] Nonstructural Component Checklist

**Seismic Evaluation and Retrofit of Existing Buildings**
### 16.1.2LS  LIFE SAFETY BASIC CONFIGURATION CHECKLIST

#### Low Seismicity

**Building System**

**General**

- **LOAD PATH**: The structure shall contain a complete, well-defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation. (Commentary: Sec. A.2.1.1. Tier 2: Sec. 5.4.1.1)

- **ADJACENT BUILDINGS**: The clear distance between the building being evaluated and any adjacent building is greater than 4% of the height of the shorter building. This statement shall not apply for the following building types: W1, W1a, and W2. (Commentary: Sec. A.2.1.2. Tier 2: Sec. 5.4.1.2)

- **MEZZANINES**: Interior mezzanine levels are braced independently from the main structure or are anchored to the seismic-force-resisting elements of the main structure. (Commentary: Sec. A.2.1.3. Tier 2: Sec. 5.4.1.3)

**Building Configuration**

- **WEAK STORY**: The sum of the shear strengths of the seismic-force-resisting system in any story in each direction is not less than 80% of the strength in the adjacent story above. (Commentary: Sec. A.2.2.2. Tier 2: Sec. 5.4.2.1)

- **SOFT STORY**: The stiffness of the seismic-force-resisting system in any story is not less than 70% of the seismic-force-resisting system stiffness in an adjacent story above or less than 80% of the average seismic-force-resisting system stiffness of the three stories above. (Commentary: Sec. A.2.2.3. Tier 2: Sec. 5.4.2.2)

- **VERTICAL IRREGULARITIES**: All vertical elements in the seismic-force-resisting system are continuous to the foundation. (Commentary: Sec. A.2.2.4. Tier 2: Sec. 5.4.2.3)

- **GEOMETRY**: There are no changes in the net horizontal dimension of the seismic-force-resisting system of more than 30% in a story relative to adjacent stories, excluding one-story penthouses and mezzanines. (Commentary: Sec. A.2.2.5. Tier 2: Sec. 5.4.2.4)

- **MASS**: There is no change in effective mass more than 50% from one story to the next. Light roofs, penthouses, and mezzanines need not be considered. (Commentary: Sec. A.2.2.6. Tier 2: Sec. 5.4.2.5)

- **TORSION**: The estimated distance between the story center of mass and the story center of rigidity is less than 20% of the building width in either plan dimension. (Commentary: Sec. A.2.2.7. Tier 2: Sec. 5.4.2.6)

#### Moderate Seismicity: Complete the Following Items in Addition to the Items for Low Seismicity.

**Geologic Site Hazards**

- **LIQUEFACTION**: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building’s seismic performance shall not exist in the foundation soils at depths within 50 ft under the building. (Commentary: Sec. A.6.1.1. Tier 2: Sec. 5.4.3.1)

- **SLOPE FAILURE**: The building site is sufficiently remote from potential earthquake-induced slope failures or rockfalls to be unaffected by such failures or is capable of accommodating any predicted movements without failure. (Commentary: Sec. A.6.1.2. Tier 2: Sec. 5.4.3.1)

- **SURFACE FAULT RUPTURE**: Surface fault rupture and surface displacement at the building site are not anticipated. (Commentary: Sec. A.6.1.3. Tier 2: Sec. 5.4.3.1)

#### High Seismicity: Complete the Following Items in Addition to the Items for Low and Moderate Seismicity.

**Foundation Configuration**

- **OVERTURNING**: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than 0.65. (Commentary: Sec. A.6.2.1. Tier 2: Sec. 5.4.3.3)

- **TIES BETWEEN FOUNDATION ELEMENTS**: The foundation has ties adequate to resist seismic forces where footings, piles, and piers are not restrained by beams, slabs, or soils classified as Site Class A, B, or C. (Commentary: Sec. A.6.2.2. Tier 2: Sec. 5.4.3.4)
# B. ASCE 41-13 CHECKLISTS

## 16.10LS  LIFE SAFETY STRUCTURAL CHECKLIST FOR BUILDING TYPES C2: CONCRETE SHEAR WALLS WITH STIFF DIAPHRAGMS AND C2A: CONCRETE SHEAR WALLS WITH FLEXIBLE DIAPHRAGMS

### Low and Moderate Seismicity

#### Seismic-Force-Resisting System

<table>
<thead>
<tr>
<th>Code</th>
<th>Code</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>C NC</td>
<td>N/A</td>
<td>COMPLETE FRAMES: Steel or concrete frames classified as secondary components form a complete vertical-load-carrying system. (Commentary: Sec. A.3.1.6.1. Tier 2: Sec. 5.5.2.5.1)</td>
</tr>
<tr>
<td>C NC</td>
<td>N/A</td>
<td>REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2.  (Commentary: Sec. A.3.2.1.1. Tier 2: Sec. 5.5.1.1)</td>
</tr>
<tr>
<td>C NC</td>
<td>N/A</td>
<td>SHEAR STRESS CHECK: The shear stress in the concrete shear walls, calculated using the Quick Check procedure of Section 4.5.3.3, is less than the greater of 100 lb/in.² or (2\sqrt{f_c^2}). (Commentary: Sec. A.3.2.2.1. Tier 2: Sec. 5.5.3.1.1)</td>
</tr>
<tr>
<td>C NC</td>
<td>N/A</td>
<td>REINFORCING STEEL: The ratio of reinforcing steel area to gross concrete area is not less than 0.0012 in the vertical direction and 0.0020 in the horizontal direction. (Commentary: Sec. A.3.2.2.2. Tier 2: Sec. 5.5.3.1.3)</td>
</tr>
</tbody>
</table>

#### Connections

<table>
<thead>
<tr>
<th>Code</th>
<th>Code</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>C NC</td>
<td>N/A</td>
<td>WALL ANCHORAGE AT FLEXIBLE DIAPHRAGMS: Exterior concrete or masonry walls that are dependent on flexible diaphragms for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections have adequate strength to resist the connection force calculated in the Quick Check procedure of Section 4.5.3.7. (Commentary: Sec. A.5.1.1. Tier 2: Sec. 5.7.1.1)</td>
</tr>
<tr>
<td>C NC</td>
<td>N/A</td>
<td>TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls. (Commentary: Sec. A.5.2.1. Tier 2: Sec. 5.7.2)</td>
</tr>
<tr>
<td>C NC</td>
<td>N/A</td>
<td>FOUNDATION DOWELS: Wall reinforcement is doweled into the foundation with vertical bars equal in size and spacing to the vertical wall reinforcing immediately above the foundation. (Commentary: Sec. A.5.3.5. Tier 2: Sec. 5.7.3.4)</td>
</tr>
</tbody>
</table>

#### High Seismicity: Complete the Following Items in Addition to the Items for Low and Moderate Seismicity.

#### Seismic-Force-Resisting System

<table>
<thead>
<tr>
<th>Code</th>
<th>Code</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>C NC</td>
<td>N/A</td>
<td>DEFLECTION COMPATIBILITY: Secondary components have the shear capacity to develop the flexural strength of the components. (Commentary: Sec. A.3.1.6.2. Tier 2: Sec. 5.5.2.5.2)</td>
</tr>
<tr>
<td>C NC</td>
<td>N/A</td>
<td>FLAT SLABS: Flat slabs or plates not part of the seismic-force-resisting system have continuous bottom steel through the column joints. (Commentary: Sec. A.3.1.6.3. Tier 2: Sec. 5.5.2.5.3)</td>
</tr>
<tr>
<td>C NC</td>
<td>N/A</td>
<td>COUPLING BEAMS: The stirrups in coupling beams over means of egress are spaced at or less than (d/2) and are anchored into the confined core of the beam with hooks of 135 degrees or more. The ends of both walls to which the coupling beam is attached are supported at each end to resist vertical loads caused by overturning. (Commentary: Sec. A.3.2.2.3. Tier 2: Sec. 5.5.3.2.1)</td>
</tr>
</tbody>
</table>

#### Connections

<table>
<thead>
<tr>
<th>Code</th>
<th>Code</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>C NC</td>
<td>N/A</td>
<td>UPLIFT AT PILE CAPS: Pile caps have top reinforcement, and piles are anchored to the pile caps. (Commentary: Sec. A.5.3.8. Tier 2: Sec. 5.7.3.5)</td>
</tr>
</tbody>
</table>

#### Diaphragms (Flexible or Stiff)

<table>
<thead>
<tr>
<th>Code</th>
<th>Code</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>C NC</td>
<td>N/A</td>
<td>DIAPHRAGM CONTINUITY: The diaphragms are not composed of split-level floors and do not have expansion joints. (Commentary: Sec. A.4.1.1. Tier 2: Sec. 5.6.1.1)</td>
</tr>
<tr>
<td>C NC</td>
<td>N/A</td>
<td>OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 25% of the wall length. (Commentary: Sec. A.4.1.4. Tier 2: Sec. 5.6.1.3)</td>
</tr>
</tbody>
</table>

Seismic Evaluation and Retrofit of Existing Buildings 473
### Flexible Diaphragms

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th>CROSS TIES: There are continuous cross ties between diaphragm chords. (Commentary: Sec. A.4.1.2. Tier 2: Sec. 5.6.1.2)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>STRAIGHT SHEATHING: All straight sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered. (Commentary: Sec. A.4.2.1. Tier 2: Sec. 5.6.2)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>SPANS: All wood diaphragms with spans greater than 24 ft consist of wood structural panels or diagonal sheathing. (Commentary: Sec. A.4.2.2. Tier 2: Sec. 5.6.2)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 ft and aspect ratios less than or equal to 4-to-1. (Commentary: Sec. A.4.2.3. Tier 2: Sec. 5.6.2)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>OTHER DIAPHRAGMS: The diaphragm does not consist of a system other than wood, metal deck, concrete, or horizontal bracing. (Commentary: Sec. A.4.7.1. Tier 2: Sec. 5.6.5)</td>
</tr>
</tbody>
</table>
### B. ASCE 41-13 CHECKLISTS

#### 16.15LS LIFE SAFETY STRUCTURAL CHECKLIST FOR BUILDING TYPES RM1: REINFORCED MASONRY BEARING WALLS WITH FLEXIBLE DIAPHRAGMS AND RM2: REINFORCED MASONRY BEARING WALLS WITH STIFF DIAPHRAGMS

**Low and Moderate Seismicity**

**Seismic-Force-Resisting System**

<table>
<thead>
<tr>
<th>Item</th>
<th>Status</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2.</td>
<td>NC</td>
<td>(Commentary: Sec. A.3.2.1.1. Tier 2: Sec. 5.5.1.1)</td>
</tr>
<tr>
<td>SHEAR STRESS CHECK: The shear stress in the reinforced masonry shear walls, calculated using the Quick Check procedure of Section 4.5.3.3, is less than 70 lb/in².</td>
<td>NC</td>
<td>(Commentary: Sec. A.3.2.4.1. Tier 2: Sec. 5.5.3.1.1)</td>
</tr>
<tr>
<td>REINFORCING STEEL: The total vertical and horizontal reinforcing steel ratio in reinforced masonry walls is greater than 0.002 of the wall with the minimum of 0.0007 in either of the two directions; the spacing of reinforcing steel is less than 48 in., and all vertical bars extend to the top of the walls.</td>
<td>NC</td>
<td>(Commentary: Sec. A.3.2.4.2. Tier 2: Sec. 5.5.3.1.3)</td>
</tr>
</tbody>
</table>

**Stiff Diaphragms**

<table>
<thead>
<tr>
<th>Item</th>
<th>Status</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>TOPPING SLAB: Precast concrete diaphragm elements are interconnected by a continuous reinforced concrete topping slab.</td>
<td>NC</td>
<td>(Commentary: Sec. A.4.5.1. Tier 2: Sec. 5.6.4)</td>
</tr>
</tbody>
</table>

**Connections**

<table>
<thead>
<tr>
<th>Item</th>
<th>Status</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>WALL ANCHORAGE: Exterior concrete or masonry walls that are dependent on the diaphragm for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections shall have adequate strength to resist the connection force calculated in the Quick Check procedure of Section 4.5.3.7.</td>
<td>NC</td>
<td>(Commentary: Sec. A.5.1.1. Tier 2: Sec. 5.7.1.1)</td>
</tr>
<tr>
<td>WOOD LEDGERS: The connection between the wall panels and the diaphragm does not induce cross-grain bending or tension in the wood ledgers.</td>
<td>NC</td>
<td>(Commentary: Sec. A.5.1.2. Tier 2: Sec. 5.7.1.3)</td>
</tr>
<tr>
<td>TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls.</td>
<td>NC</td>
<td>(Commentary: Sec. A.5.2.1. Tier 2: Sec. 5.7.2)</td>
</tr>
<tr>
<td>TOPPING SLAB TO WALLS OR FRAMES: Reinforced concrete topping slabs that interconnect the precast concrete diaphragm elements are doweled for transfer of forces into the shear wall or frame elements.</td>
<td>NC</td>
<td>(Commentary: Sec. A.5.2.3. Tier 2: Sec. 5.7.2)</td>
</tr>
<tr>
<td>FOUNDATION DOWELS: Wall reinforcement is doweled into the foundation.</td>
<td>NC</td>
<td>(Commentary: Sec. A.5.3.5. Tier 2: Sec. 5.7.3.4)</td>
</tr>
<tr>
<td>GIRDER–COLUMN CONNECTION: There is a positive connection using plates, connection hardware, or straps between the girder and the column support.</td>
<td>NC</td>
<td>(Commentary: Sec. A.5.4.1. Tier 2: Sec. 5.7.4.1)</td>
</tr>
</tbody>
</table>

**Flexible Diaphragms**

<table>
<thead>
<tr>
<th>Item</th>
<th>Status</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 25% of the wall length.</td>
<td>NC</td>
<td>(Commentary: Sec. A.4.1.4. Tier 2: Sec. 5.6.1.3)</td>
</tr>
<tr>
<td>OPENINGS AT EXTERIOR MASONRY SHEAR WALLS: Diaphragm openings immediately adjacent to exterior masonry shear walls are not greater than 8 ft long.</td>
<td>NC</td>
<td>(Commentary: Sec. A.4.1.6. Tier 2: Sec. 5.6.1.3)</td>
</tr>
</tbody>
</table>

**Seismic Evaluation and Retrofit of Existing Buildings**

Lake Oswego City Hall
January 27, 2017

B-6
<table>
<thead>
<tr>
<th>Category</th>
<th>Checklist Description</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Seismic Assessment</strong></td>
<td><strong>STRAIGHT SHEATHING</strong>: All straight sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered. (Commentary: Sec. A.4.2.1. Tier 2: Sec. 5.6.2)</td>
</tr>
<tr>
<td><strong>B-7</strong></td>
<td><strong>SPANS</strong>: All wood diaphragms with spans greater than 24 ft consist of wood structural panels or diagonal sheathing. (Commentary: Sec. A.4.2.2. Tier 2: Sec. 5.6.2)</td>
</tr>
<tr>
<td><strong>Connections</strong></td>
<td><strong>DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS</strong>: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 ft and aspect ratios less than or equal to 4-to-1. (Commentary: Sec. A.4.2.3. Tier 2: Sec. 5.6.2)</td>
</tr>
<tr>
<td></td>
<td><strong>OTHER DIAPHRAGMS</strong>: The diaphragm shall not consist of a system other than wood, metal deck, concrete, or horizontal bracing. (Commentary: Sec. A.4.7.1. Tier 2: Sec. 5.6.5)</td>
</tr>
<tr>
<td></td>
<td><strong>STIFFNESS OF WALL ANCHORS</strong>: Anchors of concrete or masonry walls to wood structural elements are installed taut and are stiff enough to limit the relative movement between the wall and the diaphragm to no greater than 1/8 in. before engagement of the anchors. (Commentary: Sec. A.5.1.4. Tier 2: Sec. 5.7.1.2)</td>
</tr>
</tbody>
</table>
C. ASCE 41-13 CALCULATIONS

Design Maps Summary Report

User-Specified Input

Report Title Lake Oswego City Hall
Wed December 21, 2016 18:19:14 UTC

(which utilizes USGS hazard data available in 2008)

Site Coordinates 45.4194°N, 122.66768°W

Site Soil Classification Site Class D – “Stiff Soil”

USGS–Provided Output

\[ S_{S,20/50} = 0.291 \, g \quad S_{S,S,E-1E} = 0.456 \, g \]
\[ S_{1,20/50} = 0.109 \, g \quad S_{X1,S,E-1E} = 0.259 \, g \]

Although this information is a product of the U.S. Geological Survey, we provide no warranty, expressed or implied, as to the accuracy of the data contained therein. This tool is not a substitute for technical subject-matter knowledge.
C. ASCE 41-13 CALCULATIONS

11/22/2016

Design Maps Detailed Report

ASCE 41-13 Retrofit Standard, BSE-1E (45.4194°N, 122.6676°W)

Site Class C – “Very Dense Soil and Soft Rock”, Risk Category IV (e.g. essential facilities)

Section 2.4.1 – General Procedure for Hazard Due to Ground Shaking

20%/50-year maximum direction spectral response acceleration for 0.2s and 1.0s periods, respectively:

\[
\text{From Section 2.4.1.4} \quad S_{s,20/50} = 0.291 \text{ g}
\]

\[
\text{From Section 2.4.1.4} \quad S_{s,10/50} = 0.109 \text{ g}
\]

Section 2.4.1.6 – Adjustment for Site Class

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class C, based on the site soil properties in accordance with Section 2.4.1.6.1.

<table>
<thead>
<tr>
<th>SITE CLASS</th>
<th>SOIL PROFILE NAME</th>
<th>Soil shear wave velocity, (v_s) (ft/s)</th>
<th>Standard penetration resistance, (N)</th>
<th>Soil undrained shear strength, (\bar{\sigma}_{uu}) (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Hard rock</td>
<td>(v_s &gt; 5,000)</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>B</td>
<td>Rock</td>
<td>(2,500 &lt; v_s \leq 5,000)</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>C</td>
<td>Very dense soil and soft rock</td>
<td>(1,200 &lt; v_s \leq 2,500)</td>
<td>(\bar{N} &gt; 50)</td>
<td>&gt;2,000 psf</td>
</tr>
<tr>
<td>D</td>
<td>Stiff soil profile</td>
<td>(600 \leq v_s &lt; 1,200)</td>
<td>(15 \leq \bar{N} \leq 50)</td>
<td>1,000 to 2,000 psf</td>
</tr>
<tr>
<td>E</td>
<td>Stiff soil profile</td>
<td>(v_s &lt; 600)</td>
<td>(\bar{N} &lt; 15)</td>
<td>&lt;1,000 psf</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
| E          |                   | Any profile with more than 10 ft of soil having the characteristics:
|            |                   | 1. Plasticity index \(PI > 20\),
|            |                   | 2. Moisture content \(w \geq 40\%\), and
|            |                   | 3. Undrained shear strength \(\bar{\sigma}_{uu} < 500\) psf |
| F          |                   | Any profile containing soils having one or more of the following characteristics:
|            |                   | 1. Soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soils.
|            |                   | 2. Peats and/or highly organic clays (\(H > 10\) feet of peat and/or highly organic clay where \(H =\) thickness of soil)
|            |                   | 3. Very high plasticity clays (\(H > 25\) feet with plasticity index \(PI > 75\))
|            |                   | 4. Very thick soft/medium stiff clays (\(H > 120\) feet) |

For SI: \(1\text{ft/s} = 0.3048 \text{ m/s} \quad 1\text{lb/ft}^2 = 0.0479 \text{ kN/m}^2\)
Table 2–3. Values of $F_s$ as a Function of Site Class and Mapped Short-Period Spectral Response Acceleration $S_s$

<table>
<thead>
<tr>
<th>Site Class</th>
<th>Mapped Spectral Acceleration at Short-Period $S_s$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$S_s \leq 0.25$</td>
</tr>
<tr>
<td>A</td>
<td>0.8</td>
</tr>
<tr>
<td>B</td>
<td>1.0</td>
</tr>
<tr>
<td>C</td>
<td><strong>1.2</strong></td>
</tr>
<tr>
<td>D</td>
<td>1.6</td>
</tr>
<tr>
<td>E</td>
<td>2.5</td>
</tr>
<tr>
<td>F</td>
<td>Site-specific geotechnical and dynamic site response analyses shall be performed</td>
</tr>
</tbody>
</table>

Note: Use straight-line interpolation for intermediate values of $S_s$

**For Site Class = C and $S_s = 0.291$ g, $F_s = 1.200$**

Table 2–4. Values of $F_s$ as a Function of Site Class and Mapped Spectral Response Acceleration at 1 s Period $S_1$

<table>
<thead>
<tr>
<th>Site Class</th>
<th>Mapped Spectral Acceleration at 1 s Period $S_1$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$S_1 \leq 0.10$</td>
</tr>
<tr>
<td>A</td>
<td>0.8</td>
</tr>
<tr>
<td>B</td>
<td>1.0</td>
</tr>
<tr>
<td>C</td>
<td><strong>1.7</strong></td>
</tr>
<tr>
<td>D</td>
<td>2.4</td>
</tr>
<tr>
<td>E</td>
<td>3.5</td>
</tr>
<tr>
<td>F</td>
<td>Site-specific geotechnical and dynamic site response analyses shall be performed</td>
</tr>
</tbody>
</table>

Note: Use straight-line interpolation for intermediate values of $S_1$

**For Site Class = C and $S_1 = 0.109$ g, $F_s = 1.691$**
C. ASCE 41-13 CALCULATIONS

Provided as a reference for Equation (2-4):

\[ F_a S_{S,20/50} = 1.200 \times 0.291 \text{ g} = 0.349 \text{ g} \]

Provided as a reference for Equation (2-5):

\[ F_v S_{S,1,20/50} = 1.691 \times 0.109 \text{ g} = 0.185 \text{ g} \]

Provided as a reference for Equation (2-4):

\[ S_{X, BSE-1N} = \frac{2}{3} \times S_{X, BSE-2N} = \frac{2}{3} \times F_a S_{S, BSE-2N} = 0.659 \text{ g} \]

Provided as a reference for Equation (2-5):

\[ S_{X, BSE-1N} = \frac{2}{3} \times S_{X, BSE-2N} = \frac{2}{3} \times F_v S_{S, BSE-2N} = 0.387 \text{ g} \]

Equation (2-4):

\[ S_{X, BSE-1E} = \text{MIN}[F_a S_{S,20/50}, S_{X, BSE-1N}] = \text{MIN}[0.349g, 0.659g] = 0.349g \]

Equation (2-5):

\[ S_{X, BSE-1E} = \text{MIN}[F_v S_{S,20/50}, S_{X, BSE-1N}] = \text{MIN}[0.185g, 0.387g] = 0.185g \]

Section 2.4.1.7.1 — General Horizontal Response Spectrum

Figure 2-1. General Horizontal Response Spectrum

\[ S_e = \begin{cases} 
0 < T < T_0: & S_{X, S} \left( \frac{T}{T_0} - 2 \right) \frac{T}{T_0} + 0.4 \\
T_0 < T < T_y: & S_{X, S} \left( \frac{T}{T_y} - 2 \right) \frac{T}{T_y} \\
T_y < T: & S_{X, S} \left( \frac{T}{T_y} \right)^2 
\end{cases} \]

Spectral Response Acceleration, \( S_e \) (g)

Spectral Response Acceleration, \( S_e \) (g) vs. Period, \( T \) (sec)
Section 2.4.1.7.2 — General Vertical Response Spectrum

The General Vertical Response Spectrum is determined by multiplying the General Horizontal Response Spectrum by $\frac{2}{3}$.

![Diagram showing spectral response acceleration (Sa) vs. period (T) for general vertical response spectrum.](image)
C. ASCE 41-13 CALCULATIONS

ASCE 41-13 ANALYSIS FOR LA CITY HALL

**Roof Weight**: Roof area = 11,100 ft²

- Roof Insulation: 1 psf BUILT UP ROOF
- Sheathing: 2 psf 3/4" Plywood Decking
- Ceiling: 3 psf Suspended Acoustic Ceiling
- Lights: 1 psf
- Mech.: 2 psf
- Misc.: 1.5 psf

**Rebar**: 12.4 psf (11400 sq ft) = 143.84 k

- W12 x 22 (22.5 fl) = 0.59 k
- W12 x 216 (21.6 fl) = 0.035 k
- W12 x 216 (21.6 fl) = 1.12 k
- W216 x 216 (21.6 fl) = 1.14 k
- W216 x 216 (21.6 fl) = 1.20 k
- W216 x 216 (21.6 fl) = 1.20 k
- W216 x 216 (21.6 fl) = 1.20 k

**Concrete**: (4" thick)

- Area = (20 ft x 10 ft) = 200 sq ft
- 150 psf = (4") x (10") = 10 k

**Walls**

- (3) Core Walls: 10.5 ft x 7 ft
- 8" CHU Grout @ Rein (24" O.C.)
- 510 psf = (0.54 ft x 2.5 ft x 2) x 1.8 k = 477.94 k

**Exterior**

- 6" Steel Studs: 43 m2 @ (10") O.C.
- Perimeter: 4814.4
- (1.83 psf + 1.5 psf + 1.5 psf) = 24.09 k

**Roof**: 143.84 k

**Felt**: 20.04 k

- TJI: 18.25 k
- Walls: 477.94 k

- 24.09 k

**By**: OQ5

**Date**: 1/27/17

**Job**: 21U0339.00

**Stock**: of

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M.

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Architecture + Interiors + Planning + Engineering

Lake Oswego City Hall
January 27, 2017

C-6
SECOND AND THIRD FLOOR WEIGHT
AER = 13,400 ft²

1 ⅜” concrete 13 psf
Sheathing 2.4 psf
Ceiling 3 psf
Lintel 1 psf
Misc. 1.5 psf
Misc. wood 1.5 psf
Steel beams 3.4 psf
Joists 3.2 psf

Total:
51 psf
51 psf (12 sf) (2 (8 sf) + 2 (18 sf)) (3 courses) = 95.47 K

EXTENSION:
6” steel studs 43.8 ml @ 12” o.c.
1.43 psf

Studs, gypsum, EPS insulation
(1.93 psf + 1.05 psf + 2.5 psf) (23 sf) (504 ft²) = 50.38 K

SECOND FLOOR CONCRETE CONSTRUCTION:
8” concrete + CMU

150 psf (8½”) (7½”) (2 (8½”) + 2 (18¾”) (3 courses) = 117 K
CMU 8” adair @ 18” o.c.
(51 psf) (6 sf) (2 (8½”) + 2 (18¾”) (3 courses) = 117 K

EXTENSION:
6” steel studs 43 ml @ 16” o.c.
(1.83 psf + 1.05 psf + 1.5 psf) (15.5 sf) (504 ft²) = 56.65 K

STAIR BEAMS
12” x 10” 30.14 long 30 sf T/E
101 lb/ft² / 100 = 3.14 psf

Joists (T/E)
4.75 lb/ft² @ 18” o.c.
4.25 lb/ft² / 13.25 = 3.2 psf

WALLS = 3rd
(3) Core Walls (23sf/TL)
8” CMU wall
Ground @ Rainw.
Cells (24” o.c.)
51 psf

51 psf (12 sf) (2 (8 sf) + 2 (18 sf)) (3 courses) = 95.47 K

PERIMETER: 121” + 91” + 30” + 29” + 32” + 30” + 13” + 32” + 13” = 541’

3rd WEIGHT FLOOR: 388.6K
Walls: 95.17 K
50.38 K
531.5 K

2nd WEIGHT FLOOR: 388.6 K
Walls: 114 K
47.71 K
56.48 K
610 K

By OQK
Date 1/22/14
2160377.00

C-7

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Seismic Assessment
C. ASCE 41-13 CALCULATIONS

FIRST FLOOR WEIGHT \( A = 11,900 \text{ ft}^2 \)

Floors: 38.7 psf 2 1/2" CONCRETE SLAB ON DECK
Ceilings: 3 psf SUSPENDED ACOUSTIC CEILING
Lighting: 1 psf
MISC. 1.5 psf
MISC. Wood 1.5 psf
Steel Beams 3.1 psf

Walls:
(1) Core Walls (155 ft HT.) 8 ft
B' Conc. Wall 10 ft

MATERIALS

STEEL BEAMS

<table>
<thead>
<tr>
<th>Width (in)</th>
<th>Depth (in)</th>
<th>lbs/lin ft</th>
<th>LF</th>
</tr>
</thead>
<tbody>
<tr>
<td>12</td>
<td>22</td>
<td>92</td>
<td>8</td>
</tr>
<tr>
<td>12</td>
<td>22</td>
<td>92</td>
<td>8</td>
</tr>
<tr>
<td>12</td>
<td>22</td>
<td>92</td>
<td>8</td>
</tr>
</tbody>
</table>

Total Weight: 92 + 92 + 92 = 276 lbs/lin ft

CONCRETE WALLS

|M" Conc. Wall | 109.5 psf (10,000 sq ft) | 110 k |

Concentrated Load: 120 lb/ft

PARKING LOT

10' Hollow Core Pavers 67 psf
Concrete Sidewalk 51 psf 2 1/2" CONCRETE SLAB
Misc. 1.5 psf
Concrete Beams

B' Conc. Beams

ANALYSIS

<table>
<thead>
<tr>
<th>1st Floor Weight:</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>Floor: 627.3 ft²</td>
<td>627.3 k</td>
</tr>
<tr>
<td>Walls: 234.4 ft²</td>
<td>234.4 k</td>
</tr>
<tr>
<td>1st Floor Weight:</td>
<td>861.7 k</td>
</tr>
</tbody>
</table>

TOTAL: 2670.5 k

1124.2 k
C. ASCE 41-13 CALCULATIONS

ASCE TIER 1 ANALYSIS - WITH POSITIVE

PERIOD: \( T = \frac{C_h}{n^2} \) (ASCE41-4-5)

\( n = 21 \) ft to roof line (not measured)

\( p = 0.125 \)

\( C_h = 0.02 \)

\( T = 0.02 \times (54)^{0.25} = 0.398 \)

SPECTRAL ACCELERATION \( S_a = \frac{S_k}{T} \) (ASCE41-4-4)

\( S_k = 0.185 \)

\( T = 0.398 \)

\( S_a = \frac{0.185}{0.398} = 0.465 \)

SEISMIC PSEUDO FORCE

\( V = C_s a W \) (ASCE41-4-4)

\( C_s = 1.0 \)

\( V = (1.0)(0.185)(407.5 \text{ k}) = 189.5 \text{ k} \)

STORY SHEAR FORCE

\( F_x = \frac{W_1 h_k}{k} V_k = 1.5 \) (ASCE41-4-3a)

Roof:

\( F_{roof} = \frac{W_{roof} h_d}{k} V = \frac{2,540.5 \text{ k}(15.5 \text{ k}) + 610 \text{ k}(30 \text{ k})}{15.5 \text{ k} + 534.5 \text{ k}(4.2 \text{ k}) + 254.5 \text{ k}(54 \text{ k})} \approx 8.76 \text{ k} = 94,645 \text{ k} \)

2nd:

\( F_3 = \frac{2,531.5 \text{ k}(4.2 \text{ k})}{964.5} = 449.5 \text{ k} \)

2nd:

\( F_2 = \frac{(2,670.5 \text{ k})(30 \text{ k})}{964.5} = 360.4 \text{ k} \)

1st:

\( F_1 = \frac{2,719.5 \text{ k}(54 \text{ k})}{964.5} = 808.8 \text{ k} \)

\( V_y = \frac{F_x}{k} \)

\( V_{roof} = 275.3 \text{ k} \)

\( V_3 = 275.3 \text{ k} + 449.5 \text{ k} = 724.8 \text{ k} \)

\( V_2 = 724.8 \text{ k} + 360.4 \text{ k} = 1,091.2 \text{ k} \)

\( V_1 = 1,091.2 \text{ k} + 808.8 \text{ k} = 1,899.7 \text{ k} \)
C. ASCE 41-13 CALCULATIONS

ASCE TUBE 1 ANALYSIS - 1.0 PONDUM

PERIOD: \( T = \frac{\pi}{\sqrt{g \cdot h}} \) (ASCE 41 4-5)
\( h = 23.5 \text{ ft} \)
\( g = 32.2 \text{ ft/s}^2 \)
\( T = 0.312 \text{ s} \)
\( S_0 = \frac{5x1}{T} \) (ASCE 41 4-4)
\( S_0 = 0.185 \)
INTERSECT SCALING FORCE

SEISMIC REDUCTION FACTOR

\( V = C \cdot S_0 \cdot W \) (ASCE 41 4-1)
\( C = 1.0 \)
\( V = 1.0 \cdot (0.593) \cdot (1000) = 830 \text{ K} \)

STUDY SHEAR FORCE

\( F_x = \frac{w_i \cdot h_i}{1} \cdot \frac{V}{K} = 1.0 \)

ROOF: \( F_{\text{roof}} = \frac{w_{\text{roof}} \cdot h_{\text{roof}}}{33510.9} \cdot V \)
\( = \frac{254.6 \text{ k} \cdot (39.5 \text{ ft})}{33510.9} = 244 \text{ k} \)

3rd: \( F_3 = \frac{531.5 \text{ k} \cdot (22.6 \text{ ft})}{33510.9} = 368 \text{ k} \)

2nd: \( F_2 = \frac{610 \text{ k} \cdot (15.5 \text{ ft})}{33510.9} = 237 \text{ k} \)

\( V = \frac{1}{2} \cdot F_x \)

\( V_{\text{roof}} = 244 \text{ k} \)
\( V_3 = 368 \text{ k} \)
\( V_2 = 237 \text{ k} \)

Building Weight Without Podium

1st: 254.6 k
2nd: 531.5 k
3rd: 610 k
1400 k
Localized Mass Locations - Center of Mass Calculation
C. ASCE 41-13 CALCULATIONS

Center of Mass Calculation - 2nd and 3rd Floor

\[ m_1 = 29 \text{ psf} \times (40 \text{ ft})(65.6 \text{ ft}) = 114.1 \text{ k} \]
\[ y_1 = 32.8 \text{ ft} \]

\[ m_2 = 29 \text{ psf} \times (39.3 \text{ ft})(15.4 \text{ ft}) = 17.6 \text{ k} \]
\[ y_2 = 14.5 \text{ ft} \]

\[ m_3 = 29 \text{ psf} \times (90.5 \text{ ft})(30.5 \text{ ft}) = 783 \text{ k} \]
\[ y_3 = 74.5 \text{ ft} \]

\[ m_4 = 29 \text{ psf} \times (35.5 \text{ ft})(60.1 \text{ ft}) = 61.8 \text{ k} \]
\[ y_4 = 101.5 \text{ ft} \]

\[ m_5 = (18.1 \text{ psf})(18.3 \text{ ft})(60.5 \text{ ft}) = 19.5 \text{ k} + 164.7 \text{ k} \text{ (walls)} \]
\[ y_5 = 74.5 \text{ ft} \]

\[ m_6 = 29 \text{ psf} \times (15.5 \text{ ft})(30.5 \text{ ft}) = 13.1 \text{ k} \]
\[ y_6 = 104.5 \text{ ft} \]

\[ m_7 = 29 \text{ psf} \times (0.5 \text{ ft})(30.5 \text{ ft}) = 52.2 \text{ k} \]
\[ y_7 = 59.5 \text{ ft} \]

\[ \sum m_c = 511.3 \text{ k} \]
\[ \bar{y} = \frac{114.1 (32.8) + 12.6 (14) + 78.3 (74) + 61.8 (101.5) + 164.7 (74) + (13.1) (104.5) + 52.2 (59)}{511.3} \]
\[ \bar{y} = 45.9 \text{ ft} \]

Center of Rigidity is 45.9 ft - only separated by about 8 ft < 20% building width
C. ASCE 41-13 CALCULATIONS

- With podium
  - Grade beam depth 30 ft.
  - Building Ht. = 54 ft.
  - \( \frac{20}{54} = 0.370 \) 50.0 sq

0.0 sq = 0.0 \((0.592)\) = 0.354 < 0.350 COMPLIANT
C. ASCE 41-13 CALCULATIONS

SIMPLE STRESS CHECK - WITH PODIUM

CONCRETE = N = S

$V_c = \frac{1}{1.5} \left( \frac{V_c}{h_{w}} \right)$

$V_c = 3000$ psi

$V_c = 2.0$

8" THICK CONCRETE SIDE WALL

$\text{Wall Length} = 4.935 + 7 + 8.667 + 8.667 + 7 + 8.667 = 41 - 4"$

$41 - 4" (8/12) = 29.56 SQFT = 4250 \text{ in}^2$

$V_c = \frac{1}{2.0} \left( \frac{109.724 \text{ in}^2}{4250 \text{ in}^2} \right) = 128.2 \text{ in}^3$

$2.5\sqrt{c} = 2 \times 1000 = 109.5 \text{ lb/in}^2 < 128.2 \text{ lb/in}^2$

$\text{NOT COMPLIANT}$

$V_{1,2} = \frac{1}{2.0} \left( \frac{V_{1,2}}{h_{w}} \right)$

$\text{Wall Length} = 189.5 \text{ in}^2 \times \frac{1000 \text{ lb}}{1 \text{ k}} = 202.8 \text{ k}

$202.8 \text{ k} < 2 \times 1000 = 126.5 \text{ lb/in}^2$

$\text{NOT COMPLIANT}$

CONCRETE - E TO W

$V_{3,4} = \frac{1}{2.0} \left( \frac{V_{3,4}}{h_{w}} \right)$

$\text{Wall Length} = 189.5 \text{ in}^2 \times \frac{1000 \text{ lb}}{1 \text{ k}} = 202.8 \text{ k}

$202.8 \text{ k} < 2 \times 1000 = 126.5 \text{ lb/in}^2$

$\text{COMPLIANT}$
**Seismic Assessment**

**C. ASCE 41-13 CALCULATIONS**

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**SHEAR STRENGTH CHECK – NO PODIUM**

**Concrete – H-702**

\[ V_{20\alpha g} = \frac{1}{M_s} \left( \frac{V_e}{f_{dw}} \right) \]

\[ = \frac{1}{2.0} \left( \frac{830k}{47.5k/sq ft} \right) \]

\[ = 97.5 \text{ psi} \]

\[ 2.5' \times 2.0' \times 2.5' = 109.5' > 97.5 \text{ psi} \]  **CONFORMANT**

**Concrete – E-Town**

\[ V_{20\alpha g} = \frac{1}{M_s} \left( \frac{V_e}{f_{dw}} \right) \]

Wall Length: \(16'-4" + 12'-4" = 30' - 8"\)

\[ A_w = 30' - 8" (8") = 320 \text{ sq ft} \]

\[ V_{20\alpha g} = \frac{1}{2.0} \left( \frac{830k}{3200} \right) = 117.9 \text{ psi} > 109.5 \text{ psi} \]  **NON CONFORMANT**

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**M**

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C. ASCE 41-13 CALCULATIONS

REINFORCING STEEL

VERT. @ 5' 15" O.C. 6 @ 9" EACH SIDE

VERT.

AREA FOR 15" SECTION

\[ AC = \frac{15''}{8''} = 120\text{in}^2 \]

\[ A_s = 0.31\text{in}^2 \]

\[ \frac{A_s}{AC} = \frac{0.31}{120} = 0.0026 > 0.0012 \text{ COMPLIANT} \]

100#-

AREA FOR 15" SECTIONS

\[ A_s = \frac{15''}{8''} = 120\text{in}^2 \]

\[ A_s = 0.31\text{in}^2 \]

\[ \frac{A_s}{AC} = \frac{0.31}{120} = 0.0026 > 0.0020 \text{ COMPLIANT} \]

TRANSFER TO SHEAR WALLS

Dimension: \( \frac{1}{16}'' \) CIP concrete over \( \frac{1}{4}'' \) plywood backing.
C. ASCE 41-13 CALCULATIONS

Shear Stress Check - With Pounding

Masonry: 16" thick, masonry shearwalls

\[ V_{m} = \frac{1}{m} \left( \frac{V_{total}}{A_{w}} \right) \]

8" thick, masonry shearwalls

\[ V_{total} = 4 \times 3.21 + 3 \times 8 \times 0.7 + 8 \times 0.7^2 + 7 + 8 \times 0.7^2 = 44.4' \text{ in} = 53.2 \text{ in} \]

\[ A_{w} = 622.1 \text{ in}^2 \]

\[ V_{m} = \frac{1}{2} \left( \frac{74.5 \text{ in}^2}{472 \text{ in}^2} \right) = 32.3 \text{ psi} < 70 \text{ psi} \text{ - COMPLIANT} \]

\[ V_{s} = \frac{1}{m} \left( \frac{V_{s}}{A_{w}} \right) \]

\[ V_{s} = \frac{1}{2} \left( \frac{7.2 \times 42.4 \text{ in}^2}{36 \text{ in}^2} \right) = 91.8 \text{ psi} > 70 \text{ psi} \text{ - NOT COMPLIANT} \]

Eroa

\[ V_{erola} = \frac{1}{m} \left( \frac{V_{erola}}{A_{w}} \right) \]

\[ V_{erola} = \frac{1}{2} \left( \frac{275.3 \text{ in}^2}{94.9 \text{ in}^2} \right) = 14.5 \text{ psi} < 70 \text{ psi} \text{ - COMPLIANT} \]

\[ V_{s} = \frac{1}{m} \left( \frac{V_{s}}{A_{w}} \right) \]

\[ V_{s} = \frac{1}{2} \left( \frac{74.5 \text{ in}^2}{472 \text{ in}^2} \right) = 32.3 \text{ psi} < 70 \text{ psi} \text{ - COMPLIANT} \]
C. ASCE 41-13 CALCULATIONS

SHEAR STRESS CHECK - NO PODIUM

MASSIVE N 4 TO SOUTHWEST M_S = 2.10

V_{roof} = \frac{1}{m_S} \left( \frac{V_{roof}}{b_w} \right)

8" THICK MASSIVE

WALL \quad \text{WALL AVERAGE} = \frac{4.53'' + 7'' + 8.64'' + 8.64'' + 7'' + 8.64'' = 49.4''}{6} = 8.24'' = 209.4 M

W\_w = 532.1 \frac{8.24}{12} = 425.1 M^2

V_{roof} = 1.2 \left( \frac{24 L}{1250 \times 1250} \right) = 28.9 \text{ psi} < 70 \text{ psi} \quad \text{COMPLIANT}

V_{3} = \frac{1}{m_S} \left( \frac{V_3}{b_w} \right)

WALL \quad \text{WALL AVERAGE} = \frac{7'' + 7'' + 8.64'' + 8.64''}{4} = 7.88'' = 200 M

W\_w = 317.8 \frac{200}{12} = 532.1 \text{ M}^2

V_{3} = 1.2 \left( \frac{644}{532.1 \times 532.1} \right) = 7.9 \text{ psi} > 70 \text{ psi} \quad \text{NOT COMPLIANT}

WALL \quad \text{WALL AVERAGE} = \frac{18'' + 9'' + 36.0'' - 8''}{4} = 13.38'' = 339 M

W\_w = 840 \frac{339}{12} = 254 \text{ M}^2

V_{roof} = 1.2 \left( \frac{189.4}{254 \times 254} \right) = 34.8 \text{ psi} < 70 \text{ psi} \quad \text{COMPLIANT}

V_{3} = \frac{1}{m_S} \left( \frac{V_3}{b_w} \right)

WALL \quad \text{WALL AVERAGE} = \frac{18'' + 9'' + 36.0'' - 8''}{4} = 13.38'' = 339 M

W\_w = 840 \frac{339}{12} = 254 \text{ M}^2

V_{3} = 1.2 \left( \frac{644}{254 \times 254} \right) = 85.8 \text{ psi} > 70 \text{ psi} \quad \text{NOT COMPLIANT}
C. ASCE 41-13 CALCULATIONS

DRAG STRUT CAPACITIES

4.1.4 DRAG STRUT 30°F

\[ b = 3.5 \text{ in} \]
\[ d = 3.5 \text{ in} \]
\[ \frac{b}{d} = \frac{3.5}{3.5} = 1.0 \]

**Compression:**

\[ N_D = 3.7 \times 1.1 \]  
SLEWING RATIO (b/d) SHALL NOT EXCEED 50  

\[ T' = F' A \]
\[ A = (2.5'' - 1'') (3.5'') = 8.45 \text{ in}^2 \]
\[ F' = F_e C_o \]
\[ C_o = (3.5'') (3.5) = 12.15 \text{ in}^2 \]
\[ 12.15 \text{ in}^2 \times 1500 \text{ psi} = 18,225 \text{ psi} \]

**Tension:**

\[ T' = F' A \]
\[ A = (2.5'' - 1'') (3.5) = 12.15 \text{ in}^2 \]
\[ 12.15 \text{ in}^2 \times 1500 \text{ psi} = 18,225 \text{ psi} \]

3/8" x 10" 618, DRAG STRUT 30°F

\[ b = 3.5 \text{ in} \]
\[ d = 3.5 \text{ in} \]
\[ \frac{b}{d} = \frac{3.5}{3.5} = 1.0 \]

**Compression:**

\[ N_D = 3.7 \times 1.1 \]  
SLEWING RATIO (b/d) SHALL NOT EXCEED 50  

**Tension:**

\[ T' = F' A \]
\[ F' = F_e C_o \]
\[ C_o = (2.5'' - 1'') (3.5) = 12.15 \text{ in}^2 \]
\[ 12.15 \text{ in}^2 \times 1500 \text{ psi} = 18,225 \text{ psi} \]

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3. Retrofit Floor Plans
D. RETROFIT FLOOR PLANS

Lake Oswego City Hall
January 27, 2017

Seismic Assessment
2160377.00

BASEMENT FLOOR PLAN

Add to existing foundation per sketch F1
Strengthen wall/columns w/ shotcrete and reinforcing per sketches S1 and B6.
Dowel to enlarged footing per sketch S2

Strengthen concrete walls w shotcrete and reinforcing. Improve wall connections to foundations per sketches S1 and S3.

Enlarge foundations as required w/ reinforcing and concrete per sketch F1

New continuous foundation
New braced frame at transition per sketch B2

Improve existing concrete shearwalls w/ concrete and reinforcing per sketch S1.
Dowel to enlarged footing per sketch S2

Improve existing concrete shearwalls w/ concrete and reinforcing per sketch S1.
Dowel to enlarged footing per S2.

Add to existing foundation per sketch F1
Strengthen column w/ 4" add'l shotcrete and reinforcing similar to sketch B6.
Enlarge foundations per sketch F1
Lake Oswego City Hall
January 27, 2017

Seismic Assessment
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D. RETROFIT FLOOR PLANS

THIRD FLOOR PLAN

Strap between wood joists across girder per sketch V1, typ.
New braced frame, beams, and column per sketch B4
New steel braced frame per sketch B1
Improve steel connection as chord/tie/dragstrut per sketch V2, typ.
New braced frame and column per sketch B1
New braced frame and column per sketch B1
Replace existing 4x4 dragstrut w/ HSS per sketch V3
Replace existing GLB dragstrut w/ HSS per sketch V3
Replace existing 4x4 dragstrut w/ HSS per sketch V3
Replace existing GLB dragstrut w/ HSS per sketch V3
Strengthen walls w/ shotcrete/reinforcing per sketch S1
New braced frames, columns, and beams per sketch B3
City Hall Existing Conditions Assessment
380 A Avenue
Lake Oswego, OR 97034

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Replace existing 4x4 dragstrut w/ HSS per sketch V3
Replace existing GLB dragstrut w/ HSS per sketch V3

Improve steel connection as chord/tie/dragstrut per sketch V2, typ.

Strap between wood joists across girder per sketch V1, typ.

ROOF PLAN

D. RETROFIT FLOOR PLANS