INFORMATIONAL MEMORANDUM

TO: Finance and Governance Committee
FROM: David Cline, City Administrator
BY: Hari Ponnekanti, Public Works Director/ City Engineer
Rachel Bianchi, Deputy City Administrator
CC: Mayor Allan Ekberg
DATE: July 22, 2022
SUBJECT: City Facilities Overview and Seismic Update for City Hall, 6300 building and TCC

ISSUE
The City's adopted Strategic Plan Goal #4, *High Performing Organization*, includes a strategy to "Ensure City facilities are safe, efficient, and inviting to the public." As part of this strategy the City has invested in its facilities over several decades, most recently completing the Council-adopted and community-supported Public Safety Plan. This investment included the opening of the new Justice Center, two new Fire Stations (51 and 52), and completing Phase 1 of the Public Works Consolidated Operations Center (Fleet and Facility Building). These buildings provide safe, efficient, and inviting facilities for our first responders for the next several decades.

Staff is seeking Council review, input, and recommendation on the next phase of facility planning and investments for the six-year Council Adopted Capital Improvement Plan (CIP). This memo will outline the history and current state of recent facility investments, provide an updated seismic study of three key facilities (City Hall, 6300 Building, and the Tukwila Community Center), and provide options for the next phase of facility planning.

BACKGROUND
Historically, the City has managed its facility investments in three main areas:

1. Facilities Maintenance – including landscaping, minor improvements, and custodial services. The city added three new buildings to the facilities without adding additional staffing to Parks or Public Works, who maintain the outside and inside of the buildings, respectively.
2. Major Maintenance – including painting, roof and siding repairs, HVAC. In the 2021-2022 Adopted CIP this included new siding and painting of the Tukwila Community Center, painting of Fire Station 53, and a planned investment in siding and painting of City Hall
3. New Facilities – most recently planned for and budgeted through specific funding sources such as the Public Safety Bond approved by the public in 2016 and Councilmanic financing.

Overview of Past Facility Planning – 2008 – 2022
In 2008, Reid Middleton conducted an in-depth seismic evaluation which was presented to City Council in September 2008. Based on the information received at that time, Council requested that staff return to the Finance & Safety Committee with a recommendation on a program that would entail all costs associated with the project including timelines and funding options. The Great Recession of 2009-2010 delayed action on this plan. However, it was picked up again in 2013 and resulted in the *Essential Government Services Facilities Plan* adopted by the Council in 2015, which culminated in the City's 2016 Public Safety Plan.

2008 Seismic Study
The Reid Middleton seismic evaluation revealed that 10 of 11 essential post-earthquake City facilities failed seismic evaluation for immediate occupancy. This does not make the City's buildings or its current seismic situation an outlier from other employers or governments that have facilities of the same age. This means that these 10 facilities would not be habitable, for customers or employees, after a substantial earthquake. (Some of this was addressed during the City's 2016 Public Safety Plan, which added four new facilities and took three seismically deficient ones offline.) It is likely that essential services performed in these buildings would not be provided during an incident or the recovery period following a major seismic event.

NOTE: The Golf/Parks Maintenance Building, Foster Clubhouse, and Fire Station 53 were not included in the 2008 Seismic Program because they were constructed under more recent seismic code.

Essential Government Facilities Plan – 2013-2015
Recognizing the need for a comprehensive study and plan, in 2013 the Council approved funding and contract for a Facilities Master Plan. The City Council adopted this plan in 2015, which outlined the current conditions of the facilities, the future needs of facilities and estimated costs for implementation. In 2016, the City Council prioritized the needs of First Responders and created the Public Safety Plan, which included a Public Safety Bond approved by voters in November 2016. Since then, the City has opened a new Justice Center, two new fire stations, and completed Phase 1 of the Public Works Shops Consolidated Facility, the Fleet and Facility Building. This added four new seismically safe buildings to the City's inventory and removed three seismically-deficient buildings from significant use.

Police and Court employees are currently housed in a safe facility at the Justice Center. The City also included in that facility an emergency operations center, which is vital to recovery in an emergency. The City's fire stations 51 & 52 are new and are built seismically safe. The new Fleet and Facility Building, through Council direction and additional funding, is built to a higher seismic standard as well.

Public Works Shops Consolidated Facility – Phase 1 and Phase 2
As part of the Council adopted Public Safety Plan, the City purchased properties in 2018 for a Consolidated Public Works Maintenance and Administration facilities. The new Fleet and Facility tenant improvements, the first part of the planned multi-phase Consolidated Public works campus, were completed in June 2022.

In May 2022, the Council directed staff to put forth a Request for Qualifications (RFQ) for a consultant to design Phase 2 of the Public Works Operations Campus (Streets and Utilities). This process will be brought back to Council in August 2022. The 2015 Facility Study envisioned a combined City Shops facility (currently Minkler and George Long Shops located in separate locations) that will improve safety and efficiency. Other partnerships with agencies have been identified to provide efficiencies and potential revenues. These partnerships include a new decant facility, which will also allow for storage and handling of vactor waste, potential new customers for Fleet services, and Police vehicle evidence storage. The sale of the Minkler and George Long Shops may be used to help offset the cost of the new facility. Additionally, it is anticipated that the City's utility enterprise funds will pay for half of the construction associated with Phase 2.

2022 Seismic Study Update
As part of the Facilities Maintenance Plan, the City contracted with Reid Middleton in early 2022 to update the seismic studies for three city facilities (City Hall, 6300 Building, and Tukwila Community Center). As part of the update Reid Middleton has also updated the cost estimates.

The multi-building seismic update report indicates that City Hall, 6300 Building, and TCC are inadequate to resist design-level earthquake forces and do not meet the ASCE 41-17 performance objectives, including the Collapse Prevention (CP) performance objective. This does not mean that the buildings are unsafe, but it indicates that upgrades are required for the buildings to perform better in an earthquake scenario. While the three buildings do not meet ASCE 41-17 performance objectives, this does not make them outliers from buildings of similar age and construction. Buildings designed prior to the current building code often include structural configurations and connections detailing that, based on post-earthquake evaluations of damaged buildings, have historically contributed to poor seismic performance in structures.

Current List of City Facilities and Conditions

City Hall – 6200 Southcenter Boulevard	
Date Constructed	1977
Total Square Feet	25,075
Current Use	Mayor's Office, Finance, Clerk's Office, Legislative Analyst, Council Chambers, Records Center
Historical Modifications or changes:	None
Seismic Status:	Does not meet Life Safety or Collapse Prevention criteria (2022 Seismic Study Update)
Known Needed Upgrades:	Seismic bracing, HVAC and other Mechanical Electronical Plumbing (MEP) modifications, lighting upgrades to meet efficiency standards, ADA restrooms; reconfiguration to maximize use.*
Cost Information	2022 seismic estimates to increase to Life Safety status is $4.57M and $4.46M for Collapse Prevention. Expect costs to increase in five years to $6.1 and $6M, respectively.
Prior Council Direction	Council approved 2015 Facilities Needs Assessment that contemplated completely renovating City Hall by moving staff into the 6300 Building on a temporary basis, gutting, renovating and enlarging the building. Council allocated $100K in 2022 City Hall painting and siding repairs; City staff expects that the bids will come in higher than what was budgeted.

* Note: Some upgrades could trigger required additional upgrades per the State Building Code.

6300 Building – 6300 Southcenter Boulevard	
Date Constructed	1978
Total Square Feet	32,950

Current Use	Community Development, Public Works Engineering, Human Resources, Community Services & Engagement, Technology & Innovation Services, Fire Marshal's Office, Sound Cities Association (tenant).
Historical Modifications or changes:	Nothing structural, internal suite modifications over the years.
Seismic Status:	Does not meet Life Safety or Collapse Prevention criteria (2022 Seismic Study Update).
Known Needed Upgrades:	Seismic bracing, HVAC and other Mechanical Electronical Plumbing (MEP) modifications, roof replacement, lighting upgrades to meet efficiency standards; reconfiguration to maximize use. 2015 Facilities Needs Assessment indicated that lifecycle costs should be compared against building replacement costs – in other words, it may be more efficient to demolish and build new than renovate this specific building.*
Cost Information	2022 seismic estimates to increase to Life Safety status is $3.1M and $13.6M for Collapse Prevention. Expect costs to increase in five years to $4.1 and $18.2M, respectively.
Prior Council Direction	Council approved 2015 Facilities Needs Assessment that contemplated demolishing the 6300 building after serving as an interim City Hall while upgrades are made to the 6200 Building.

* Note: Some upgrades could trigger required additional upgrades per the State Building Code.

Tukwila Community Center	
Date Constructed	1995
Total Square Feet	55,000
Current Use	Parks and Recreation Administration, Recreation Programming and Classes, Workout and Gym, Rentals and Community Events. Designated as City's official Emergency Shelter.
Historical Modifications or changes:	None
Seismic Status:	Does not meet Life Safety or Collapse Prevention criteria (2022 Seismic Study Update); likely in liquefaction zone due to adjacency to Duwamish River.
Known Needed Upgrades:	Seismic bracing, new HVAC system and other Mechanical Electronical Plumbing (MEP) modifications, exterior improvements and backup generator needed.

Cost Information	To bring the building to seismic Immediate Occupancy level (necessary for it to be an Emergency Shelter), cost would be $13.7, with five-year escalation going to $18.7. Seismic upgrade costs would be lower if retrofitted to a Life Safety or Collapse Prevention standard, but new location for emergency shelter would need to be identified. HVAC replacement in the $3M range; current ask of $1.8M to Senator Patty Murray for a member-directed request.
Prior Council Direction	Council allocated $150K in 2020 for TCC siding repairs, $10K for HVAC repairs in 2021 and $140K for TCC painting and staining in 2021 as a part of the 303 fund.

Minkler Shops	
Date Constructed	1972
Total Square Feet	4,700 sq ft workroom and storage building; 7,200 sq ft office and garage building; 8,850 sq ft covered parking structure; and 300 sq ft restroom and shower modular building.
Current Use	Streets, Sewer/Surface Water, Water maintenance and operations; traffic operations; material storage; sign shop; offices; vehicle washing bay
Historical Modifications or changes:	Renovations to former bays to turn them into offices and other cosmetic alterations; no structural improvements.
Seismic Status:	2008 study found the buildings do not meet Immediate Occupancy levels. Site susceptible to liquefaction given adjacency to Green River. Unknown whether the buildings meet Life Safety or Collapse Prevention levels.
Known Needed Upgrades:	New facilities.
Cost Information	Updated costs to be developed during schematic design of next phase of consolidated shops facilities; previous cost estimates too out of date to be useful.
Prior Council Direction	Council approved 2015 Facilities Needs Assessment that included moving Minkler operations to the consolidated Public Works facility on land acquired in 2018 as a part of the Public Safety Plan and sell the Minkler to help finance the Public Safety Plan. Council allocated $500K for improvements to facility, including $280K for modular restroom/shower facilities that were installed in 2021. Additional near-future improvements include security fencing and lighting

Parks & Golf Maintenance	
Date Constructed	1998
Total Square Feet	8,890
Current Use	Parks and golf maintenance; material and equipment storage.
Historical Modifications or changes:	None
Seismic Status:	Not included in 2008 or 2022 study, so unknown.
Known Needed Upgrades:	Exterior maintenance
Cost Information	Unknown
Prior Council Direction	None

Fire Station 53	
Date Constructed	1995
Total Square Feet	7,392
Current Use	Fire Station
Historical Modifications or changes:	None
Seismic Status:	2008 study indicated no retrofit required; updated study needed to know whether this is still accurate.
Known Needed Upgrades:	No known seismic upgrades needed; confirmed same in 2022
Cost Information	Unknown
Prior Council Direction	Council allocated $50K to paint the exterior of the building this year; project complete. Fire Station 53 was not included in the Public Safety Plan.

Fire Station 54	
Date Constructed	1961
Total Square Feet	5,398
Current Use	Fire Station
Historical Modifications or changes:	Remodeled and expanded on the east side in 1990.
Seismic Status:	2008 study found the buildings do not meet Immediate Occupancy levels. Unknown whether the buildings meet Life Safety or Collapse Prevention levels.
Known Needed Upgrades:	New facility
Cost Information	Would need to be determined during schematic design.
Prior Council Direction	Council removed Fire Station 54 from the Public Safety Plan due to construction escalation associated with the overall Program.

Other City Owned Facilities	
Former Fire Station 51 *Andover Park East*	Fire Department use ended in 2020. Currently part of the budling being used for vehicle evidence storage for the Police Department. Previous Council direction was to sell to help finance the Public Safety Plan.
Former Fire Station 52 *Top of Tukwila Hill*	Fire Department use ended in 2020. Previous Council direction was to sell to identify community input on what to do with the site. Community event was held in 2020, general community interest in having a facility there/expanding the park but understanding that the current building may cost more to retrofit than to demolish and rebuild, particularly given the architectural and structural deficiencies. Former Fire Station 52 site, park and former City Hall/Library building are all one parcel and deed restricted for community use as the land was donated to the City by the School District in 1946.
Former Allentown Fire Station *42nd Ave. S., across from river*	Previous Council direction was to sell to help finance the Public Safety Plan.
George Long Shops *Interurban Ave.*	Previous Council Direction was to sell to help finance the Public Safety Plan. Sale in progress
Newporter Site (vacant land) *TIB & 148th*	Previous Council Direction was to sell to help finance the Public Safety Plan.
Longacres Site (vacant land) *Near Train Stop/RR tracks*	Previous Council Direction was to sell to help finance the Public Safety Plan.

Facility Planning for 2023-2024

Recognizing facility planning history and current needs, it would be helpful to create an updated plan to help prioritize the next investments in City facilities. This could include a review of all of remaining facilities or provide a more limited scope such as a focus on key facilities like City Hall/6300. The last direction provided by City Council in the 2015 Facilities Needs Assessment assumed that City Hall should be renovated and the 6300 building should be demolished.

COVID has upended many organizations' actual needs for facilities, and this should be taken into account as well. For example, some organizations – including neighboring local governments – have decreased the space needs for individual employees due to hybrid/remote work possibilities like shared workspaces or hoteling. Some organizations have gone completely remote and eliminated needs for facilities.

Current plans

- Public Works Phase 2 – planned to return to Council in August 2022 for direction on design
- Teen and Senior Center – pending direction from City Council
- City Hall/6300 Building – Siding/Painting planned in 2022 for City Hall
- TCC – Currently seeking federal grants to upgrade HVAC systems.

In the past, City funding has been allocated using a variety of funding options. Most recently, the adopted and updated Public Safety Plan included voter approved funds, councilmanic debt obligations, utility funds, sale of facilities, and other sources. Future facility plans will also need a diverse set of funding options as the General Fund alone will not be adequate to address these needs.

RECOMMENDATION

Staff recommends keeping current planning efforts on track, including moving forward with the design of Phase 2 of the Public Works Operations Campus, Streets and Utilities. Further, Staff recommends including funding for an updated facilities plan in the upcoming biennial budget to specifically focus on next steps for the City Hall Campus – City Hall and the 6300 Building – as well as necessary upgrades to the Tukwila Community Center's HVAC and other MEP (Mechanical, Electrical, Plumbing) needs. The recommended updated facilities plan would allow the City to better understand the need for space at the City Hall campus in light of the new realities of remote work. The plan would provide proposed funding, phasing and timelines that would allow the Council to make informed decisions on the next steps regarding investing in the City Hall campus and TCC. Staff estimates the proposed study to cost between $250,000 and $350,000.

Staff recommends holding on additional significant investments in Fire Stations 53 and 54 – while recognizing the current state of Fire Station 54 – until the City better understands the long-term relationship with the Puget Sound Regional Fire Authority (PSRFA). Currently the City is working to engage in a short-term contract with PSRFA with a goal of annexation within two years. If annexation is successful, the PSRFA would take over City fire stations in one capacity or another, and it makes sense for that effort to work itself out before determining next steps in investing in those two buildings.

Staff recognizes the Council also has deep interest in the proposed Teen and Senior Center project initiated by the Council in the fall of 2019. However, this has been on hold as the City determines next steps with current budget realities and the Council was provided this overview on existing City facility obligations. Staff is hoping that the Council's overall facilities discussion can provide some direction on whether this project should move forward at this time to Schematic Design or potentially hold until annexation into the PSRFA, which could provide additional opportunities for the project.

ATTACHMENTS

1. Multi building Seismic upgrade Report by Reid Middleton, June 2022 (with appendices)

The following documents were referenced in the memo, and can be made available upon request:

- 2008 Seismic Study
  Tukwila Seismic Report & Appendices 8.13.08.pdf
- 2015 Facilities Needs Assessment (with appendices)
  http://www.tukwilawa.gov/wp-content/uploads/PW-Project-FS-Facilities-Study-12-14-15-Report-DRAFT.pdf

CITY OF TUKWILA

MULTI-BUILDING SEISMIC ASSESSMENTS UPDATE

Final Submittal
June 2022

PREPARED FOR

PREPARED BY

Reid Middleton

CITY OF TUKWILA
MULTI-BUILDING SEISMIC ASSESSMENTS UPDATE

ASCE 41-17 Tier 1 and Tier 2 Seismic Evaluations
Updates of City Hall, 6300 Building, and the Tukwila Community Center

June 2022

6/20/2022

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Table of Contents

Figures

Tables

Appendix

1.0 Executive Summary

In 2008, Reid Middleton completed a seismic assessment of several City of Tukwila buildings. These evaluations were completed using ASCE 41-06 Tier 1, 2, and 3 procedures. For this report, the City of Tukwila desired an update to the study previously prepared by Reid Middleton, Inc., and submitted by Rice Fergus Miller Architecture & Planning PLLC, titled "City of Tukwila Architectural Assessment for Seismic Program," dated July 25, 2008. This assessment consists of updating the seismic study and concepts for Tukwila City Hall, the 6300 Building, and the Tukwila Community Center.

This report provides the results of a Tier 1 and Tier 2 deficiency-based seismic evaluation, conducted in accordance with the American Society of Civil Engineers' Standard 41-17, *Seismic Evaluation and Retrofit of Existing Buildings* (ASCE 41-17), and preliminary recommendations for the seismic upgrades required for the three buildings to meet the designated performance level. A Tier 3 evaluation was not completed for this update.

Reid Middleton used information from the field investigation and building record drawings to update the seismic evaluations of the three buildings to the current code, ASCE 41-17 *Seismic Evaluation and Retrofit of Existing Buildings*. The three buildings were previously evaluated to the Immediate Occupancy (IO) performance level. For the seismic update, City Hall and the 6300 Building have been revised to the Life Safety (LS) performance level, as they do not house emergency services and are not required to be operational after a seismic event. The Tukwila Community Center evaluation remained at the IO objective level, since it is an emergency shelter for the city.

The results of the seismic evaluation indicate that all three buildings are inadequate to resist design-level earthquake forces and do not meet the ASCE 41-17 performance objectives, including the Collapse Prevention (CP) performance objective. This does not mean that the buildings are unsafe, but it indicates that upgrades are required for the buildings to perform better in an earthquake scenario. Buildings that do not meet the CP performance level do not meet modern seismic code requirements for typical buildings. Buildings are evaluated for very large earthquakes that occur infrequently but are still possible. The chance of this large earthquake occurring in a given year is approximately 0.1%, meaning that it is 999 times as likely not to happen as it is to happen. The building is at an elevated risk of damage in a large earthquake, but the chances of a large earthquake occurring in a given year are relatively small.

While the 6300 Building, City Hall, and the Community Center do not meet ASCE 41-17 performance objectives, this does not make them outliers from buildings of similar age and construction. Buildings designed prior to the current building code often include structural configurations and connections detailing that, based on post-earthquake evaluations of damaged buildings, have historically contributed to poor seismic performance in structures. Additionally, recent research and studies of regional seismicity have shown that the expected seismic ground motions are higher than was expected in the past. Higher ground motions, structural configurations, and poor connection detailing may result in seismic evaluation deficiencies among buildings constructed to previous building code requirements.

This report includes a description of each building, the identified seismic deficiencies, seismic upgrade concept designs, and recommendations for upgrades. All three buildings were found to have seismic deficiencies, and none of the buildings meet the required performance objective. Concept-level seismic upgrade designs were completed for the three facilities, and concept plans are provided describing options for mitigation of seismic deficiencies. Recommendations consist of strengthening and supplementing the existing lateral systems, improving lateral load paths, and improving connections. An opinion of probable construction costs for the recommended structural upgrades is provided for each building.

2.0 Introduction and Seismic Evaluation Criteria

The seismic evaluations for the City of Tukwila buildings are based on the performance-based earthquake engineering (PBEE) guidelines presented in ASCE 41-17 *Seismic Evaluation and Retrofit of Existing Buildings* (American Society of Civil Engineers, 2017). This section includes a general background of PBEE and an overview of seismic retrofit objectives, seismic hazard levels, seismic performance levels, and seismic evaluation and retrofit procedures.

The seismic evaluations do not consider compliance with the seismic requirements of the current building code for new construction. Buildings designed prior to the current building code often include structural configurations and connections detailing that have historically contributed to poor seismic performance in structures, based on post-earthquake evaluations of damaged buildings. Additionally, recent research and studies of regional seismicity have shown that the expected seismic ground motions are higher than was expected in the past. Higher ground motions, structural configurations, and poor connection detailing may result in seismic evaluation deficiencies among buildings constructed to previous building code requirements. Buildings designed to older building code standards are evaluated using evaluation and design guidelines specifically developed for existing structures by the Federal Emergency Management Agency (FEMA) and the American Society of Civil Engineers (ASCE).

The structural findings and recommendations presented in this report are based on visual observations of the buildings and a review of the record drawings. The available record documents do not contain all of the information necessary to confirm the structural configuration of some portions of the buildings, which is typical for older structures.

Reid Middleton participated in a walk-through of City Hall, the 6300 Building, and the Tukwila Community Center on March 10, 2022. Visual observations of existing conditions were performed, which did not include destructive or nondestructive testing to confirm or supplement information shown in the record drawings.

The seismic evaluation of the buildings is based on the PBEE guidelines presented in ASCE 41-17. The ASCE 41 Tier 1 and Tier 2 evaluations of the buildings were completed using the Life Safety (LS) or Immediate Occupancy (IO) performance objective, depending on the building use. Buildings that meet the IO performance objective will have similar seismic performance to new buildings that are designed as essential facilities, while buildings that meet the LS performance objective will require repairs after a design-level seismic event.

2.1 Background

ASCE 41-17 employs a Performance-Based Design methodology that allows building owners, design professionals, and the local building authorities to establish seismic hazard levels and performance goals for individual buildings. PBEE is the engineering of a structure to resist earthquake demands while also meeting the needs and objectives of building owners and other stakeholders. PBEE allows for the design and analysis of structures for different levels of seismic performance and allows the levels of seismic performance to be related to the relative seismic hazard.

Seismic analysis and design of structures traditionally focused on one performance level – reducing the risk to loss of life in a design earthquake. The concept of designing essential facilities, which are needed immediately after an earthquake, to a higher performance standard evolved after hospitals and other critical facilities were damaged in the 1971 San Fernando, California, earthquake. That concept is balanced by the recognition that the cost of retrofitting existing buildings to higher levels of seismic performance may be onerous to both stakeholders and policy makers.

A comprehensive program was started in 1991, in cooperation with FEMA, to develop guidelines tailored to address this variation of performance levels. The first formal applications of performance-based evaluation and design guidelines were FEMA 310 *Handbook for the Seismic Evaluation of Buildings – A Prestandard (1998)* and FEMA 273 *NEHRP Guidelines for the Seismic Rehabilitation of Buildings (1997)*. After the release of these documents in the 1990s, three additional documents were released in the following years. Another prestandard document, FEMA 356 *Prestandard and Commentary for the Seismic Rehabilitation of Buildings*, was released in the year 2000. Then, the first national standard seismic evaluation document, ASCE 31-03 *Seismic Evaluation of Existing Buildings*, was released in the year 2003. Following the release of ASCE 31-03, the first national standard seismic rehabilitation document, ASCE 41-06 *Seismic Rehabilitation of Existing Buildings*, was released in the year 2007. ASCE 31-03 and ASCE 41-06 superseded the PBEE documents produced in the previous decade. ASCE 31-03 and ASCE 41-06 used the general framework outlined by previous documents but were updated to incorporate the latest standard of PBEE for the time.

ASCE 31-03 and ASCE 41-06 still had flaws, and soon after the release of ASCE 41-13, there was an effort undertaken to combine ASCE 31-03 and ASCE 41-06 into a single national standard in an attempt to streamline the documents and eliminate discrepancies. The newest PBEE document, ASCE 41-17 *Seismic Evaluation and Retrofit of Existing Buildings*, combines information from all of the previous documents, reflects advancements in technology and analysis techniques, and incorporates case studies and lessons learned from recent earthquakes.

ASCE 41-17 provides criteria by which existing structures can be seismically evaluated and retrofitted to attain a wide range of different performance levels when subjected to earthquakes of varying severity.

2.2 Seismic Hazard Levels

Earthquake ground motions are variable and complicated, and every earthquake is different. In addition, an earthquake's intensity and energy magnitude depend on fault type, fault movement, depth to epicenter, and soil strata. In earthquake-prone areas, often very small and frequent earthquakes occur every few days or weeks without being noticed by humans, but large earthquakes that occur much less frequently can have a devastating effect on infrastructure and can result in the temporary displacement of a large number of people. Earthquakes are also unpredictable, and the precise location, intensity, and start time of an earthquake cannot be predicted before an event occurs. However, earthquake hazards for certain geographic areas are well understood based on historical patterns of earthquakes from the geologic record, measured earthquake ground motions, understanding of plate tectonics, and seismological studies.

Geologists, seismologists, and geotechnical engineers have categorized the seismic hazard for particular locations using probabilistic seismic hazard levels. Each seismic hazard level describes a different probabilistic earthquake magnitude based on the probability of a certain magnitude earthquake occurring in a given time period. Table 2-1 shows commonly used seismic hazard levels, their corresponding probabilities of exceedance, and mean return periods.

Table 2-1. Probabilistic Seismic Hazard Levels and Mean Return Period.

Seismic Hazard Level	Probability of Exceedance in 50 Years	Mean Return Period (Years)
50%/50-year	50%	72
20%/50-year (BSE-1E)	20%	225
10%/50-year	10%	475
5%/50-year (BSE-2E)	5%	975
2%/50-year	2%	2,475

Seismic events with longer mean return periods and smaller probabilities of exceedance are seismic events that are associated with stronger seismic motions, larger ground accelerations, and more potential to damage facilities. Consequently, structures designed or retrofit to a seismic hazard level with a longer return period will generally experience better performance in an earthquake than a structure designed or retrofit to a lower seismic hazard level.

ASCE 41-17 codifies four different Seismic Hazard Levels at which to evaluate or retrofit structures. For voluntary seismic evaluations and voluntary seismic upgrades, the owner of a structure and the structural engineer can decide the Seismic Hazard Level at which it is appropriate to evaluate or retrofit a structure. The codified Seismic Hazard Levels are grouped into two categories: two Seismic Hazard Levels (BSE-1E and BSE-2E) associated with the Basic Performance Objectives for Existing Buildings (BPOE), and two Seismic Hazard Levels (BSE-1N and BSE-2N) associated with the Basic Performance Objectives Equivalent to New Building Standards (BPON).

Please note that the ASCE 41-17 defined Seismic Hazard Levels for existing buildings are shown in Table 2-1, along with their respective probabilities of exceedance and mean return period; however, the BSE-1N and BSE-2N Seismic Hazard Levels are not shown in Table 2-1 because they cannot be directly related to a probability of exceedance or mean return period. Instead, the BSE-2N Seismic Hazard Level is determined by a target risk of 1% chance of structural collapse in 50 years, and the BSE-1N is taken as two-thirds of the BSE-2N. The 1% risk of collapse does not correspond to actual expected collapse rates[1], but it is a theoretical risk target used to compare various regions across the United States with different seismic hazards. Structures designed for heightened performance objectives (Immediate Occupancy, Damage Control) will

[1] Federal Emergency Management Agency (FEMA) P-1050 (2015) "NEHRP Recommended Seismic Provisions for New Buildings and Other Structures."

have a lower risk of collapse. Historically (and in previous standards), the BSE-2N Seismic Hazard Level was taken as the 2%/50-year earthquake, and the BSE-1N was taken as the 10%/50-year earthquake.

Historically, existing buildings have been seismically evaluated and retrofitted to a lower Seismic Hazard Level than would be typical in new building design. This approach has been historically justified for three primary reasons:

1. It ensures recently constructed structures are not immediately rendered seismically deficient due to minor building code changes.
2. Existing buildings often have a shorter remaining life than a new building would; therefore, lower structural resiliency is tempered by a decreased probability of a major seismic event.
3. Often the burdensome cost of retrofitting historic structures to a "new building equivalence" performance level is disproportionate to the incremental benefit.

2.3 Building Performance Levels and Seismic Retrofit Objectives

A target building performance level must be selected for the design or retrofit of a structure. The target performance levels are discrete damage states selected from among the infinite spectrum of possible damage states that a building could experience during an earthquake. The terminology used for target building performance levels is intended to represent goals for design but not necessarily predict building performance during an earthquake.

Since actual ground motions during an earthquake are seldom comparable to those used for design, the target building performance level may only determine relative performance during most events but not predict the actual level of damage following an event. Even given a ground motion similar to that used in design, variations from stated performance objectives should be expected. Variations in actual performance could be associated with differences in the level of workmanship, variations in actual material strengths, deterioration of materials, unknown geometry and sizes of existing members, differences in assumed and actual live loads in the building at the time of the earthquake, influence of nonstructural components, and variations in response of soils beneath the building.

ASCE 41-17 describes performance levels for structural components and nonstructural components of a structure. Historically, much attention was provided to the seismic performance of structural components. However, in recent years, it has been realized that attention to the seismic performance of nonstructural components can be just as important as or more important than the seismic performance of structural components. The ASCE 41-17 identified Structural Performance Levels are shown in Table 2-2, and the ASCE 41-17 identified Nonstructural Performance Levels are shown in Table 2-3.

Table 2-2. Identified Structural Performance Levels.

Performance Level Abbreviation	Performance Level Name
S-1	Immediate Occupancy
S-2	Damage Control
S-3	Life Safety
S-4	Limited Safety
S-5	Collapse Prevention
S-6	Structural Performance Not Considered

Table 2-3. Identified Nonstructural Performance Levels.

Performance Level Abbreviation	Performance Level Name
N-A	Operational
N-B	Position Retention
N-C	Life Safety
N-D	Hazards Reduced
N-E	Nonstructural Performance Not Considered

Individual Structural Performance Levels and Nonstructural Performance Levels can be aggregated to form a combined Building Performance Level. Structural performance during an earthquake is related to the amount of lateral deformation or drift of the structure and the capacity or ability of the structure to deform. Any Structural Performance Level can be combined with any Nonstructural Performance Level, although it is not recommended to combine high levels of structural performance with low levels of nonstructural performance and vise-versa.

Theoretically, there are 23 different Building Performance Levels that are combinations of different Structural Performance Levels and Nonstructural Performance Levels. However, ASCE 41-17 recommends that only 15 Building Performance Levels be used in practice due to their recommendation of avoiding mismatching high and low levels of nonstructural and structural performance. ASCE 41-17 defines four specific common Building Performance Levels, as shown in Table 2-4. Figure 2-1 shows a visual representation of these common Building Performance Levels plotted against lateral deformation.

Table 2-4. Specific Common Building Performance Levels.

Performance Level Abbreviation	Performance Level Name	Structural & Nonstructural Performance Level Combination
1-A	Operational	S-1 & N-A
1-B	Immediate Occupancy	S-1 & N-B
3-C	Life Safety	S-3 & N-C
5-D	Collapse Prevention	S-5 & N-D

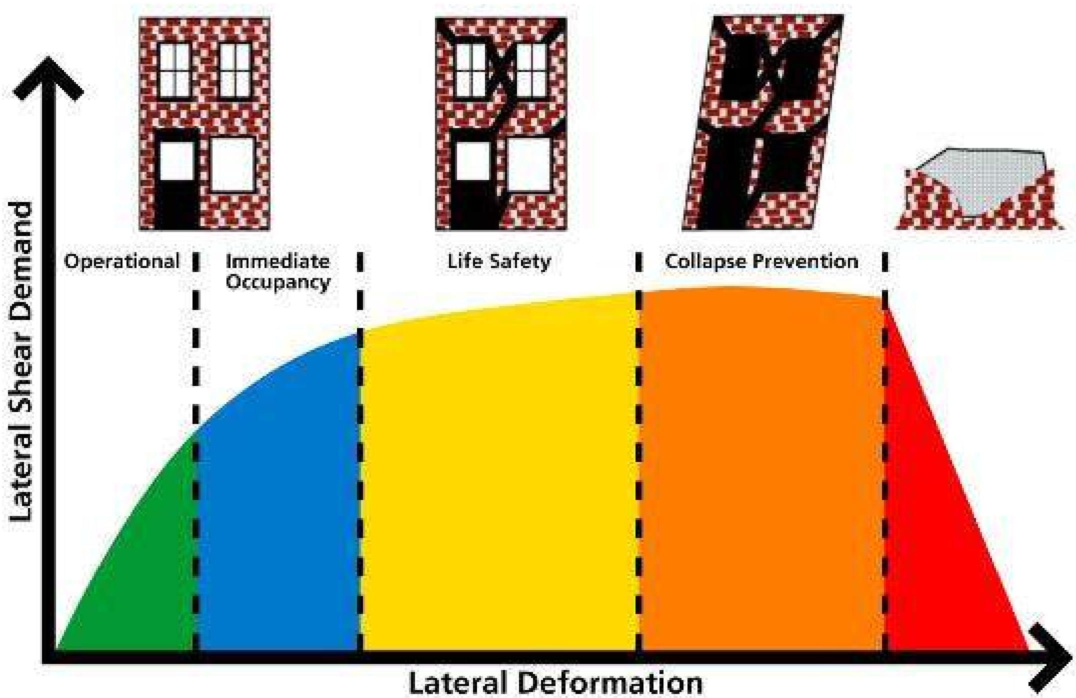

Figure 2-1. Building Performance Levels.

A decision must be made for each structure as to the acceptable behavior for different levels of seismic hazard, balanced with the construction cost of retrofitting a structure to obtain that behavior. ASCE 41-17 defines “baseline” basic performance objectives for structures based on their defined Risk Category. The Risk Category is the same that is defined in the International Building Code and ASCE 7. For example, for a Risk Category II structure retrofitted to the BPON standards, the structure would need to be retrofitted for the 3-B Building Performance Level at the BSE-1N Seismic Hazard Level and the 5-D Building Performance Level at the BSE-2N Seismic Hazard Level. ASCE 41-17 allows for higher (enhanced) or lower (limited) objectives to be selected based on the essential nature of the facility, the expected remaining life of the building, and the associated cost and feasibility. For example, it may not be economically feasible to retrofit historic structures to the BPON standards, and ASCE 41 allows for selection of a limited objective for such situations.

A building meeting the Immediate Occupancy performance level may sustain very minor damage but remains safe to occupy and retains its pre-earthquake strength and stiffness. Nonstructural components may sustain damage but are still securely anchored to the building structure to prevent falling or breaking of utility connections. Building access and life safety equipment, such as doors, stairways, elevators, emergency lighting, and fire suppression systems, remain operational.

A building meeting the Life Safety performance level may sustain damage while still protecting occupants from life-threatening injuries and allowing occupants to exit the building. Structural and nonstructural components may be extensively damaged, but some margin against the onset of partial or total collapse remains. Injuries to occupants or persons in the immediate vicinity may occur during an earthquake; however, the overall risk of life-threatening injuries as a result of structural damage is anticipated to be low. Repairs may be required before reoccupying the building, and in some cases, repairs may be economically unfeasible.

A building meeting the Collapse Prevention performance level is expected to sustain significant structural and nonstructural damage. This is the lowest performance level considered for building structures. At the Collapse Prevention level, the risk of injury to occupants is moderate and the structure is not likely repairable after an earthquake.

Table 2-5 summarizes the approximate levels of structural and nonstructural damage that may be expected at the damage states that define the structural performance levels.

Table 2-5. Approximate Expected Damage for Different Building Performance Levels[2].

	Building Performance Levels			
	Collapse Prevention	**Life Safety**	**Immediate Occupancy**	**Operational**
Overall Damage	Severe	Moderate	Light	Very Light
Permanent Drift	Large. 1% to 5%.	Some. 0.3% to 1%.	Negligible.	Same as Immediate Occupancy.
Remaining Strength and Stiffness after Earthquake	Little. Gravity system (columns and walls) functions, but building is near collapse.	Some. Gravity system functions, but building may be beyond economical repair.	Significant strength remaining. Minor cracking of structural elements.	Same as Immediate Occupancy.
Examples of Damage to Reinforced Masonry Buildings	Extensive cracking and crushing. Damage around openings at corners. Some fallen units. Transient drift to cause extensive nonstructural damage. Extensive permanent drift.	Major cracking distributed throughout wall. Some isolated crushing. Transient drift to cause nonstructural damage. Noticeable permanent drift.	Minor cracking. No out-of-plane offsets. Transient drift that causes minor or no nonstructural damage. Negligible permanent drift.	Same as Immediate Occupancy.
Examples of Damage to Steel Framing	Extensive yielding and buckling of steel bracing members. Significant connection failures.	Many braces and beams yield or buckle but do not fail totally. Moderate amount of connection failures.	Minor deformation of steel members, no connection failures.	Same as Immediate Occupancy.
Other General Description	Structure likely not repairable and not safe for reoccupancy due to potential collapse in aftershock.	Repair may be possible, but may not be economically feasible. Repairs may be required prior to reoccupancy.	Minor repairs may be required, but building is safe to occupy.	Same as Immediate Occupancy.
Nonstructural Components	Extensive damage. Some exits blocked. Infills and unbraced parapets failed or at incipient failure.	Falling hazards mitigated, but many architectural, mechanical, and electrical systems are damaged.	Minor cracking of facades, partitions, and ceilings. Equipment and contents are generally secure, but may not operate due to lack of utilities.	Negligible damage. All systems important to normal operation are functional. Power and other utilities are available, possibly from standby sources.
Comparison with New Building Design	Significantly more damage and greater risk.	Somewhat more damage and slightly higher risk.	Much less damage and lower risk.	Much less damage and lower risk.

[2] Adapted from American Society of Civil Engineers, *"Prestandard and Commentary for the Seismic Rehabilitation of Buildings,"* FEMA-356, Federal Emergency Management Agency, Washington, D.C., November 2000.

2.4 Seismic Performance, Safety, Reliability, and Construction Cost

The seismic performance, safety, and reliability of a facility must be weighed against the relative importance and construction costs associated with a facility. It is impractical for the average building to be seismically designed or retrofitted to experience no damage following a major earthquake. However, steps can be taken to mitigate seismic hazards for new and existing structures.

Some facilities have more community importance or pose special risks to a community following an earthquake, such as hospitals, fire stations, community shelters, or facilities housing highly toxic substances. It is reasonable that important facilities be designed or retrofitted to a higher performance standard than the average structure. The relative importance of a facility must be weighed against the relative construction costs associated with facility construction. There are two types of construction costs associated with seismic hazards: the cost of initial construction or seismic retrofit construction and the costs to repair or replace a facility following an earthquake. The better a structure performs during an earthquake, the faster a structure can be returned to service and the lower the repair costs will be for a structure following an earthquake. So, building expected damage states during a seismic event can be directly linked to:

- Repair/Replacement Costs – Cost of restoring the facility to pre-earthquake condition.
- Public Safety – Number of critical injuries and casualties to building occupants.
- Downtime – Length of time taken to make repairs to return a structure to service.

Figure 2-2 shows estimated performance-related consequences compared with different increasing post-earthquake structural damage states (which correspond to the design Structural Performance Levels for a given seismic hazard).

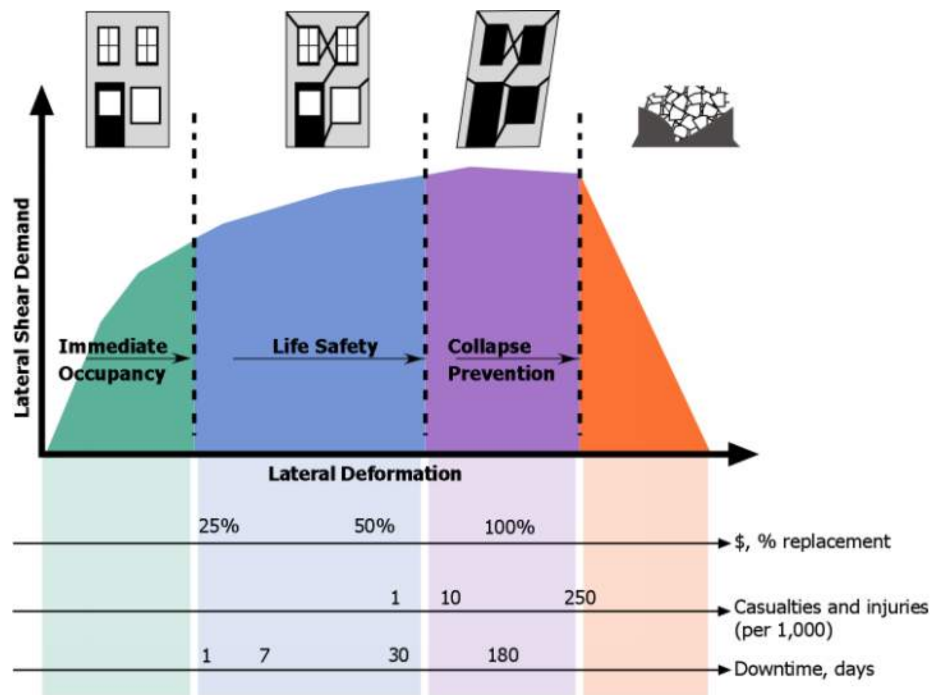

Figure 2-2. Estimated Performance-Related Consequences at Different Structural Performance Levels[3].

Figure 2-3 presents the schematic relationship between different retrofit building performance objectives and probable retrofit program cost.

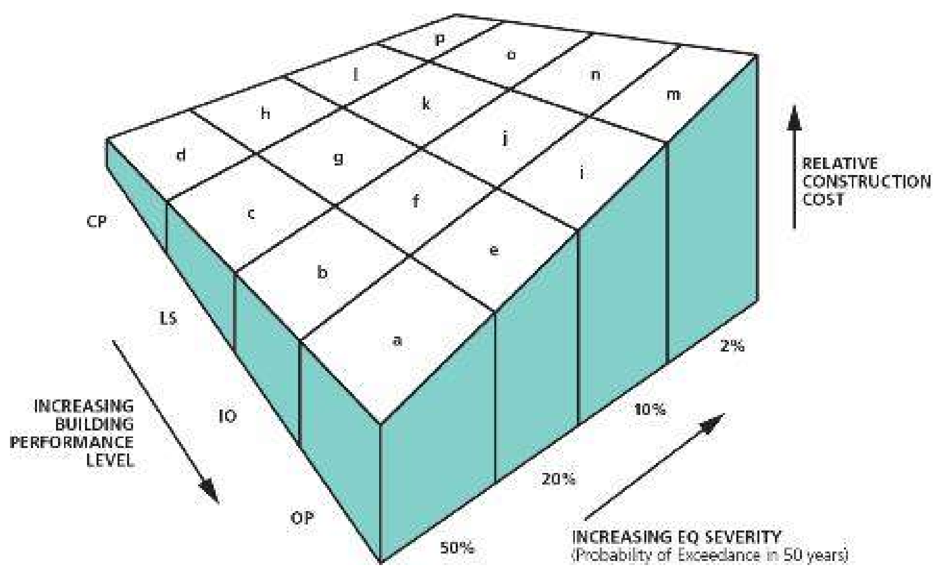

Figure 2-3. Surface Matrix of ASCE 41 Building Performance Levels Compared with Construction Cost[4].

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3 J. Moehle, "A Framework for Performance-Based Earthquake Engineering," *Proceedings from ATC 15-9, 10th US-Japan Workshop on the Improvement of Structural Design and Construction Practices*, Applied Technology Council, Makena, Hawaii, 2003.

4 Adapted from Applied Technology Council, "*NEHRP Commentary for the Seismic Rehabilitation of Buildings*," FEMA-274, Federal Emergency Management Agency, Washington, D.C., October 1997.

2.5 Seismic Performance of Nonstructural Components

Mitigation of nonstructural seismic hazards is a complex issue that is addressed independently in the evaluation and retrofit guidelines. For much of the 20th Century, little attention was given to designing nonstructural components and their anchorage for forces induced by earthquakes. Nonstructural component damage witnessed during earthquakes in recent years has demonstrated the importance of nonstructural component performance during earthquakes for life safety and post-earthquake safety and building function.

In addition to the life safety hazards posed by nonstructural components, the cost to repair nonstructural components following an earthquake can be high. In many cases, the cost to repair or replace nonstructural components can be higher than the cost of repairing structural components following an earthquake. The relative monetary importance of nonstructural components can be seen in Figure 2-4, comparing the relative construction costs of the contents, nonstructural components, and the structural components of three types of typical new buildings. In offices and hotels, the building nonstructural components cost the most to construct, by a significant margin. In hospitals, the costs of constructing the building contents and nonstructural components are similar, but still far exceed the cost of the building structural systems.

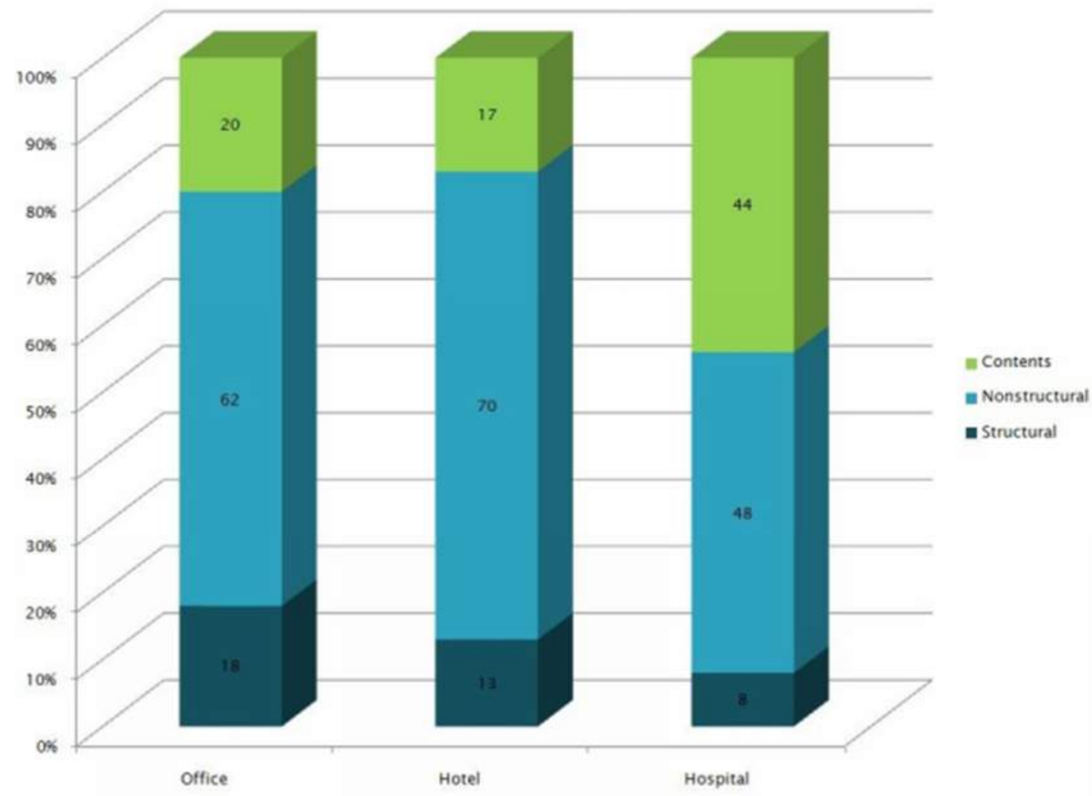


Figure 2-4. Typical Construction Costs for Different Building Components.[5]

Many nonstructural components, if adequately secured to the structure, are seismically rugged. However, mitigation of some nonstructural hazards (such as bracing for mechanical and

[5] Federal Emergency Management Agency, "*Reducing the Risks of Nonstructural Earthquake Damage – A Practical Guide*," FEMA E-74, Federal Emergency Management Agency, Washington, D.C., December 2012.

electrical components within suspended ceiling systems or the improvement of ceiling systems themselves) can result in extensive disruption of occupancy. Repairing or replacing these components following an earthquake can also be very costly. These costs and benefits need to be taken into consideration when determining desired nonstructural performance levels and the goals of any seismic evaluation or retrofit.

Finally, the use of the structure and the required level of building performance needs to be taken into consideration. For example, essential facilities that are expected to have minimal structural damage following the design earthquake must have nonstructural components that are designed to match the seismic performance level of the facility.

2.6 Seismic Evaluation Procedure

ASCE 41-17 provides a three-tiered evaluation procedure using performance-based criteria. The process for seismic evaluation is depicted in Figure 2-5. The evaluation process consists of the following three tiers: Screening Phase (Tier 1), Evaluation Phase (Tier 2), and Detailed Evaluation Phase (Tier 3). A summary of each phase follows.

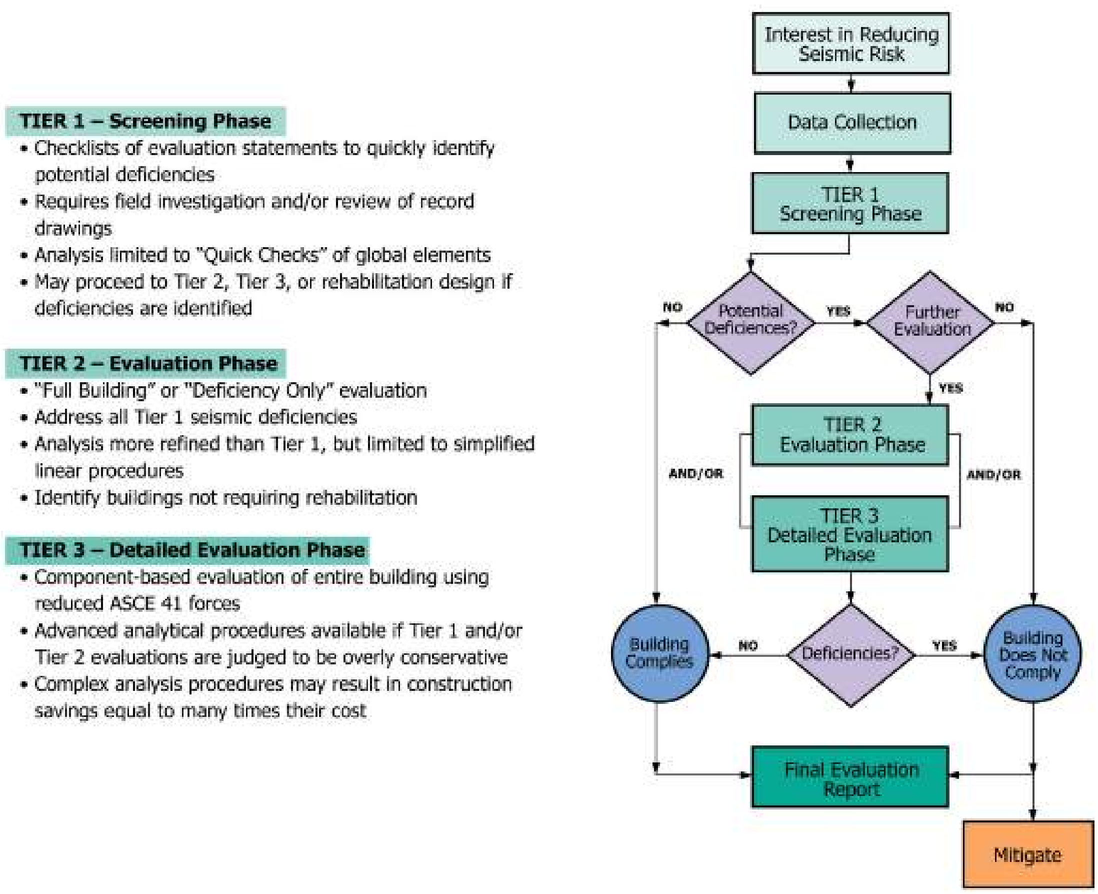


Figure 2-5. Flow Chart and Description of ASCE 41-17 Seismic Evaluation Procedure.

The Tier 1 checklists in ASCE 41-17 are specific to each common building type and contain seismic evaluation statements based on observed structural damage in past earthquakes. These checklists screen for potential seismic deficiencies by examining the lateral-force-resisting systems (LFRS) and details of construction that have historically caused poor seismic performance in similar buildings. Tier 1 screenings include basic "Quick Check" analyses for primary components of the lateral system. They also include prescriptive checks for proper seismic detailing of connections, diaphragm spans and continuity, and overall system configuration. Tier 2 evaluations then follow with additional calculations and assessments to either confirm the potential deficiencies identified in the Tier 1 review or demonstrate their adequacy. A Tier 3 evaluation involves an even more detailed analysis and advanced computations to review each structural component's seismic demand and capacity. A Tier 3 evaluation is similar in scope and complexity to the types of analyses often required to design a new building in accordance with the IBC, with a comprehensive analysis aimed at evaluating each component's seismic performance. As indicated in the Scope of Services, these evaluations include a Tier 1 and 2 screening.

2.7 Seismic Retrofit/Upgrades Procedure

If seismic deficiencies are identified in the evaluation process, the owner and design team should review all initial conditions before proceeding with the hazard mitigation. Many conditions may affect the retrofit design significantly, such as results of the seismic evaluation and seismic hazard study, building use and occupancy requirements, presence of hazardous materials, and other anticipated building remodeling. The basic process for performance-based retrofit design is illustrated in Figure 2-6.

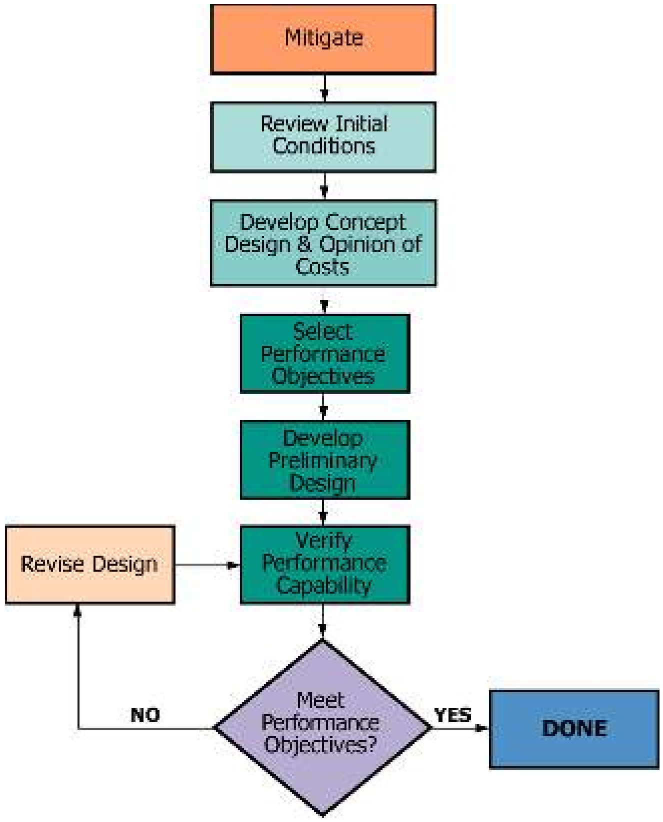


Figure 2-6. Seismic Rehabilitation Flow Diagram.

Following the review of initial conditions, concept designs may be performed in order to develop rough opinions of probable construction costs for one or more performance objectives. The owner and design team can then develop a rehabilitation strategy considering the associated costs and feasibility. Schematic and final design can then proceed through an iterative process until verification of acceptable building performance is obtained.

LIMITATIONS

The professional services described in this report were performed based on available as-built information and limited observation of the structure. No destructive testing was performed to qualify as-built conditions or verify the quality of materials and workmanship. No other warranty is made as to the professional advice included in this report. This report provides an overview of the seismic evaluation results and proposed upgrades and does not address programming and planning issues.

This report has been prepared for the exclusive use of The City of Tukwila. It is not intended for use by other parties, nor may it contain sufficient information for purposes of other parties or their uses. This report does not address any portion of the structure other than those areas mentioned, nor does it provide any warranty, either expressed or implied, for any portion of the existing structure.

3.0 Seismic Evaluation

3.1 City Hall

3.1.1 Building Description

Year Built:	1977
Number of Stories:	2
Floor Area:	27,000 SF

The City Hall is a two-story, concrete- and wood-framed structure located in the central area of Tukwila. The building is approximately 195 feet by 128 feet in plan, 37 feet tall, and has an L-shaped footprint with distinctive saw-tooth wall lines on the southeastern elevations of the structure. The main roof is stepped in 24-foot-wide sections that align with the saw-tooth wall lines and slope monolithically from northwest to southeast. The upper story is wood-framed construction with structural-panel walls and long-span timber roof trusses. The lower story construction consists of concrete walls and columns, steel posts, and wood-framed walls supporting the level above. The building is located on a site that slopes downhill from north to south. The first story is below grade on the north side and portions of the east and west sides. A partial basement level is located below the first story in the southwest corner.

Figure 3.1-1. City Hall, Northwest Corner.

Figure 3.1-2. City Hall, West End (looking south).

3.1.2 Structural System

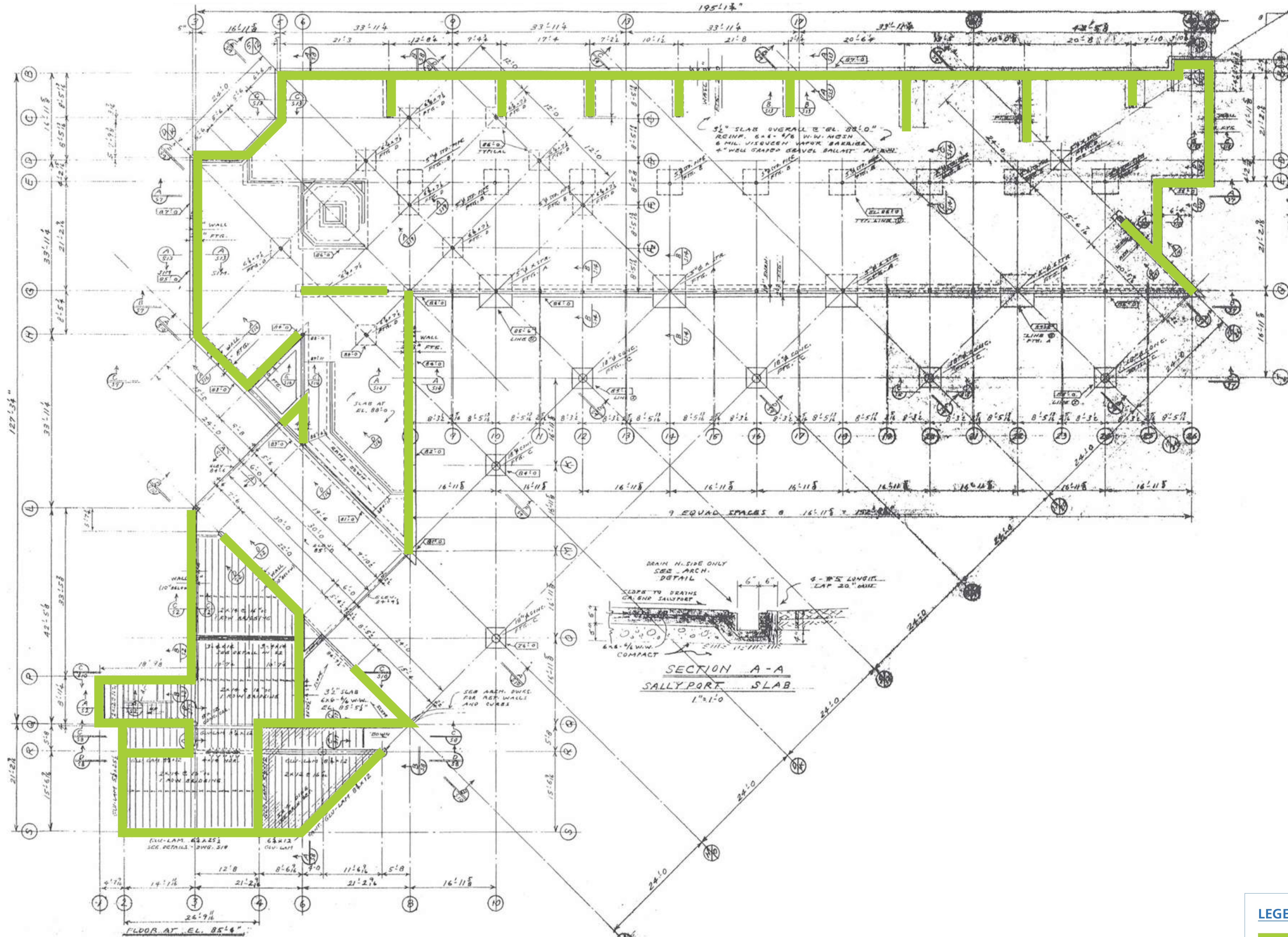
The City Hall building houses administrative departments. The partial basement level is vacant and used for storage. The building's gravity and lateral systems are summarized in Table 3.1-1 and shown graphically in Figures 3.1-3 through 3.1-5.

Table 3.1-1. Structural System Description of City Hall.

Structural System	Description
Roof	Glulam beams support plywood sheathing on 2x wood roof joists. Long-span timber trusses and wood stud walls support the roof framing.
Floor	Tongue-and-groove plywood sheathing with 1½-inch concrete topping over wood joists supported by a combination of glulam beams, concrete walls, wood stud walls, and steel wide-flange beams.
Foundations	Concrete walls on continuous concrete footings. Concrete retaining walls are present at the first story along the north and west sides of the building. Steel posts and concrete columns bear on concrete spread footings. First-floor construction is a 3½-inch concrete slab-on-grade lightly reinforced with 6x6 welded-wire fabric, except at the south corner of the building, which is a wood floor system similar to the second floor. A partial basement level is located in the south corner below the first floor; construction consists of concrete walls and slab-on-grade.
Gravity System	The second story generally consists of roof framing spanning to the exterior walls via wood trusses. Roof diaphragm and trusses are supported by wood stud walls on concrete foundation walls and glulam beams in the second-floor framing. The second floor is supported on perimeter and interior concrete walls, wood stud walls, steel posts, and concrete posts on concrete spread and continuous foundations.
Lateral, 2nd Story	Wood structural-panel shear walls resist lateral loads at the second story. The distribution of the shear walls is non-symmetrical and unbalanced. The building has a single 34-foot-long shear wall parallel to each orthogonal building dimension in the northwest corner of the building. Additional 18-foot-long shear walls are located between and oriented with the sloped sections of the stepped main roof.
Lateral, 1st Story	Reinforced concrete shear walls resist the lateral loads at the first story. The concrete shear walls are primarily oriented in the orthogonal building directions, with some walls, mostly at the south end of the building, oriented at a 45-degree angle to the principal building directions.

3.1.2.1 ASCE 41 Classified Building Type

Use of ASCE 41 for seismic evaluations requires buildings to be classified from a group of common building types historically defined in previous seismic evaluation standards (ATC-14, FEMA 310, and ASCE 31-03). The building is classified in ASCE 41, Table 3-1, as two building types: a Wood Light Frame structure, **W2,** and a Concrete Shear Wall Building with Flexible Diaphragms, **C2a**. These building types include those buildings that have bearing walls constructed of reinforced concrete and wood, with elevated floor and roof framing structural systems consisting of wood or other flexible diaphragms.



LEGEND

Concrete Shear Wall

City Hall — Existing Lateral System - Foundation & First Floor

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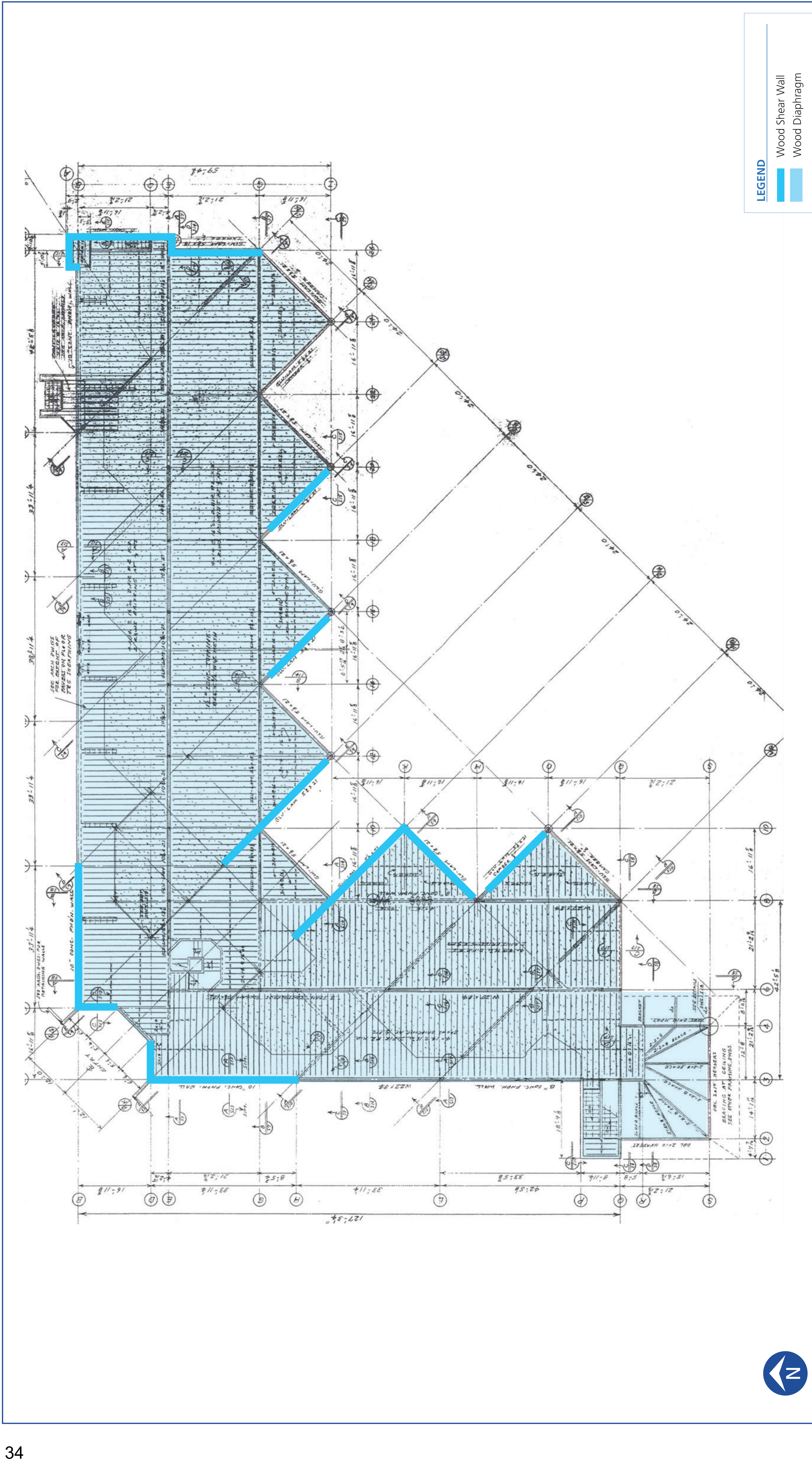
Figure 3.1-3



City Hall — Existing Lateral System - Second Floor
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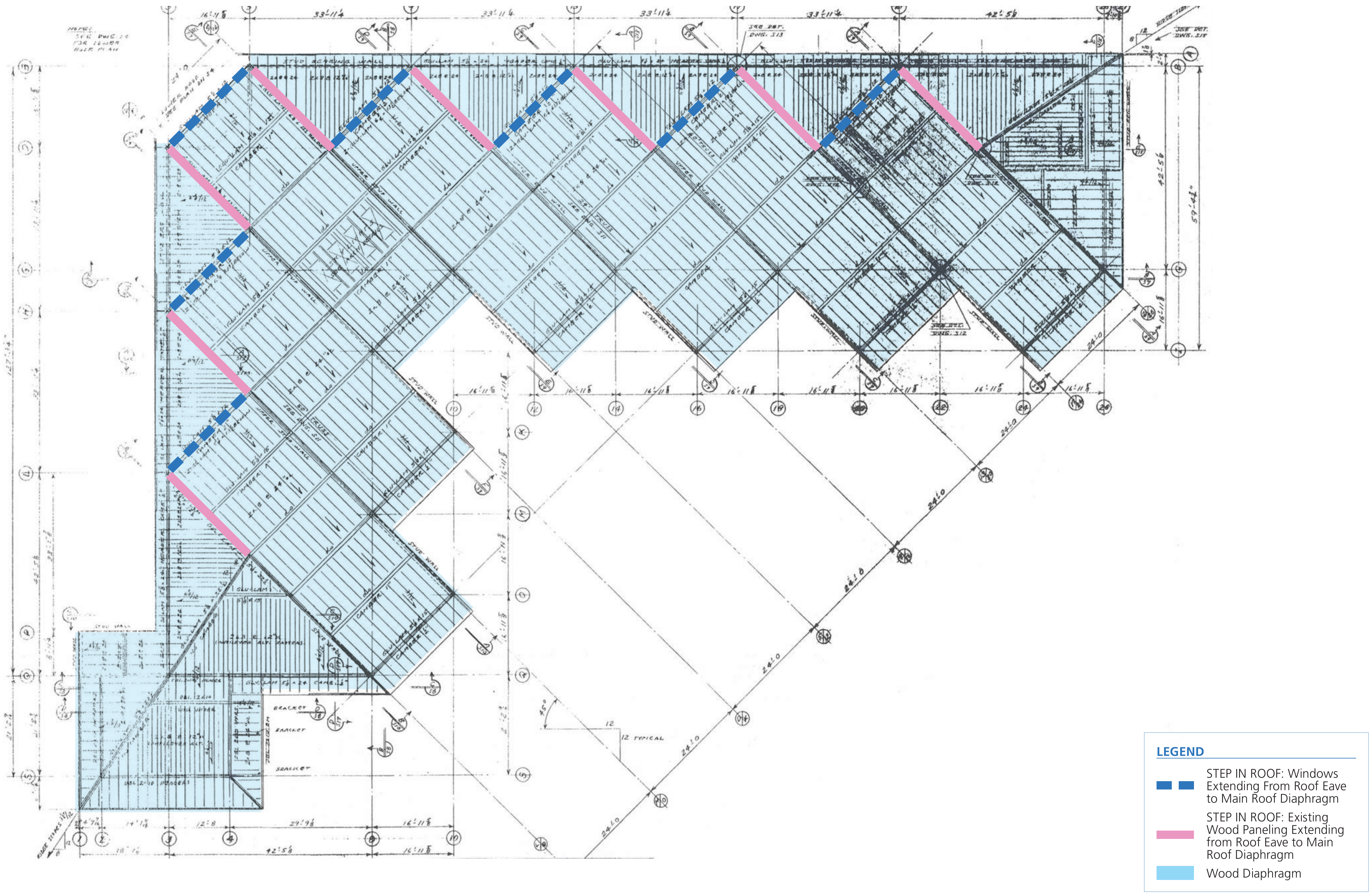
Figure 3.1-4



City Hall — Existing Lateral System - Roof

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Figure 3.1-5

3.1.3 Seismic Evaluation and Findings

3.1.3.1 Seismic Deficiencies

This section of the report describes the results of the ASCE 41-17 Tier 1 and Tier 2 deficiency-based evaluations. Deficiencies identified by the Tier 1 checklist are further evaluated by the Tier 2 evaluation procedure, and preliminary structural upgrade recommendations are provided.

Based on the results of the Tier 1 checklist and Tier 2 analysis, the City Hall Building in its current condition does not meet the Life Safety and Collapse Prevention performance objectives for the design-level earthquake. This is not unusual for buildings of similar construction type and vintage. However, the building is in good condition overall.

Table 3.1-2 summarizes the seismic deficiencies in the structural systems identified by the Tier 1 Structural Checklist. The full Tier 1 screening checklists and supporting calculations are provided in Appendix A.

Table 3.1-2. Identified Seismic Deficiencies for City Hall.

Deficiency	Description
Vertical Irregularities	The LFRS is largely non-symmetrical between Grids E and I. The building lacks walls between Grids G and I, causing a vertically discontinuous load path between the second-story LFRS walls and the foundation.
Geometry	The second-story LFRS does not include a complete orthogonal shear wall system in both principal directions. The northern portion of the building between Grids 9 and 27 lacks shear walls oriented in the north-south direction. The western portion of the building between Grids G and Q lacks shear walls oriented in the east-west direction. Lateral forces are primarily resisted in both cases by the diagonal walls. The resistance of lateral loading by the diagonal walls results in amplified forces.
Walls Connected Through Floors	The building lacks adequate seismic straps between the wood framing in the second-story floor diaphragm. The wood framing acts as seismic drag struts to transfer lateral forces from the diaphragm to the first-story concrete shear walls.
Concrete Walls Shear Stress Check	The shear stress calculated using all concrete shear walls exceeds the 100-psi quick check value.
Slope Failure	The building is located on a sloped site. Earthquake induced slope failures could cause instabilities in the building foundation, which would cause structural failures across the entire building.
Ties Between Foundation Elements	Foundation consists of isolated spread footings with no ties between them. Site soils are unknown but are typically identified as Site Class D soils, which do not meet performance objective requirements.
Wall Anchorage	The concrete shear walls have a wood ledger anchored on top of the wall, but the building lacks adequate connections from the diaphragm above to provide out-of-plane-support.
Transfer to Shear Walls	The building lacks adequate connections to transfer lateral forces between the second-story floor diaphragm and the first-story concrete shear walls.

The Tier 1 checklist is used to identify common deficiencies for a given building framing type. However, the checklist is only a rough evaluation technique, and a more-in-depth Tier 2 analysis is required to confirm if deficiencies require structural upgrades. Detailed information on the Tier 2 analysis and calculations is provided in Appendix A.

3.1.3.2 Demand-Capacity Ratios

Table 3.1-3 summarizes the results of the Tier 2 analysis in terms of Demand-to-Capacity Ratios (DCRs). The DCR is determined by the load on the structural member divided by the member capacity. A DCR value greater than 1.0 indicates that the member is inadequate.

The maximum DCR is an envelope value considering all the shear wall segments within a shear wall at that given level. The maximum DCR is provided for both evaluation criteria: BSE-1E LS and BSE-2E CP. Appendix A provides a breakdown of the results for each evaluation criteria.

Table 3.1-3. Tier 2 Analysis, Non-Compliant Item Demand-to-Capacity Ratios.

Component	BSE-2E, LS Max DCR	BSE-2E, CP Max DCR
Concrete Shear Walls	0.343	0.971

Legend:	DCR Less Than 1.0, Adequate	DCR Greater Than 1.0, Inadequate

Based on the results shown in Table 3.1-3, components of the lateral system have adequate DCRs. However, due to the deficiencies associated with a lack of connection and complete load path, the City Hall Building does not meet the Life Safety performance objective. This analysis also confirms that the City Hall Building does not meet the Collapse Prevention performance objective.

3.1.3.3 Recommendations

The City Hall has multiple structural deficiencies in the LFRS, primarily associated with incomplete load paths. Diaphragm anchors to concrete shear walls are inadequate for the LS and CP performance levels. The building is susceptible to unacceptable levels of damage and poor performance of the LFRS during a design-level earthquake. Poor performance of the building increases the risk to the building's occupants and limits the building's ability to remain operational following a seismic event. However, the structural condition of the building is generally satisfactory and is adequate to facilitate functions performed in the building.

The building includes an adequate gravity system, and portions of the LFRS satisfy the target seismic performance criteria. Most deficiencies identified in the Tier 2 evaluation may be mitigated by strengthening and adding additional elements to the existing LFRS and providing positive connections between elements of the LFRS to complete seismic load paths. Consequently, a structural retrofit is recommended to address structural deficiencies and improve the seismic performance of the City Hall to achieve the desired performance levels and post-earthquake operational objectives.

3.1.3.4 Structural Retrofit Concept Design

Figures 3.1-6 through 3.1-8 display schematic-level retrofit concepts to improve the LFRS and meet the LS performance objective. This concept-level seismic upgrade discussion represents just one of several alternative seismic upgrade design solutions and is based on preliminary seismic evaluation and analysis results. Final analysis and design for seismic upgrades must include an architectural layout, defined occupancy class, and consideration for upgraded mechanical and electrical systems.

The retrofit approach at the second story involves strengthening existing wood shear walls by adding structural panels and improving nailing to increase wall capacities. New shear walls are also needed to increase the overall capacity and improve the symmetry of the LFRS. Hold-downs should be installed to provide resistance to wall overturning forces. Steel bracing or other similar elements are recommended on the north and west walls to transfer forces from the high roof to the low roof and shear walls.

The vertical elements and foundations at the first story below the diagonal shear walls may require retrofit. Strengthening the posts and columns may be necessary to resist overturning forces from the shear walls above. Steel braces should be installed below the second floor along the saw-tooth wall lines to support overturning loads from the discontinuous shear walls at the second story. Foundation modifications involve expanding the spread footings to reduce bearing pressures and resist uplift forces.

The first-story LFRS retrofits include adding new wood shear walls in the northeast direction at the northwest corner of the building and modifying the north-south shear walls along Grid B to improve the load-resisting capacity of the system. Modifications to the foundation systems may be required for both the new and existing walls. Seismic straps should be installed between the second-floor wood framing members acting as seismic collectors to transfer loads to the shear walls. Connections between the wood framing and shear walls must also be improved using post-installed anchorages or other techniques. The addition of seismic straps and framing-to-wall connections is required for both new and existing walls.

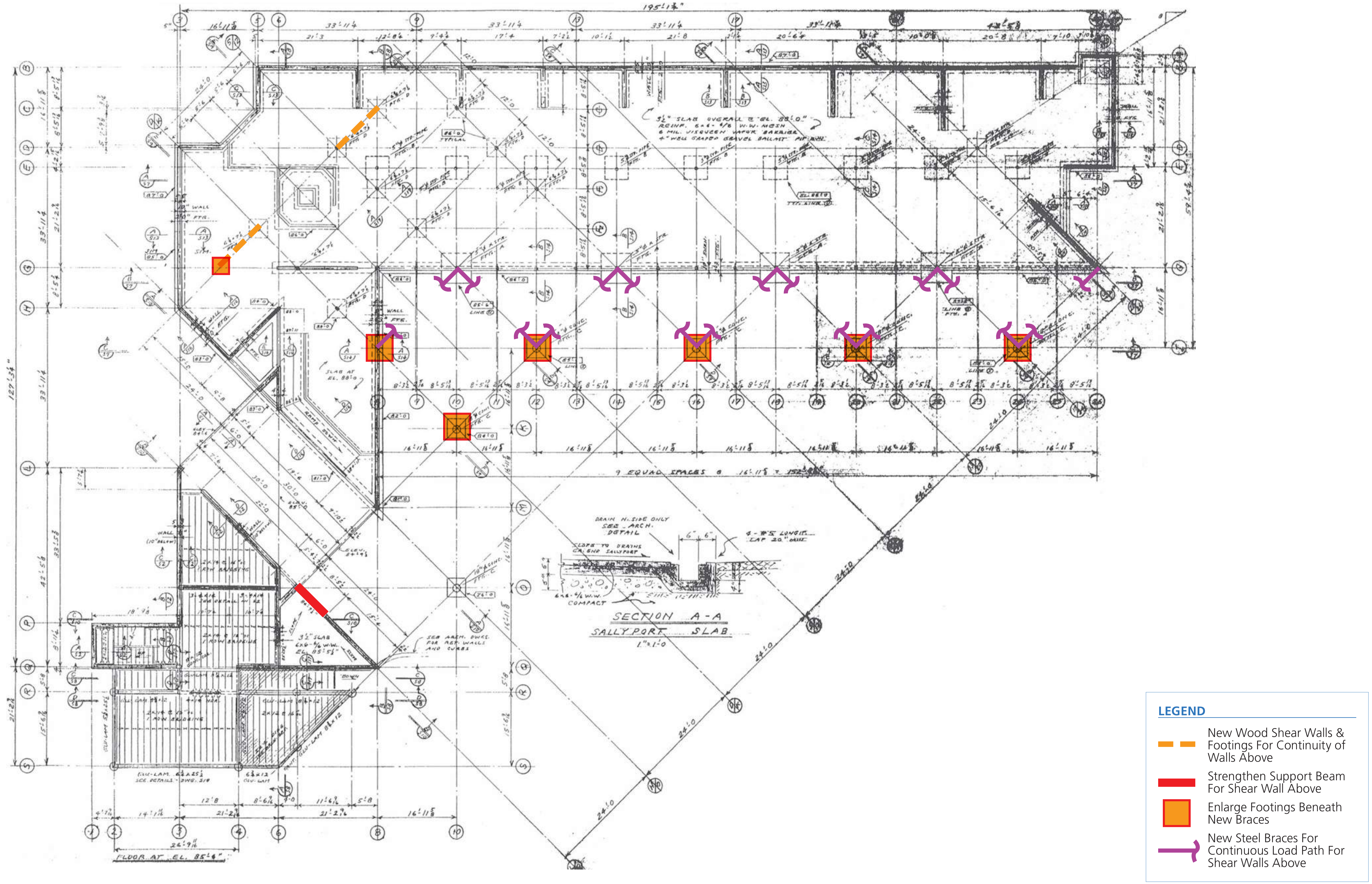
A reduced structural retrofit can be performed to meet the lower CP performance objective. However, since the majority of the deficiencies associated with the building are a lack of connections and complete load path, most upgrade requirements in the LS performance objective schematic-level retrofit concepts are still required. Reduced retrofit concepts include reductions to the amount of new wood shear walls at the first floor and reduction of bracing and wall upgrades at the partial basement level, directly below the first-floor walls.

3.1.3.5 Probable Construction Costs and Other Considerations

The probable construction cost to perform the recommended structural seismic upgrades to meet the Life Safety and Collapse Prevention performance objectives is $4.57M and $4.46M, respectively. The estimates provided in Appendix B include an escalation table showing escalation for 1 year, 2 years, and out to 5 years. The costs include labor, materials, equipment, and general contractor general conditions (mobilization), overhead, and profit. Additional geotechnical study and evaluation of the building subgrade are excluded from the construction probable cost estimates. The estimates assume the building is unoccupied and phasing is not required.

These estimates are based on the mechanical, electrical, plumbing, and fire protection (M/E/P/FP) systems being modified to accommodate seismic work, but M/E/P/FP systems are not upgraded to the latest building codes for these systems. According to the International Existing Building Code (IEBC), a voluntary seismic upgrade is considered a Level 2 alteration. A Level 2 alteration does not require upgrades to all building components. However, a Level 3 alteration, which requires the building to be brought up to full compliance with current codes, can be triggered if the work area exceeds 50 percent of the total floor area. To avoid placing the voluntary seismic upgrade work into a Level 3 alteration, care would need to be taken to define the work area to only the actual floor areas occupied by the upgraded components.

It is recommended that operational limitations, historical or architectural factors, nonstructural components, and key systems in the building also be evaluated for their useful life or use issues. These include but are not limited to accessibility, emergency power, fire alarm and sprinklers, energy-efficient lighting, energy-efficient plumbing fixtures, HVAC modifications, exterior soffits/siding/windows, foundation drainage, intercom/paging/security cameras, and interior finishes/systems furniture.



City Hall — Foundation Retrofit Concept

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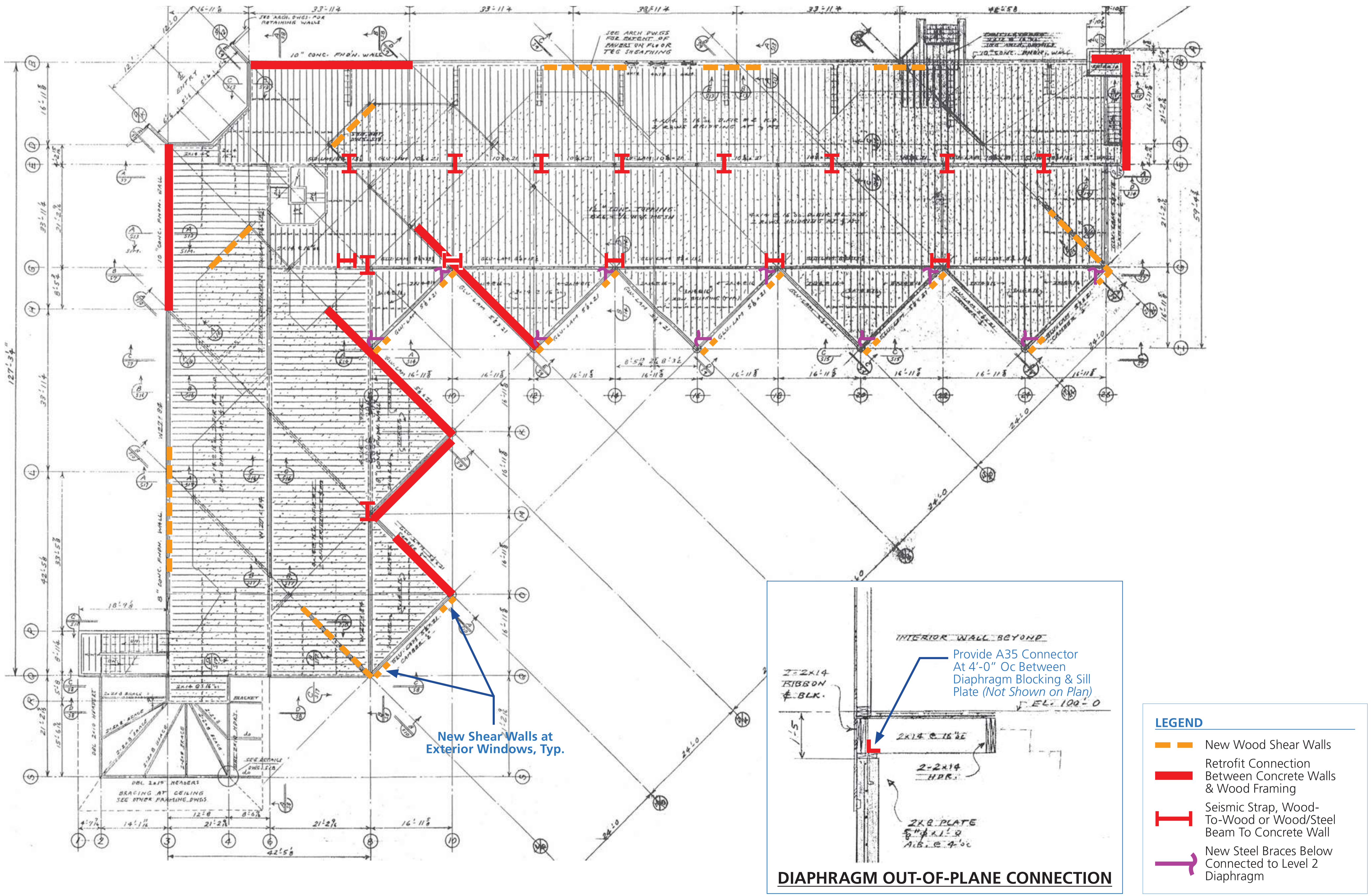
Figure 3.1-6



City Hall — First Floor Retrofit Concept

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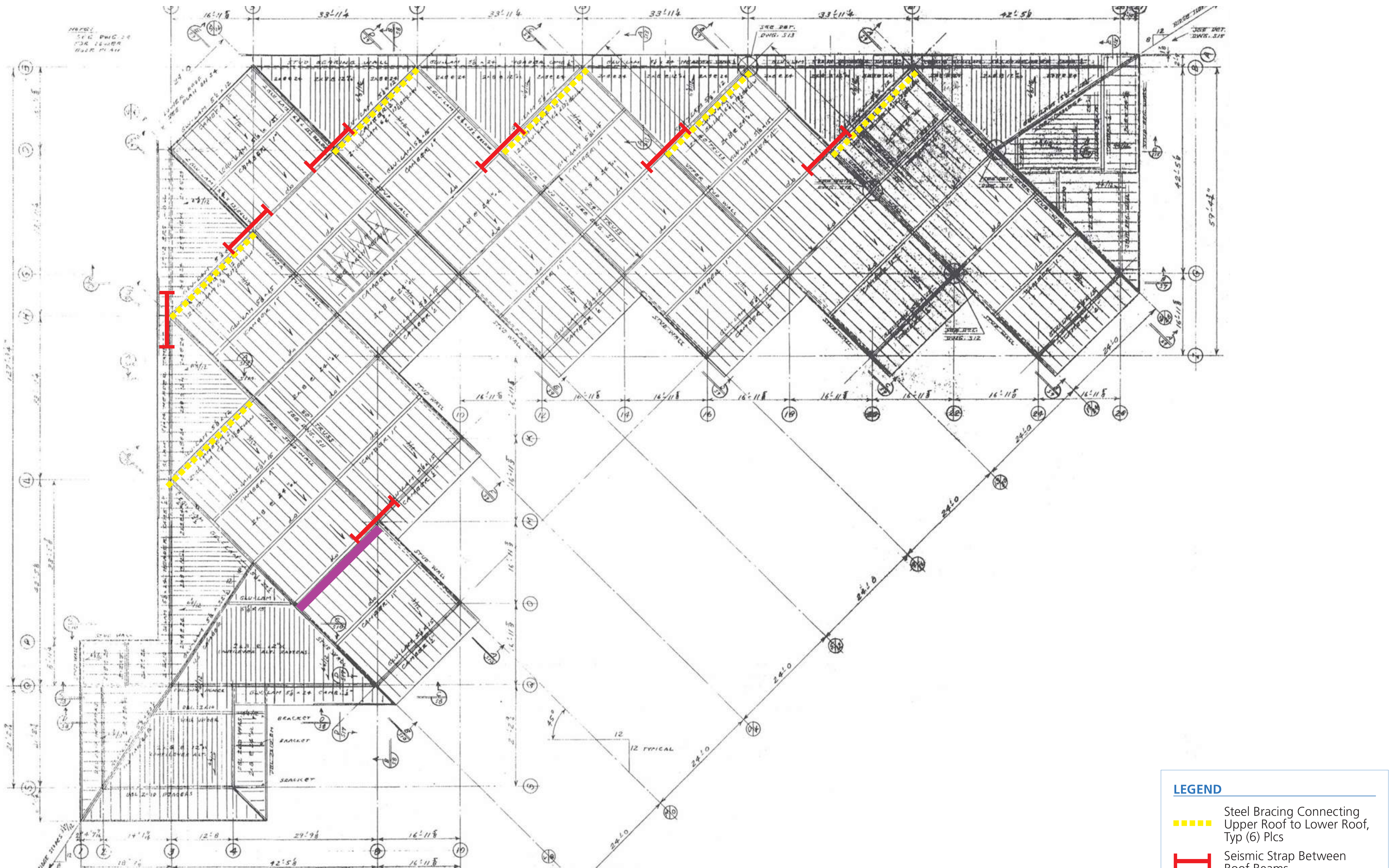
Figure 3.1-7



City Hall — Roof Retrofit Concept

City of Tukwila Multi-Building Seismic Assessments Update - June 2022

Figure 3.1-8

3.2 6300 Building

3.2.1 Building Description

Year Built:	1978
Number of Stories:	3
Floor Area:	33,600 SF

The 6300 Building is a three-story concrete- and wood-framed structure located in the central area of Tukwila, adjacent to City Hall. The rectangular building is 80 feet by 210 feet in plan and 43 feet tall. The first and second stories are primarily wood-framed construction with structural-panel walls and diaphragms. The building has a parking level below the first story. Construction of the parking level consists of concrete walls and columns supporting the levels above. The building is located on a site that slopes downhill from north to south. The north end of the parking level is below grade. Concrete walls in the northern half of the building also act as retaining walls.

Figure 3.2-1. 6300 Building, West Exterior.

Figure 3.2-2. 6300 Building, Parking Level.

3.2.2 Structural System

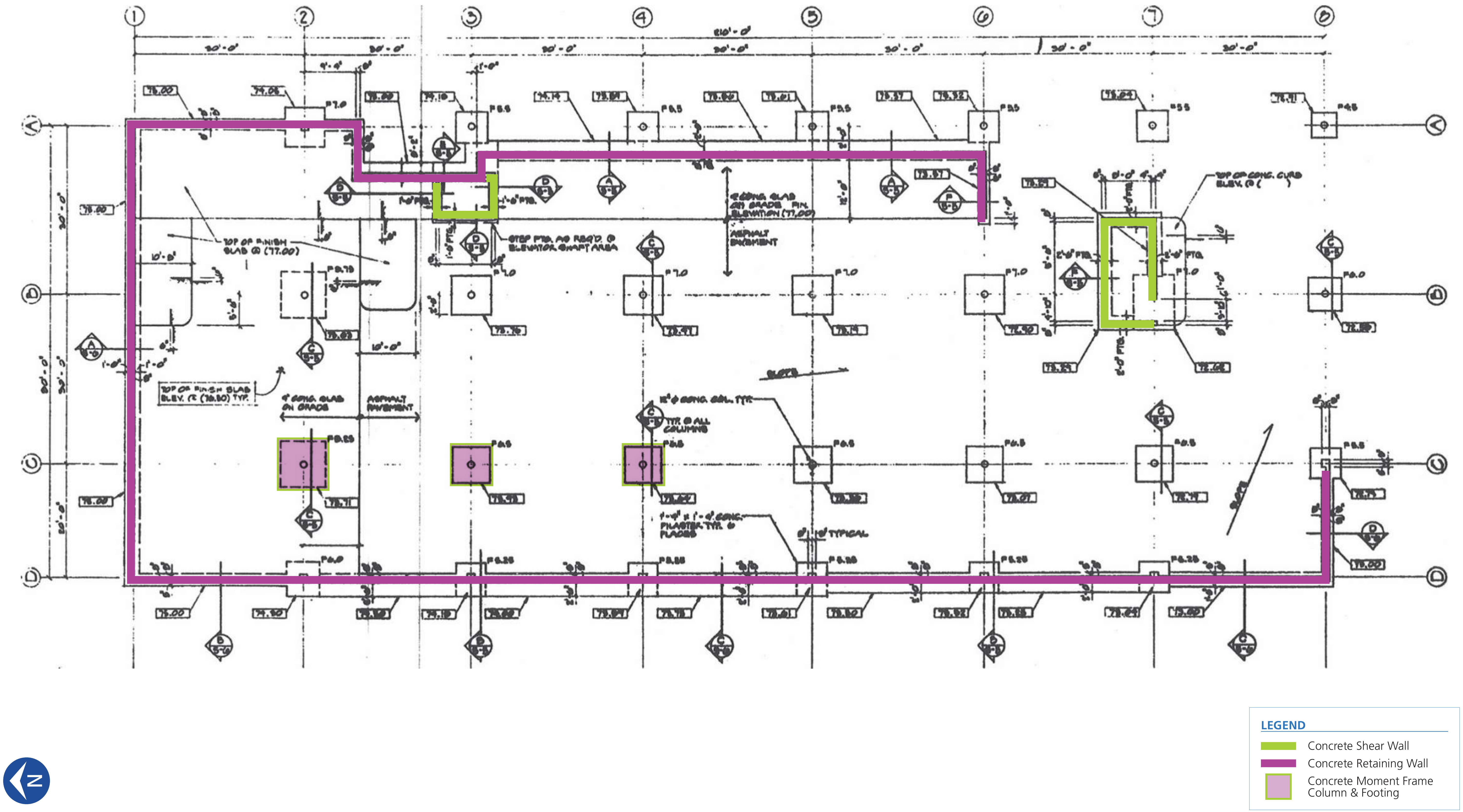
The 6300 Building houses a variety of city departments, including but not limited to community development, human services, human resources, permitting, and technology. The building's gravity and lateral systems are summarized in Table 3.2-1 and shown graphically in Figures 3.2-3 through 3.2-5.

Table 3.2-1. Structural System Description of 6300 Building.

Structural System	Description
Roof	Glulam beams support the plywood roof sheathing on roof open-web truss joists. Wood beams, stud walls, steel columns provide gravity support for the roof.
Floor	Glulam beams support the 1½-inch lightweight concrete-topped plywood floor sheathing on TJL floor joists on both the first and second floor. The floor system on the north side of the building consists of precast concrete span deck with 2-inch lightweight concrete topping.
Foundations	Concrete bearing walls are supported by continuous concrete footings. Concrete columns located within the interior of the building have isolated spread footings.
Lateral System	Concrete and wood shear walls resist lateral loads in both the transverse and longitudinal directions

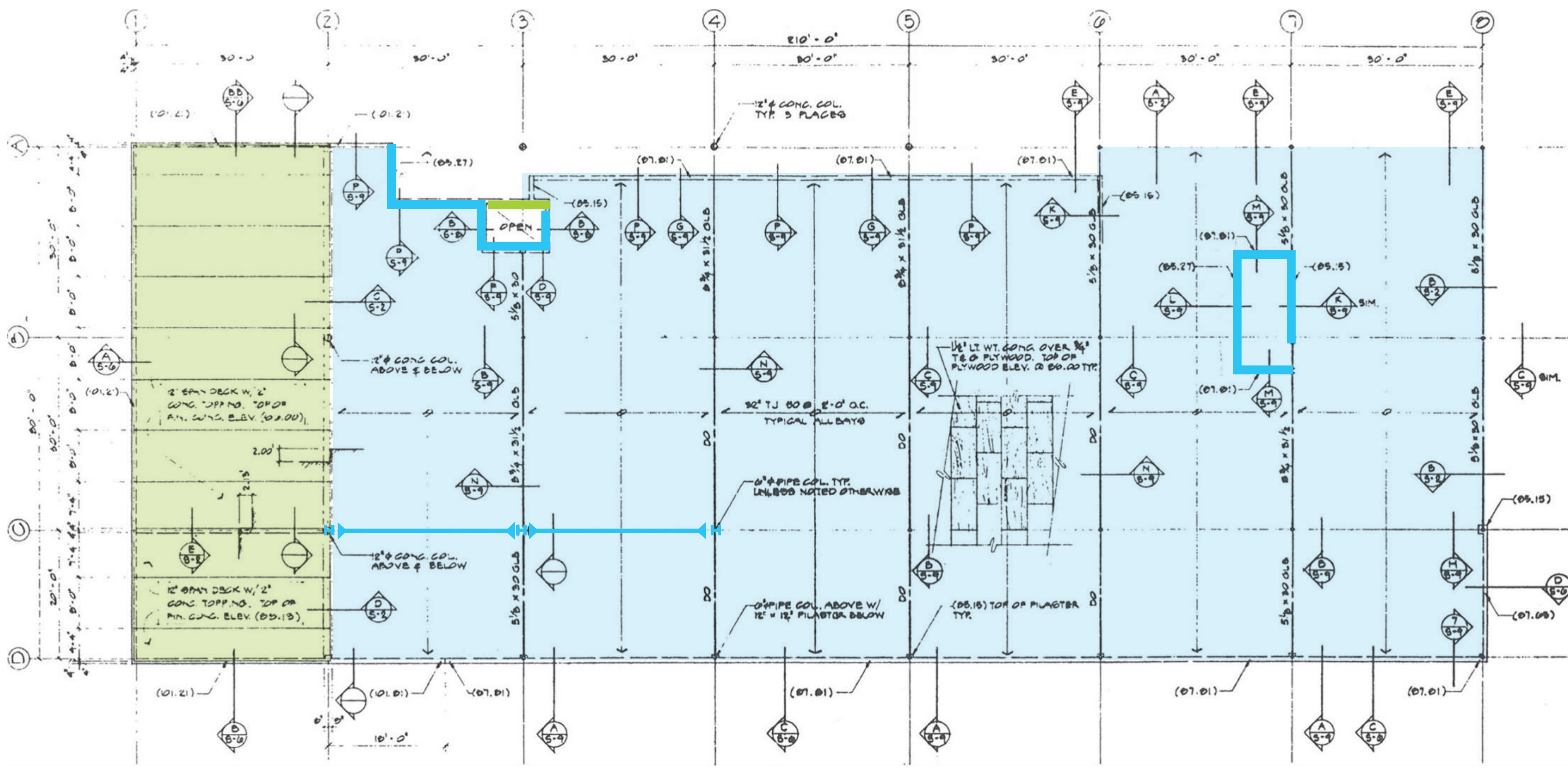
3.2.2.1 ASCE 41 Classified Building Type

Use of ASCE 41 for seismic evaluations requires buildings to be classified from a group of common building types historically defined in previous seismic evaluation standards (ATC-14, FEMA 310, and ASCE 31-03). The building is classified in ASCE 41, Table 3-1, as several different building types: a Wood Light Frame structure, **W2**; a Concrete Shear Wall Building with Flexible Diaphragms, **C2a**; and a Steel Moment Frame Building with Flexible Diaphragms, **S1a**. These building types include those buildings that have bearing walls constructed of reinforced concrete and wood, elevated floor and roof framing structural systems consisting of wood or other flexible diaphragms, and steel moment frames.



6300 Building — Existing Lateral System - Foundation
City of Tukwila Multi-Building Seismic Assessments Update - June 2022

Figure 3.2-3



LEGEND

- Wood Shear Wall
- 8" Concrete Shear Wall
- Steel Moment Frame
- Concrete Panel Diaphragm
- Wood Diaphragm

6300 Building — Existing Lateral System - First Floor

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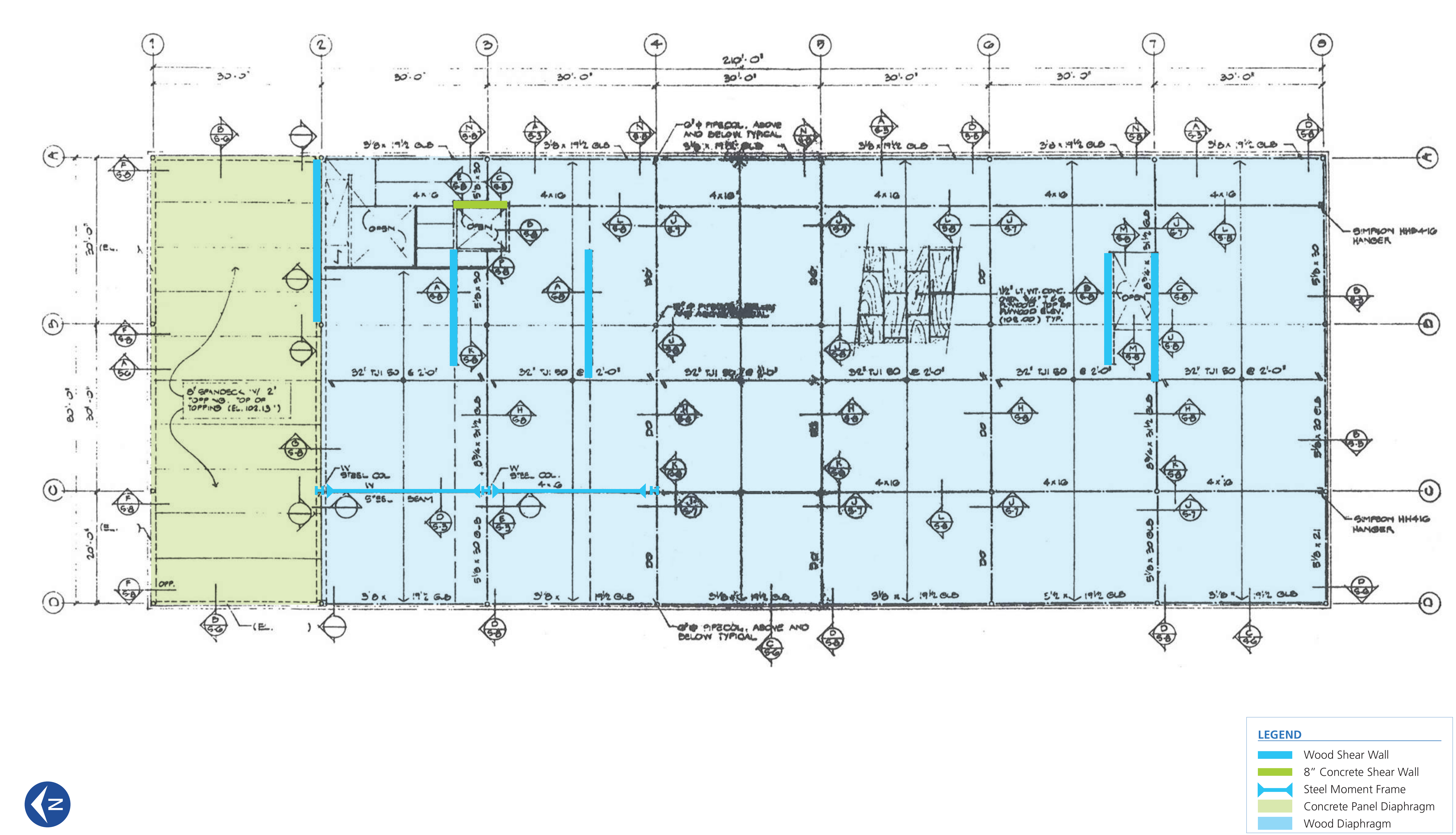
Figure 3.2-4



6300 Building — Existing Lateral System - Second Floor

City of Tukwila Multi-Building Seismic Assessments Update - June 2022

Figure 3.2-5

3.2.3 Seismic Evaluation Findings

3.2.3.1 Seismic Deficiencies

The seismic deficiencies identified during the Tier 2 detailed evaluation phase are summarized below. Commentary for each deficiency is provided based on the detailed seismic evaluation.

Table 3.2-2. Identified Seismic Deficiencies for 6300 Building.

Deficiency	Description
Slope Failure	The building is located on a sloped site. Earthquake-induced slope failures could cause instabilities in the building foundation, which would cause structural failures across the entire building.
Overstressed Wood Shear Walls	The wood shear walls located in both the transverse and longitudinal directions in the upper floors have shear DCRs > 2 and do not have adequate hold-downs. The lack of adequate hold-downs may lead to rocking of the wall, allowing excessive deflections.
Foundation Dowels	Foundation dowels do not match size or spacing of wall reinforcing. Inadequate reinforcing between the main LFRS and the foundations could cause structural failures or poor performance of the foundation and thus the entire building.
Deflection Compatibility	Columns, which act as secondary LFRS components to the concrete shear walls, do not have the shear capacity to develop their flexural capacity.
Redundancy	There is only a single line of a 2-bay moment frame in the north/south direction of the building. However, there is a single 8-inch concrete shear wall (inset from eastern interior near GL 3).
Column Axial Stress	The moment frame columns do not have adequate capacity to resist seismic forces in conjunction with gravity loads.
Frame Flexural Stress	The moment frame elements do not have adequate capacity to resist seismic forces.
Strong Column Weak Beam	The moment frame beams and columns are the same size, and as such do not satisfy strong column weak beam requirements.

3.2.3.2 Demand-Capacity Ratios

Table 3.2-3 summarizes the results of the Tier 2 analysis in terms of Demand-to-Capacity Ratios (DCRs). The DCR is determined by the load on the structural member divided by the member capacity. A DCR value greater than 1.0 indicates that the member is inadequate.

The maximum DCR is an envelope value considering all the shear wall segments within a shear wall at that given level. The maximum DCR is provided for both evaluation criteria: BSE-1E LS and BSE-2E CP. Appendix A provides a breakdown of the results for each evaluation criteria.

Table 3.2-3. Tier 2 Analysis, Non-Compliant Item Demand-to-Capacity Ratios.

Component	BSE-2E, LS Max DCR	BSE-2E, CP Max DCR
Overturning	0.457	0.457
Foundation Dowels	1.954	3.178
Deflection Compatibility	0.007	0.007
Wood Shear Walls	3.65	3.08
Column Axial Stress	12.54	12.54
Moment Frame Flexural Stress, Redundancy, Strong Column-Weak Beam	11.38	11.38

Legend: DCR Less Than 1.0, Adequate | DCR Greater Than 1.0, Inadequate

Based on the results shown in Table 3.2-3, the 6300 Building does not meet the Life Safety performance objective. This analysis also confirms that the 6300 Building does not meet the Collapse Prevention performance objective.

3.2.3.3 Recommendations

The 6300 Building has multiple structural deficiencies in the LFRS, including overstressed shear walls, inadequate foundation dowels, a lack of redundancy in the structural system, and moment frame stresses. The 6300 Building does not currently meet the LS or CP performance objectives. The building is susceptible to unacceptable levels of damage and poor performance of the LFRS during a design-level earthquake. Poor performance of the building increases the risk to the building's occupants and limits the building's ability to remain operational following a seismic event. However, the structural condition of the building is generally satisfactory and is adequate to facilitate functions performed in the building.

The building includes an adequate gravity system and portions of the LFRS satisfy the target seismic performance criteria. Many of the deficiencies identified in the Tier 2 evaluation may be mitigated by adding more wood shear walls and steel moment frames. Consequently, a structural retrofit is recommended to address structural deficiencies and improve the seismic performance of the 6300 Building to achieve the desired performance levels and post-earthquake operational objectives.

3.2.3.4 Structural Retrofit Concept Design

Figures 3.2-6 through 3.2-9 display schematic-level retrofit concepts to improve the LFRS and meet the LS and CP performance objectives. This concept-level seismic upgrade discussion represents just one of several alternative seismic upgrade design solutions and is based on preliminary seismic evaluation and analysis results. Final analysis and design for seismic upgrades must include an architectural layout, defined occupancy class, and consideration for upgraded mechanical and electrical systems.

In both the longitudinal and transverse directions of the building, new lateral-force-resisting elements are being added to strengthen the building. The added elements also act to reduce the diaphragm demands by shortening the diaphragm spans. To limit disruption to the parking area at the ground level, steel moment frames are placed in the longitudinal direction of the building and concrete shear walls are used in the transverse direction along existing column lines for durability of the seismic-force-resisting system.

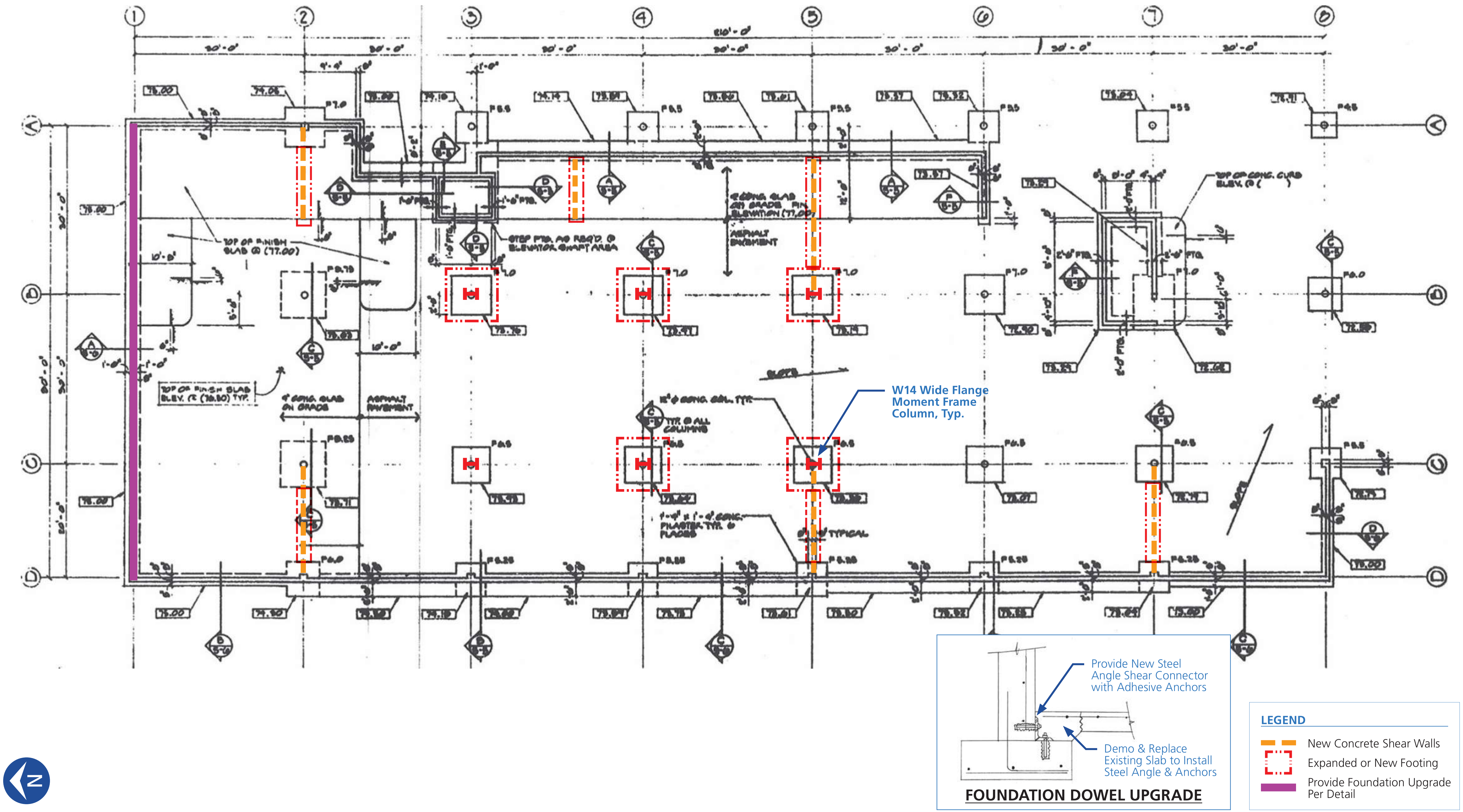
Due to the high DCR values for both the LS and CP performance objectives, a reduced structural retrofit cannot be performed to meet a lower CP performance objective versus the LS performance objective. Similar retrofits are required for both performance objectives.

3.2.3.5 Probable Construction Costs and Other Considerations

The probable construction cost to perform the recommended structural seismic upgrades to meet the Life Safety or Collapse Prevention performance objectives is \$3.08M. The estimate provided in Appendix B includes an escalation table showing escalation for 1 year, 2 years, and out to 5 years. This cost includes labor, materials, equipment, and general contractor general conditions (mobilization), overhead, and profit. Additional geotechnical study and evaluation of the building subgrade are excluded from the construction probable cost estimate. The estimate assumes the building is unoccupied and phasing is not required.

The estimate is based on the mechanical, electrical, plumbing, and fire protection (M/E/P/FP) systems being modified to accommodate seismic work, but M/E/P/FP systems are not upgraded to the latest building codes for these systems. According to the International Existing Building Code (IEBC), a voluntary seismic upgrade is considered a Level 2 alteration. A Level 2 alteration does not require upgrades to all building components. However, a Level 3 alteration, which requires the building to be brought up to full compliance with current codes, can be triggered if the work area exceeds 50 percent of the total floor area. To avoid placing the voluntary seismic upgrade work into a Level 3 alteration, care would need to be taken to define the work area to only the actual floor areas occupied by the upgraded components.

It is recommended that operational limitations, historical or architectural factors, nonstructural components, and key systems in the building also be evaluated for their useful life or use issues. These include but are not limited to the electrical power distribution system, fire alarm system, HVAC equipment, exterior roof/windows, foundation drainage, and interior finishes.



6300 Building — Foundation Retrofit Concept

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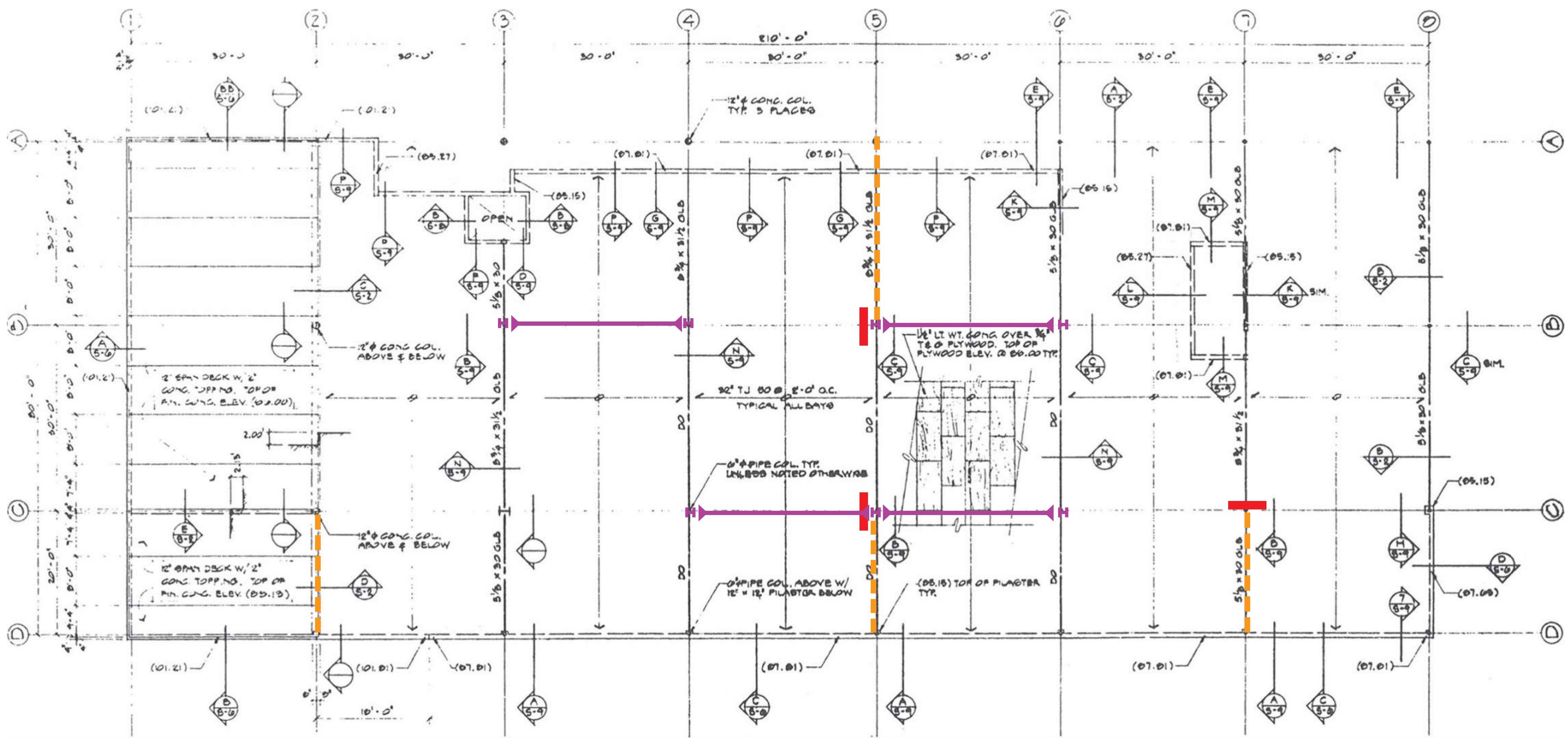
Figure 3.2-6



LEGEND

New Wood Shear Walls. Provide Connections To Concrete Walls Below

Cross Tie

New Steel Moment Frame Beams & Columns

ReidMiddleton

6300 Building — First Floor Retrofit Concept

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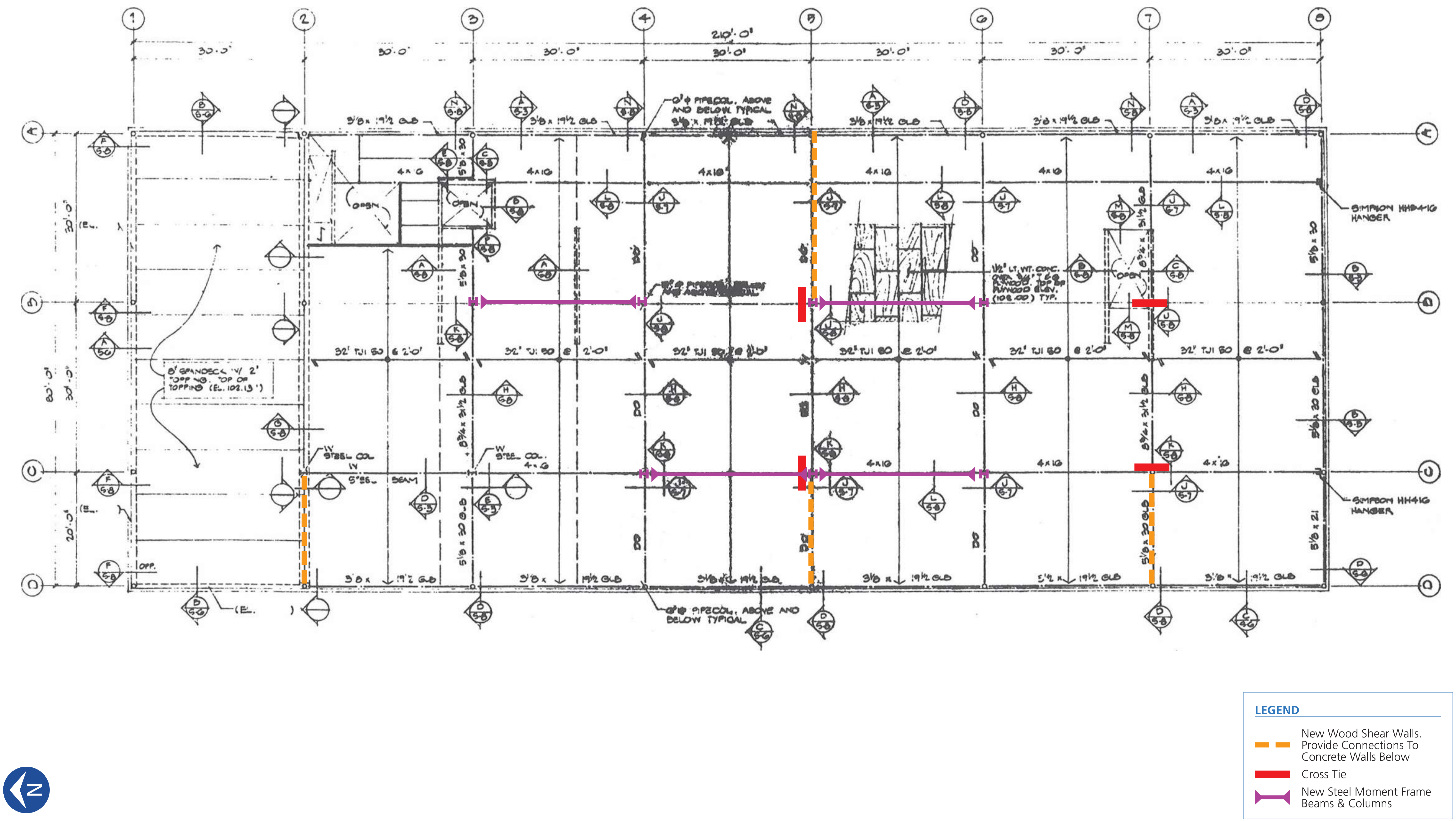
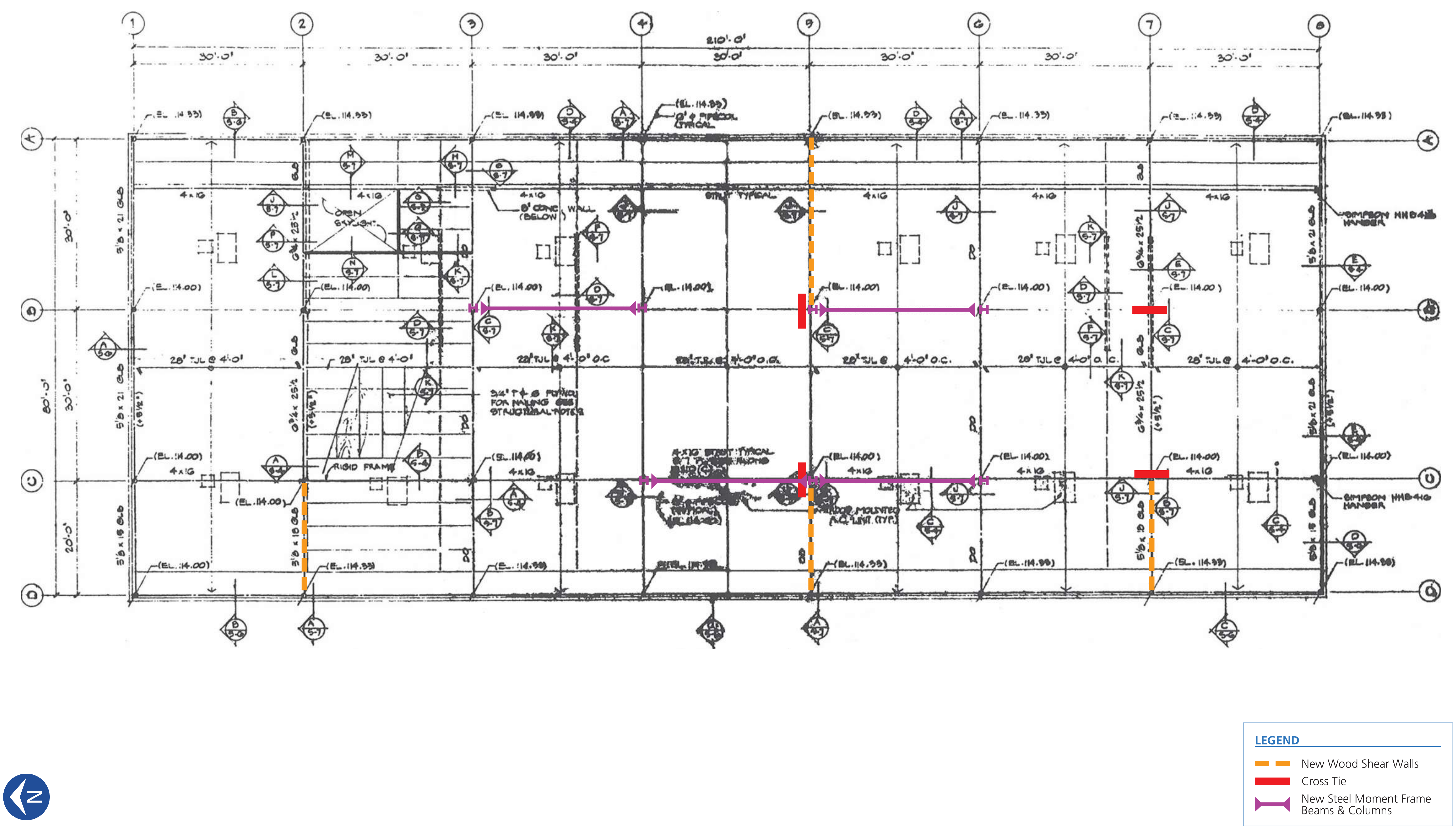
Figure 3.2-7



6300 Building — Second Floor Retrofit Concept

City of Tukwila Multi-Building Seismic Assessments Update - June 2022

Figure 3.2-8



6300 Building — Roof Retrofit Concept

City of Tukwila Multi-Building Seismic Assessments Update - June 2022

Figure 3.2-9

3.3 Tukwila Community Center

3.3.1 Building Description

Year Built:	1995
Number of Stories:	1
Floor Area:	55,000 SF

The Tukwila Community Center is a one-story building located in the northern end of Tukwila, along the Green River. The building consists of two low-rise, rectangular wing sections and a 38-foot-tall circular high-roof rotunda between the wings. The east wing also includes a 38-foot-tall high-roof gymnasium. The rotunda construction consists of a wood- and steel-framed roof with a wood structural-panel diaphragm supported by steel, masonry-clad columns. The east and west wings are generally wood- and steel-framed roofs with wood structural-panel diaphragms supported by wood and light-gage steel stud walls with a masonry façade. The gymnasium is constructed of steel roof trusses and metal roof deck supported by concrete masonry unit (CMU) perimeter walls.

Figure 3.3-1. Community Center, Southeast Exterior.

Figure 3.3-2. Community Center, West Exterior.

3.3.2 Structural System

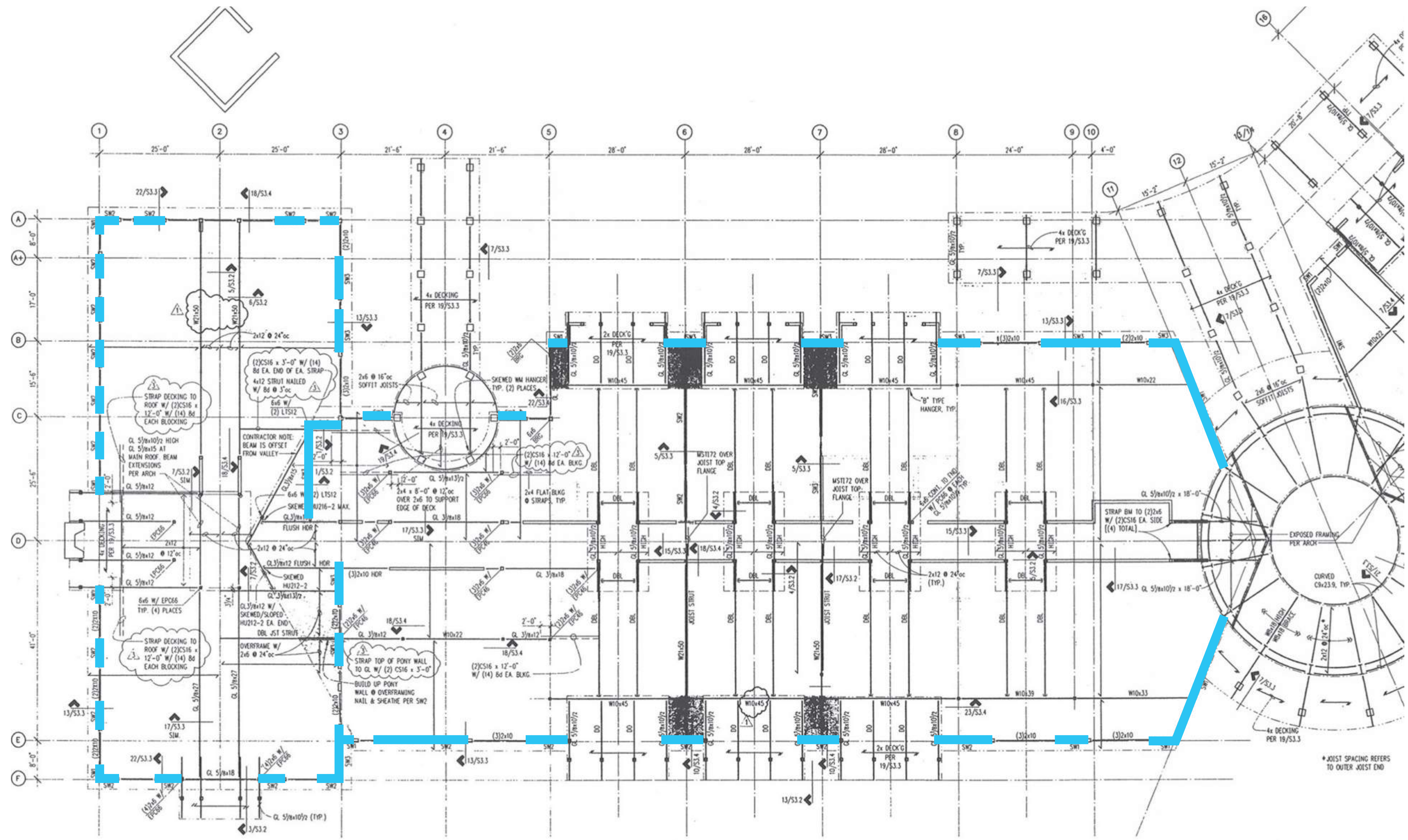
The Community Center functions as a place for the City's residents to participate in a wide range of activities, from exercising to art classes. The building also houses the Parks and Recreation administration and serves as an emergency shelter for the City. The building's gravity and lateral systems are summarized in Table 3.3-1 and shown graphically in Figures 3.3-3 through 3.3-6.

Table 3.3-1. Structural System Description of Tukwila Community Center.

Structural System	Description
Roof	At the west and east wings, wood and steel beams support plywood roof sheathing. Wood walls, built-up wood columns, and hollow steel section (HSS) columns provide gravity support for the roof framing. At the rotunda, wood joists and steel beams support plywood roof sheathing. Steel channels and beams support the center skylight. Wide-flange steel columns provide gravity support to the roof system. At the gym area, steel trusses support the 18-gauge metal roof deck. CMU walls provide gravity support to the gym roof framing. Glulam beams provide support for the plywood roof sheathing on prefabricated wood I-joists. Light gauge steel walls provide gravity support for the roof framing.
Floor	The floor is a 4-inch slab on grade.
Foundations	At the west and east wings, perimeter wood walls are supported on continuous concrete footings. Interior columns are supported on concrete spread footings. At the rotunda, wide-flange steel columns are supported on a continuous circular footing. The gym area CMU walls are supported on continuous concrete footings. The racquetball court light gauge steel walls are supported on continuous concrete footings.
Lateral	The west and east wing wood shear walls resist lateral loads in both the transverse and longitudinal directions. On the east wing and gym area, partially grouted CMU walls provide lateral support in both the transverse and longitudinal directions. At the rotunda, wide-flange steel columns acting as inverted pendulums resist lateral loads. Light gauge steel shear walls at the racquetball court area provide lateral support in both the transverse and longitudinal directions.

3.3.2.1 ASCE 41 Classified Building Type

Use of ASCE 41 for seismic evaluations requires buildings to be classified from a group of common building types historically defined in previous seismic evaluation standards (ATC-14, FEMA 310, and ASCE 31-03). The building is classified in ASCE 41, Table 3-1, as two different building types: a Wood Light Frame structure, **W2**, and a CMU Shear Wall Building with Flexible Diaphragms, **RM1**. These building types include those buildings that have bearing walls constructed of reinforced concrete, CMU block, and wood; and elevated floor and roof framing structural systems consisting of wood or other flexible diaphragms.



LEGEND

Wood Shear Wall

Community Center — First Floor West Wing Existing

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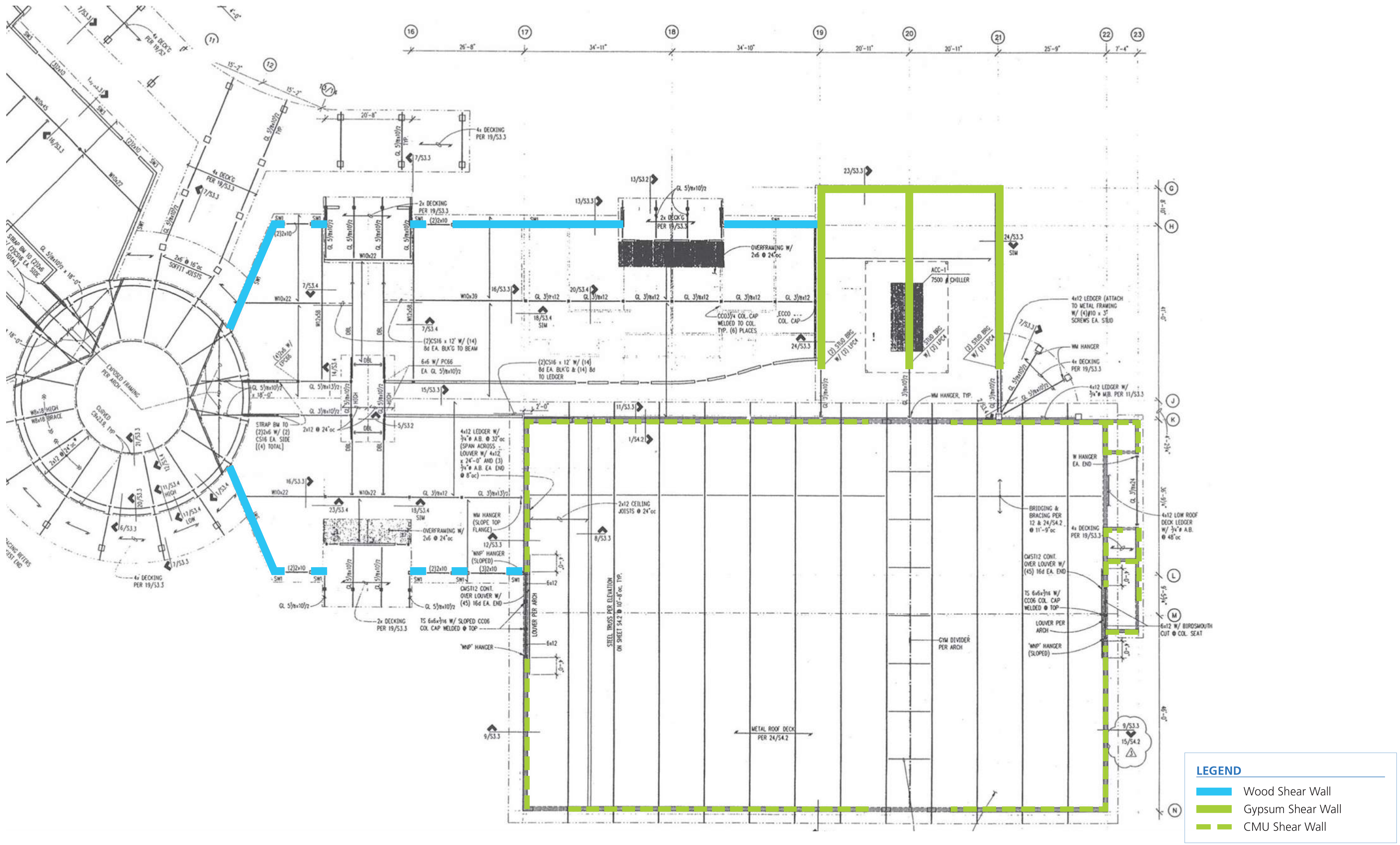
Figure 3.3-3



Community Center — First Floor East Wing Existing

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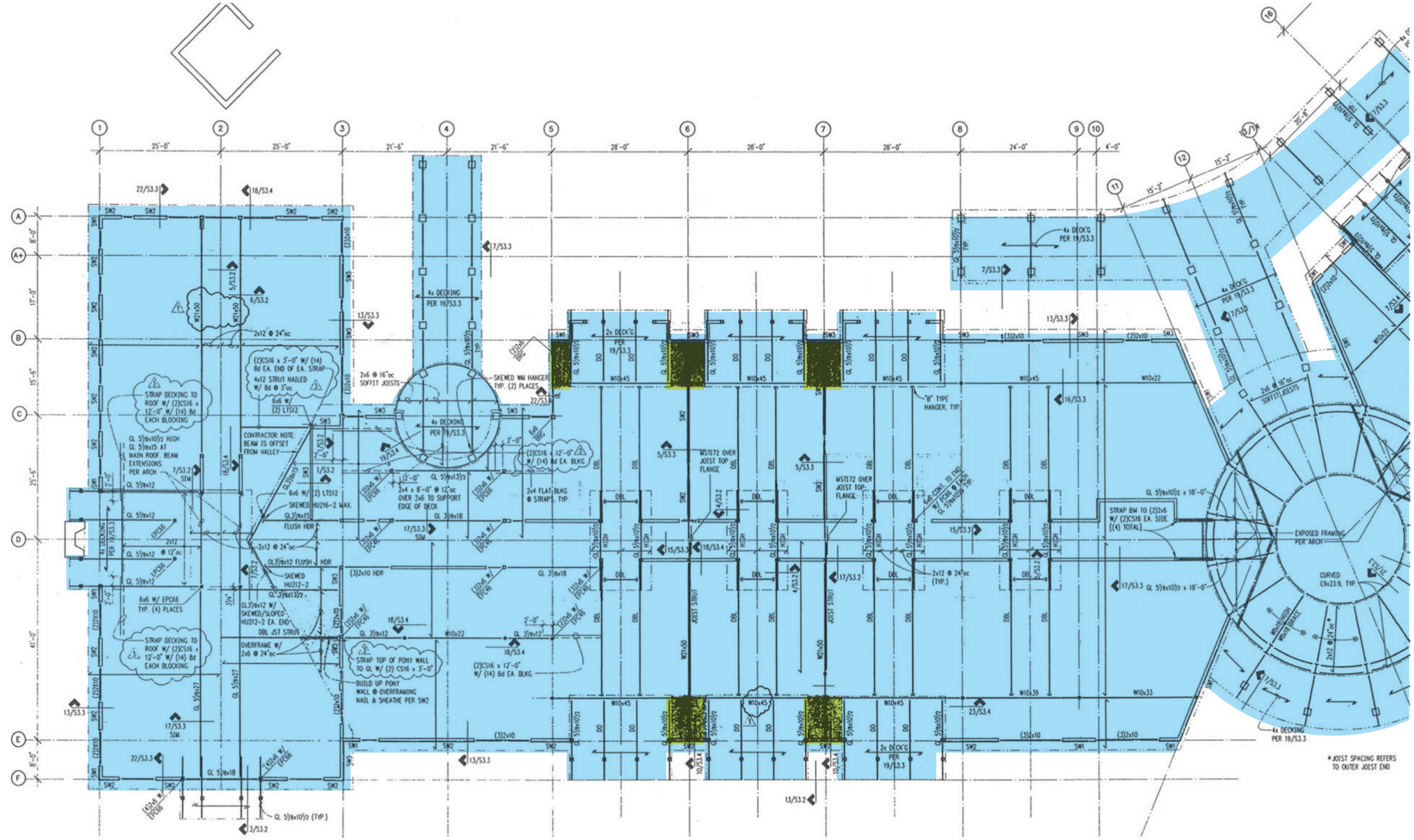
Figure 3.3-4



ReidMiddleton

Community Center — Roof West Wing Existing

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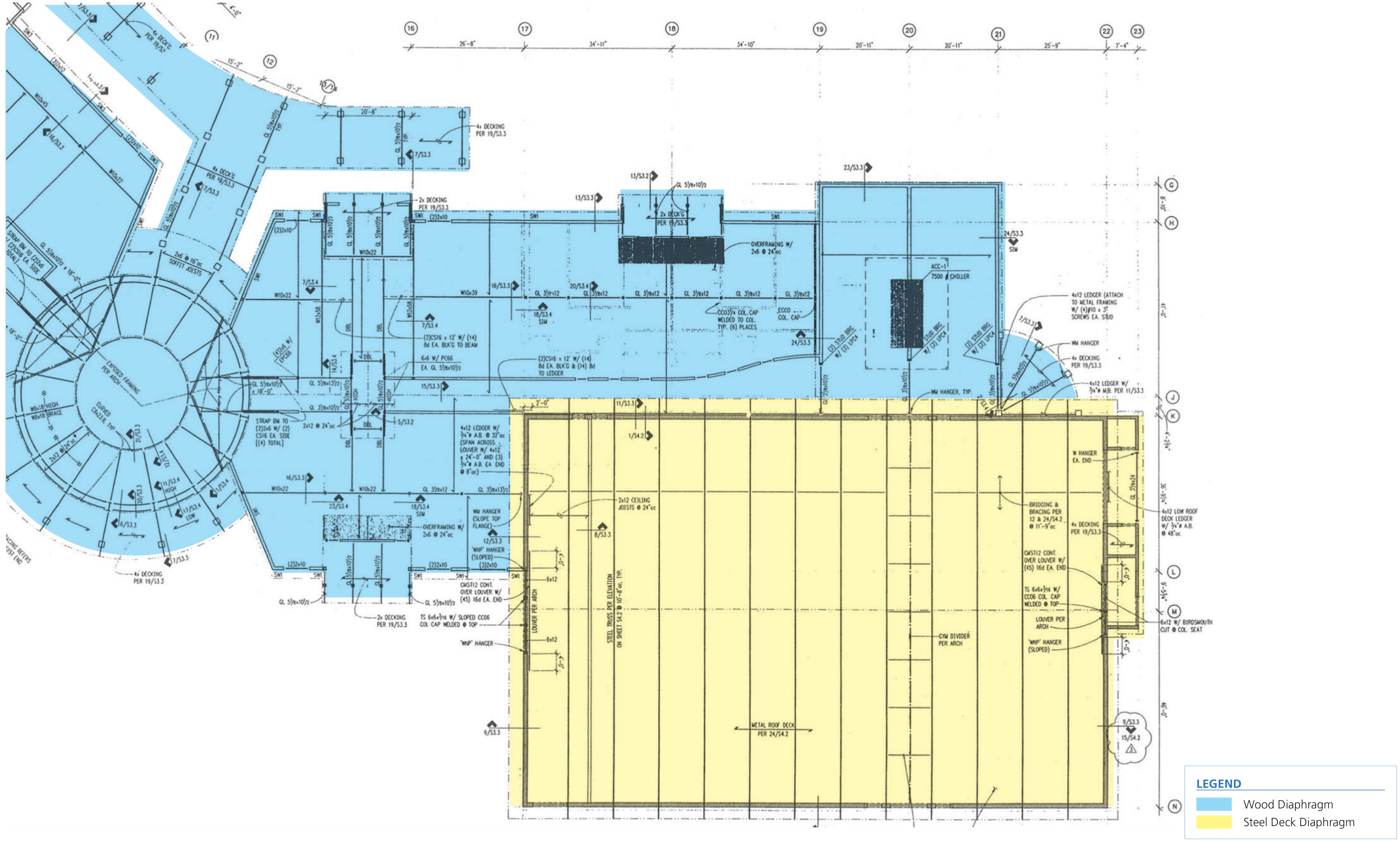
Figure 3.3-5



Community Center — Roof East Wing Existing

City of Tukwila Multi-Building Seismic Assessments Update - June 2022

Figure 3.3-6

3.3.3 Seismic Evaluation Findings

3.3.3.1 Seismic Deficiencies

The seismic deficiencies identified during the Tier 1 and Tier 2 detailed evaluation phases are summarized below. Commentary for each deficiency is provided based on the detailed seismic evaluation.

Table 3.3-2. Identified Seismic Deficiencies for Tukwila Community Center.

Deficiency	Description
Wood Shear Walls	The wood shear walls located in both the transverse and longitudinal directions have shear DCRs > 2 and do not have adequate hold-downs. The lack of adequate hold-downs may lead to rocking of the wall, allowing excessive deflections, and may lead to the walls' failure well before reaching the walls' full shear strength.
Masonry Shear Walls	The masonry shear walls are limited to the gymnasium area, which is one of the areas designated for a community shelter in the case of an emergency. Some of the masonry shear walls located around the perimeter of the gymnasium are significantly overstressed, while many of the others are very close to their design strength.
Racquetball Court Walls	These walls rely on gypsum wall board to resist lateral loads and are overstressed.
Gymnasium Roof Diaphragm	The light gauge metal roof located above the gymnasium lacks sufficient shear capacity to transfer the required lateral loads.
Wood Diaphragms	The horizontal roof diaphragm lacks ties and struts in several key locations. This will limit the diaphragm's ability to transfer forces into the shear walls below.
Foundations/ Liquefaction	The building is currently founded on traditional spread foundations. The site has potentially liquefiable soils and may experience differential settlement and lateral spreading during an earthquake. This will limit the building's ability to remain functional after an earthquake. Typically, buildings with similar site soil conditions are founded on piles and pile caps rather than spread footings.
Overstressed Steel Column Base Connections	The steel connection between the steel columns and the base plates at the rotunda are overstressed. Additionally, the connections between the base plate and foundation lack adequate concrete anchors to resist the applied loads.

3.3.3.2 Demand-Capacity Ratios

Tables 3.3-3 and 3.3-4 summarize the results of the Tier 2 analysis in terms of Demand-to-Capacity Ratios (DCRs). The DCR is determined by the load on the structural member divided by the member capacity. A DCR value greater than 1.0 indicates that the member is inadequate.

The maximum DCR is an envelope value considering all the shear wall segments within a shear wall at that given level. The maximum DCR is provided for both evaluation criteria: BSE-1E IO and BSE-2E LS. Appendix A provides a breakdown of the results for each evaluation criteria.

Table 3.3-3. Tier 2 Analysis, Wood Shear Wall Demand-to-Capacity Ratios.

Shear Wall Type and Direction	BSE-1E, IO Max DCR	BSE-2E, LS Max DCR	BSE-2E, CP Max DCR
SW1, X-Direction	3.49	3.71	3.14
SW2, X-Direction	2.39	2.55	2.15
SW3, X-Direction	1.86	1.98	1.67
SW1, Y-Direction	3.46	3.69	3.11
SW2, Y-Direction	2.37	2.53	2.13
SW3, Y-Direction	1.85	1.97	1.66

Table 3.3-4. Tier 2 Analysis, CMU Shear Wall Demand-to-Capacity Ratios.

Shear Wall Type	BSE-1E, IO Max DCR	BSE-2E, LS Max DCR	BSE-2E, CP Max DCR
8-inch CMU	4.26	4.39	2.50
12-inch CMU	3.23	3.33	3.29

Legend:	DCR Less Than 1.0, Adequate	DCR Greater Than 1.0, Inadequate

Based on the results shown in Tables 3.3-3 and 3.3-4, the Community Center does not meet the Immediate Occupancy objective. This analysis also confirms that the Community Center does not meet the Life Safety or Collapse Prevention performance objectives.

3.3.3.3 Recommendations

Currently, the Community Center does not meet the IO, LS, or CP performance levels. During a design-level earthquake, extensive damage of the lateral-force-resisting elements may occur, posing a risk to building occupants. The building's ability to remain functional and act as an emergency shelter following a seismic event could be severely limited. Based on the performance objectives, it is recommended the Community Center be seismically retrofitted. This is one of several potential shelter locations within the city, but because the proposed retrofit to meet IO performance would be intrusive to the building occupants, another option would be to lower the performance objective to a LS level. Limited structural retrofit may be required to meet the lower performance objective.

3.3.3.4 Structural Retrofit Concept Design

Figures 3.3-7 through 3.3-11 display schematic-level retrofit concepts to improve the LFRS and meet the IO performance objectives. This concept-level seismic upgrade discussion represents just one of several alternative seismic upgrade design solutions and is based on preliminary seismic evaluation and analysis results. Final analysis and design for seismic upgrades must include an architectural layout, defined occupancy class, and consideration for upgraded mechanical and electrical systems.

In both the longitudinal and transverse directions of the building, lateral-force-resisting elements are being added and strengthened. A major component to the retrofit would be to add sheathing and hold-downs to the existing wood shear walls. Additionally, new wood shear walls would be added at the corridor to minimize the stress to the small exterior walls that have overstressed stepped-blocked diaphragms. The masonry walls in the gymnasium would also require strengthening by adding grout to vertical cells and adding concrete walls in two locations. Strengthening is required in both the roof over the main building and in the gymnasium. The gymnasium roof requires adding rigid diaphragm bracing, while straps, blocking, and drag struts are being added to the wood roofs. The steel columns in the rotunda require modifications to their base connections that include adding steel plates, anchor bolts, and welds. Because the site may be prone to liquefaction and lateral spreading, compaction grouting is recommended for inside the building and 10 feet outside the building's perimeter.

A reduced structural retrofit can be performed to meet the lower CP performance objective. However, since the majority of the deficiencies associated with the building are related to the site soils, a lack of connections, and a complete load path, most upgrade requirements in the IO performance objective schematic-level retrofit concepts are still required. Reduced retrofit concepts include reductions to the amount of new wood shear walls at the first floor and reduction of bracing and wall upgrades at the first-floor walls.

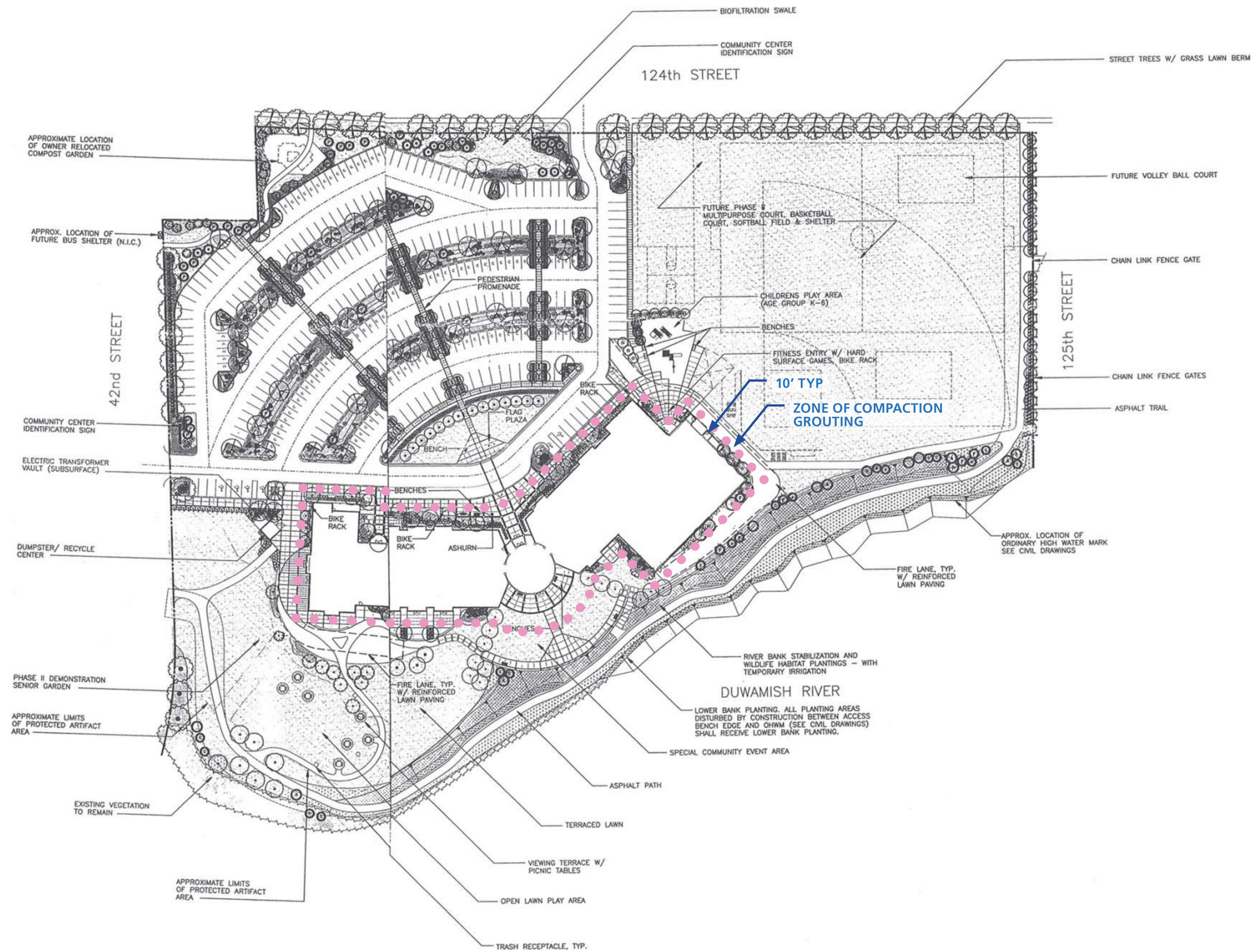
3.3.3.5 Probable Construction Costs and Other Considerations

The probable construction costs to perform the recommended structural seismic upgrades to meet the Immediate Occupancy and Collapse Prevention performance objectives are $13.71M and $13.59M, respectively. The estimates provided in Appendix B include an escalation table showing escalation for 1 year, 2 years, and out to 5 years. The costs include labor, materials, equipment, and general contractor general conditions (mobilization), overhead, and profit. Additional geotechnical study and evaluation of the building subgrade are excluded from the construction probable cost estimates. The estimates assume the building is unoccupied and phasing is not required.

These estimates are based on the mechanical, electrical, plumbing, and fire protection (M/E/P/FP) systems being modified to accommodate seismic work, but M/E/P/FP systems are not upgraded to the latest building codes for these systems. Upgrades to the lateral systems and affected M/E/P/EP systems may trigger additional upgrades. According to the International Existing Building Code (IEBC), a voluntary seismic upgrade is considered a Level 2 alteration. A Level 2 alteration does not require upgrades to all building components. However, a Level 3 alteration, which requires the building to be brought up to full compliance with current codes, can be triggered if the work area exceeds 50 percent of the total floor area. To avoid placing the voluntary seismic upgrade work into a Level 3 alteration, care would need to be taken to define the work area to only the actual floor areas occupied by the upgraded components.

It is recommended that operational limitations, historical or architectural factors, nonstructural components, and key systems in the building also be evaluated for their useful life or use issues.

These include but are not limited to HVAC, exterior lighting, access control, and interior/exterior finishes. In addition, the cost of seismic upgrades to this building to improve its ability to remain in continuous operation after a seismic event may be disproportional to the value of the building.



Community Center — Site Plan Retrofit Concept

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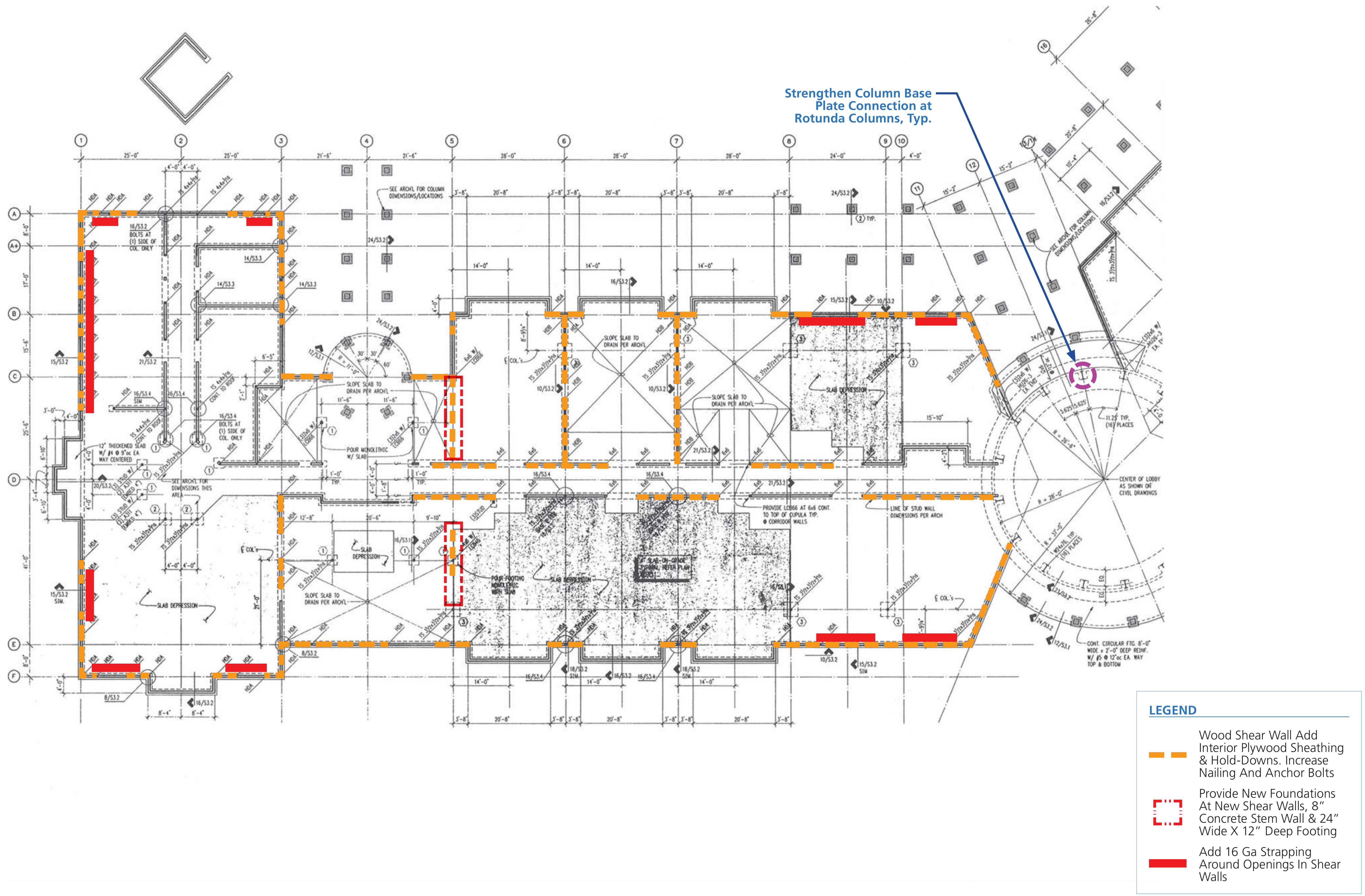
Figure 3.3-7



Community Center — First Floor West Wing Retrofit Concept
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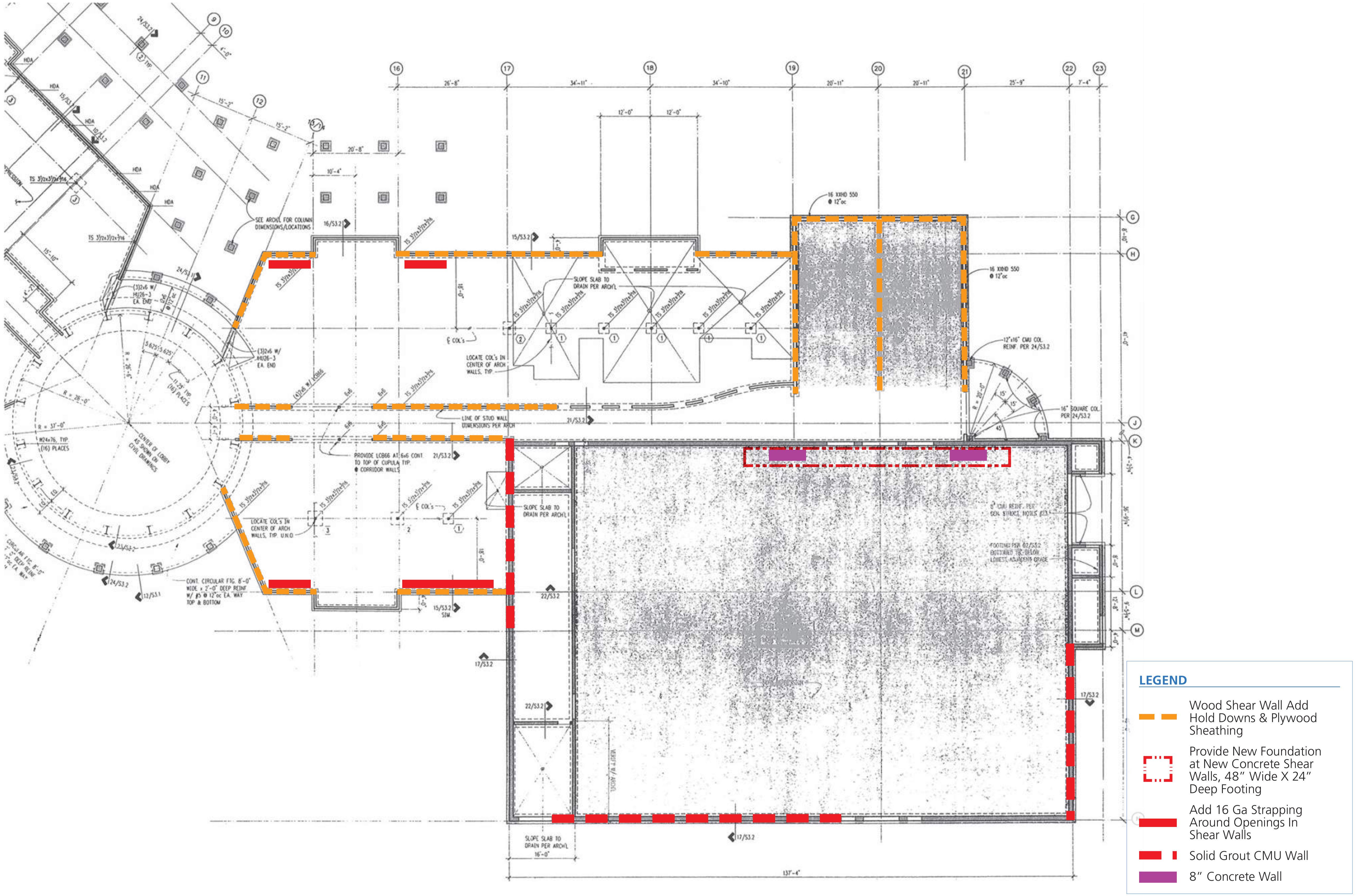
Figure 3.3-8



ReidMiddleton

Community Center — First Floor East Wing Retrofit Concept
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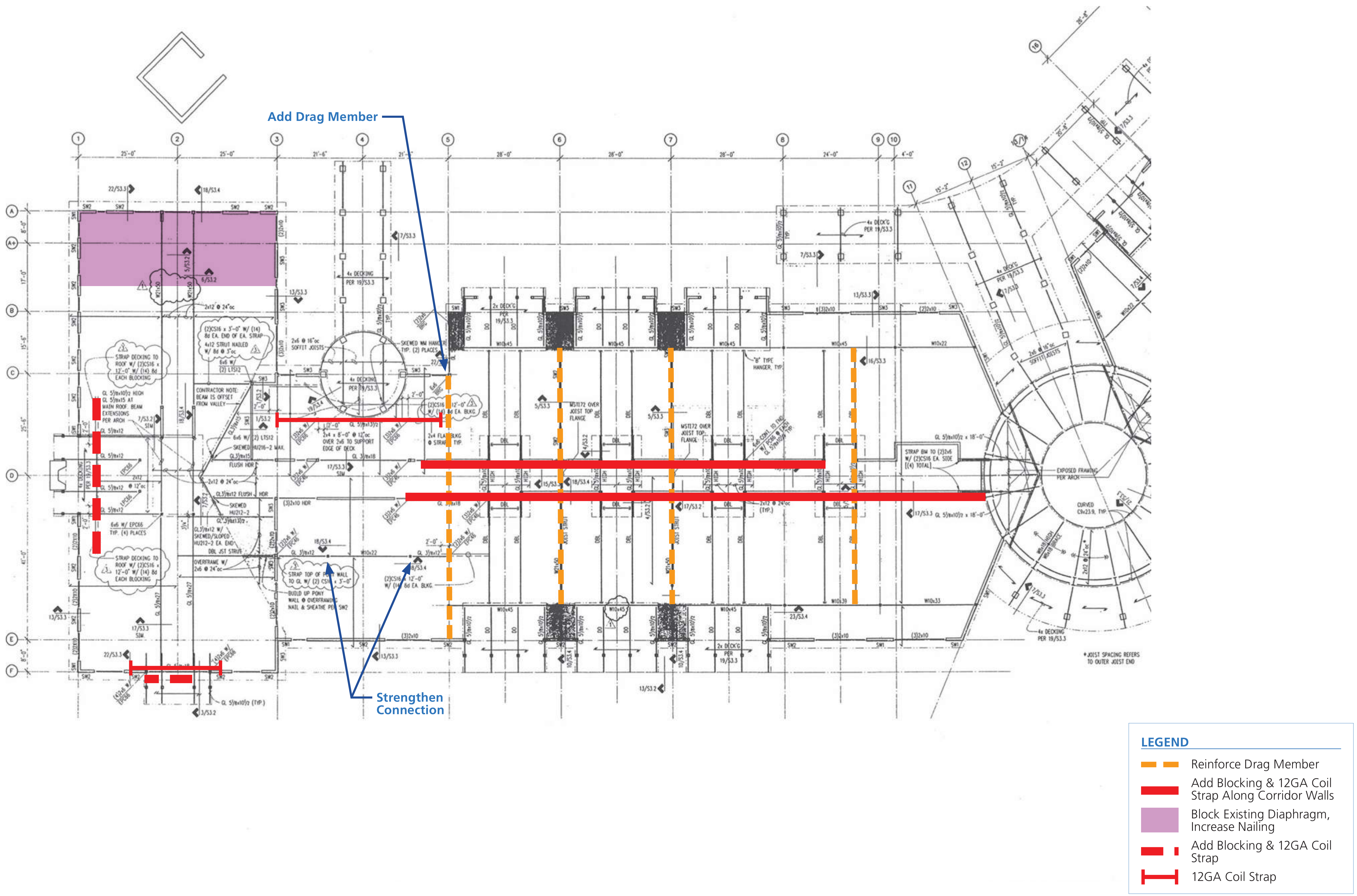
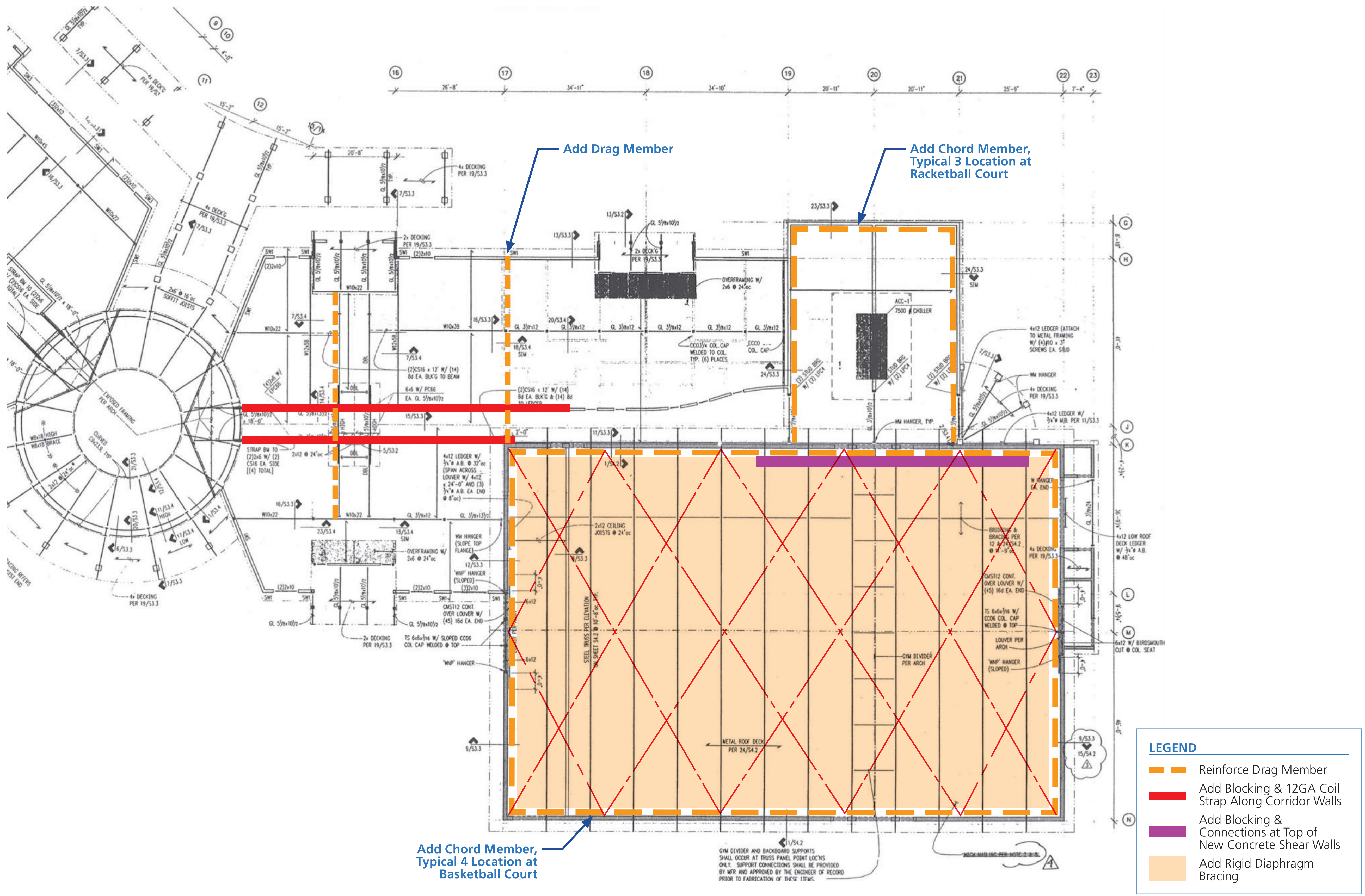
Figure 3.3-9



Community Center — Roof West Wing Retrofit Concept
City of Tukwila Multi-Building Seismic Assessments Update - June 2022

Figure 3.3-10



Community Center — Roof East Wing Retrofit Concept

City of Tukwila Multi-Building Seismic Assessments Update - June 2022

Figure 3.3-11

Appendix A
Seismic Screening Checklists and Calculations

17-2 Collapse Prevention Basic Configuration Checklist

Low Seismicity

Building System—General

C	NC	N/A	U	EVALUATION STATEMENT	COMMENT
X				LOAD PATH: The structure contains 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. *(Tier 2: Sec. 5.4.1.1; Commentary: Sec. A.2.1.10)*	
		X		ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building is greater than 0.25% of the height of the shorter building in low seismicity, 0.5% in moderate seismicity, and 1.5% in high seismicity. *(Tier 2: Sec. 5.4.1.2; Commentary: Sec. A.2.1.2)*	
X				MEZZANINES: Interior mezzanine levels are braced independently from the main structure or are anchored to the seismic-force-resisting elements of the main structure. *(Tier 2: Sec. 5.4.1.3; Commentary: Sec. A.2.1.3)*	

Building System—Building Configuration

C	NC	N/A	U	EVALUATION STATEMENT	COMMENT
X				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. *(Tier 2: Sec. 5.4.2.1; Commentary: Sec. A.2.2.2)*	
X				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. *(Tier 2: Sec. 5.4.2.2; Commentary: Sec. A.2.2.3)*	
	X			VERTICAL IRREGULARITIES: All vertical elements in the seismic-force-resisting system are continuous to the foundation. *(Tier 2: Sec. 5.4.2.3; Commentary: Sec. A.2.2.4)*	Wood shear walls are not continuous to the foundation.
	X			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. *(Tier 2: Sec. 5.4.2.4; Commentary: Sec. A.2.2.5)*	The Lateral Force Resisting System (LFRS) in the south wing of the building exists only on the east face which does not extend 30% of the LFRS dimension of the floor below.
X				MASS: There is no change in effective mass of more than 50% from one story to the next. Light roofs, penthouses, and mezzanines need not be considered. *(Tier 2: Sec. 5.4.2.5; Commentary: Sec. A.2.2.6)*	
		X		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. *(Tier 2: Sec. 5.4.2.6; Commentary: Sec. A.2.2.7)*	Building has a flexible diaphragm

17-2 Collapse Prevention Basic Configuration Checklist

Moderate Seismicity

(Complete the Following Items in Addition to the Items for Low Seismicity)

Geologic Site Hazards

C	NC	N/A	U	EVALUATION STATEMENT	COMMENT
X				LIQUEFACTION: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance do not exist in the foundation soils at depths within 50 ft (15.2 m) under the building. *(Tier 2: Sec. 5.4.3.1; Commentary: Sec. A.6.1.1)*	Not a Site Class F site per 2008 Geotechncial report completed as part of original report.
			X	SLOPE FAILURE: The building site is located away from potential earthquake-induced slope failures or rockfalls so that it is unaffected by such failures or is capable of accommodating any predicted movements without failure. *(Tier 2: Sec. 5.4.3.1; Commentary: Sec. A.6.1.2)*	Building is located on a hillside site. Stability of the slope is unknown.
X				SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site are not anticipated. *(Tier 2: Sec. 5.4.3.1; Commentary: Sec. A.6.1.3)*	

High Seismicity

(Complete the Following Items in Addition to the Items for Low and Moderate Seismicity)

Foundation Configuration

C	NC	N/A	U	EVALUATION STATEMENT	COMMENT
X				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 $0.6S_a$. *(Tier 2: Sec. 5.4.3.3; Commentary: Sec. A.6.2.1)*	127'/195' = 0.651 0.6Sa = 0.6(0.701) = 0.421
	X			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. *(Tier 2: Sec. 5.4.3.4; Commentary: Sec. A.6.2.2)*	No beams/slabs/soils classified as Site Class A, B, or C between shallow foundation elements.

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

17-6. Collapse Prevention Structural Checklist for Building Type W2

Low and Moderate Seismicity

Seismic-Force-Resisting System

C	NC	N/A	U	EVALUATION STATEMENT	COMMENT
X				REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2. *(Tier 2: Sec. 5.5.1.1; Commentary: Sec. A.3.2.1.1)*	
X				SHEAR STRESS CHECK: The shear stress in the shear walls, calculated using the Quick Check procedure of Section 4.4.3.3, is less than the following values: Structural panel sheathing, 1,000 lb/ft (14.6 kN/m); Diagonal sheathing, 700 lb/ft (10.2 kN/m); Straight sheathing, 100 lb/ft (1.5 kN/m); All other conditions, 100 lb/ft (1.5 kN/m). *(Tier 2: Sec. 5.5.3.1.1 ; Commentary: Sec.A.3.2.7.1)*	
X				STUCCO (EXTERIOR PLASTER) SHEAR WALLS: Multi-story buildings do not rely on exterior stucco walls as the primary seismic-force-resisting system. *(Tier 2: Sec. 5.5.3.6.1; Commentary: Sec. A.3.2.7.2)*	
X				GYPSUM WALLBOARD OR PLASTER SHEAR WALLS: Interior plaster or gypsum wallboard is not used for shear walls on buildings more than one story high with the exception of the uppermost level of a multi-story building. *(Tier 2: Sec. 5.5.3.6.1; Commentary: Sec. A.3.2.7.3)*	
X				NARROW WOOD SHEAR WALLS: Narrow wood shear walls with an aspect ratio greater than 2-to-1 are not used to resist seismic forces. *(Tier 2: Sec. 5.5.3.6.1; Commentary: Sec. A.3.2.7.4)*	
	X			WALLS CONNECTED THROUGH FLOORS: Shear walls have an interconnection between stories to transfer overturning and shear forces through the floor. *(Tier 2: Sec. 5.5.3.6.2; Commentary: Sec. A.3.2.7.5)*	
		X		HILLSIDE SITE: For structures that are taller on at least one side by more than one-half story because of a sloping site, all shear walls on the downhill slope have an aspect ratio less than 1-to-2. *(Tier 2: Sec. 5.5.3.6.3; Commentary: Sec. A.3.2.7.6)*	Wood shearwalls only exist above grade.
		X		CRIPPLE WALLS: Cripple walls below first-floor-level shear walls are braced to the foundation with wood structural panels. *(Tier 2: Sec. 5.5.3.6.4; Commentary: Sec. A.3.2.7.7)*	
		X		OPENINGS: Walls with openings greater than 80% of the length are braced with wood structural panel shear walls with aspect ratios of not more than 1.5-to-1 or are supported by adjacent construction through positive ties capable of transferring the seismic forces. *(Tier 2: Sec. 5.5.3.6.5; Commentary: Sec. A.3.2.7.8)*	

Connections

C	NC	N/A	U	EVALUATION STATEMENT	COMMENT
X				WOOD POSTS: There is a positive connection of wood posts to the foundation. *(Tier 2: Sec. 5.7.3.3; Commentary: Sec. A.5.3.3)*	

17-6. Collapse Prevention Structural Checklist for Building Type W2

X				WOOD SILLS: All wood sills are bolted to the foundation. *(Tier 2: Sec. 5.7.3.3; Commentary: Sec. A.5.3.4)*	
X				GIRDER–COLUMN CONNECTION: There is a positive connection using plates, connection hardware, or straps between the girder and the column support. *(Tier 2: Sec. 5.7.4.1; Commentary: Sec. A.5.4.1)*	

High Seismicity

(Complete the Following Items in Addition to the Items for Low and Moderate Seismicity)

Connections

C	NC	N/A	U	EVALUATION STATEMENT	COMMENT
X				WOOD SILL BOLTS: Sill bolts are spaced at 6 ft or less with acceptable edge and end distance provided for wood and concrete. *(Tier 2: Sec. 5.7.3.3; Commentary: Sec. A.5.3.7)*	

Diaphragms

C	NC	N/A	U	EVALUATION STATEMENT	COMMENT
X				DIAPHRAGM CONTINUITY: The diaphragms are not composed of split-level floors and do not have expansion joints. *(Tier 2: Sec. 5.6.1.1; Commentary: Sec. A.4.1.1)*	
X				ROOF CHORD CONTINUITY: All chord elements are continuous, regardless of changes in roof elevation. *(Tier 2: Sec. 5.6.1.1; Commentary: Sec. A.4.1.3)*	
X				DIAPHRAGM REINFORCEMENT AT OPENINGS: There is reinforcing around all diaphragm openings larger than 50% of the building width in either major plan dimension. *(Tier 2: Sec. 5.6.1.5; Commentary: Sec. A.4.1.8)*	
		X		STRAIGHT SHEATHING: All straight-sheathed diaphragms have aspect ratios less than 1-to-1 in the direction being considered. *(Tier 2: Sec. 5.6.2; Commentary: Sec. A.4.2.1)*	Diaphragm consists of plywood sheathing
X				SPANS: All wood diaphragms with spans greater than 12 ft (3.6 m) consist of wood structural panels or diagonal sheathing. *(Tier 2: Sec. 5.6.2; Commentary: Sec. A.4.2.2)*	
		X		DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 30 ft (9.1 m) and aspect ratios less than or equal to 3-to-1. *(Tier 2: Sec. 5.6.2; Commentary: Sec. A.4.2.3)*	Diaphragm is blocked plywood sheathing.
X				OTHER DIAPHRAGMS: The diaphragms do not consist of a system other than wood, metal deck, concrete, or horizontal bracing. *(Tier 2: Sec. 5.6.5; Commentary: Sec. A.4.7.1)*	

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

17-24 Collapse Prevention Structural Checklist for Building Types C2 and C2a

Low and Moderate Seismicity

Seismic-Force-Resisting System

C	NC	N/A	U	EVALUATION STATEMENT	COMMENT
		X		COMPLETE FRAMES: Steel or concrete frames classified as secondary components form a complete vertical-load-carrying system. *(Tier 2: Sec. 5.5.2.5.1; Commentary: Sec. A.3.1.6.1)*	
X				REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2. *(Tier 2: Sec.5.5.1.1; Commentary: Sec. A.3.2.1.1)*	
	X			SHEAR STRESS CHECK: The shear stress in the concrete shear walls, calculated using the Quick Check procedure of Section 4.4.3.3, is less than the greater of 100 lb/in.^{2} (0.69 MPa) or $2\sqrt{f'_c}$. *(Tier 2: Sec.5.5.3.1.1; Commentary: Sec. A.3.2.2.1)*	Walls are overstressed for LS
X				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. *(Tier 2: Sec.5.5.3.1.3; Commentary: Sec. A.3.2.2.2)*	

Connections

C	NC	N/A	U	EVALUATION STATEMENT	COMMENT
	X			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 strength to resist the connection force calculated in the Quick Check procedure of Section 4.4.3.7. *(Tier 2: Sec.5.7.1.1; Commentary: Sec. A.5.1.1)*	Concrete walls are not anchored to the diaphragm for out of plane forces.
	X			TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls. *(Tier 2: Sec.5.7.2; Commentary: Sec. A.5.2.1)*	
X				FOUNDATION DOWELS: Wall reinforcement is doweled into the foundation with vertical bars equal in size and spacing to the vertical wall reinforcing directly above the foundation. *(Tier 2: Sec. 5.7.3.4; Commentary: Sec. A.5.3.5)*	

High Seismicity

(Complete the Following Items in Addition to the Items for Low and Moderate Seismicity)

Seismic-Force-Resisting System

C	NC	N/A	U	EVALUATION STATEMENT	COMMENT
		X		DEFLECTION COMPATIBILITY: Secondary components have the shear capacity to develop the flexural strength of the components. *(Tier 2: Sec.5.5.2.5.2; Commentary: Sec. A.3.1.6.2)*	No secondary concrete components
		X		FLAT SLABS: Flat slabs or plates not part of the seismic-force-resisting system have continuous bottom steel through the column joints. *(Tier 2: Sec.5.5.2.5.3; Commentary: Sec. A.3.1.6.3)*	No concrete columns

17-24 Collapse Prevention Structural Checklist for Building Types C2 and C2a

C	NC	N/A	U	EVALUATION STATEMENT	COMMENT
		X		COUPLING BEAMS: The ends of both walls to which the coupling beam is attached are supported at each end to resist vertical loads caused by overturning. *(Tier 2: Sec.5.5.3.2.1; Commentary: Sec. A.3.2.2.3)*	

Diaphragms (Stiff or Flexible)

C	NC	N/A	U	EVALUATION STATEMENT	COMMENT
X				DIAPHRAGM CONTINUITY: The diaphragms are not composed of split-level floors and do not have expansion joints. *(Tier 2: Sec.5.6.1.1; Commentary: Sec. A.4.1.1)*	
X				OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 25% of the wall length. *(Tier 2: Sec.5.6.1.3; Commentary: Sec. A.4.1.4)*	

Flexible Diaphragms

C	NC	N/A	U	EVALUATION STATEMENT	COMMENT
X				CROSS TIES: There are continuous cross ties between diaphragm chords. *(Tier 2: Sec.5.6.1.2; Commentary: Sec. A.4.1.2)*	
		X		STRAIGHT SHEATHING: All straight-sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered. *(Tier 2: Sec.5.6.2; Commentary: Sec. A.4.2.1)*	All wood sheathing is plywood
X				SPANS: All wood diaphragms with spans greater than 24 ft (7.3 m) consist of wood structural panels or diagonal sheathing. *(Tier 2: Sec.5.6.2; Commentary: Sec. A.4.2.2)*	
		X		DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 ft (12.2 m) and aspect ratios less than or equal to 4-to-1. *(Tier 2: Sec.5.6.2; Commentary: Sec. A.4.2.3)*	All wood sheathing is plywood
X				OTHER DIAPHRAGMS: Diaphragms do not consist of a system other than wood, metal deck, concrete, or horizontal bracing. *(Tier 2: Sec.5.6.5; Commentary: Sec. A.4.7.1)*	

Connections

C	NC	N/A	U	EVALUATION STATEMENT	COMMENT
		X		UPLIFT AT PILE CAPS: Pile caps have top reinforcement, and piles are anchored to the pile caps. *(Tier 2: Sec.5.7.3.5; Commentary: Sec. A.5.3.8)*	Foundation does not utilize piles.

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

Design Criteria

ReidMiddleton

728 134th St. SW, Suite 200
Everett, WA 98204
425-741-3800
www.reidmiddleton.com

DESIGN SUMMARY

The City Hall building is 2 stories on a sloped grade sloping from the second floor on the north side down to the first floor on the south side. At the first floor the building is made of concrete retaining and shear walls. Starting at the second floor and going up the building is wood framed. Large glulam trusses support the roof. The lateral force resisting system at the second floor is comprised of wood shear walls.

CODES AND REFERENCES

General

- ASCE 41-17 Minimum Design Loads for Buildings and Other Structures

Concrete

- ACI 318-14 Building Code Requirements for Structural Concrete

Wood

- ANSI/AF&PA-2015 National Design Specification for Wood Construction
- AITC Timber Construction Manual, Sixth Edition

Catalogs and Miscellaneous

- Trus-Joist MacMillan Catalog
- Hilti Catalog
- Simpson Strong-Tie Catalog
- Red-Built Open-Web Truss Catalog
- Red-Built Red-I Joist Catalog

Tukwila City Hall

6200 Southcenter Blvd, Tukwila, WA 98188, USA

Latitude, Longitude: 47.463224, -122.2555133

Date	3/29/2022, 10:02:06 AM
Design Code Reference Document	ASCE41-17
Custom Probability	
Site Class	D - Default (See Section 11.4.3)

Type	Description	Value
Hazard Level		BSE-2N
S_S	spectral response (0.2 s)	1.466
S_1	spectral response (1.0 s)	0.499
S_{XS}	site-modified spectral response (0.2 s)	1.76
S_{X1}	site-modified spectral response (1.0 s)	0.898
F_a	site amplification factor (0.2 s)	1.2
F_v	site amplification factor (1.0 s)	1.801
ssuh	max direction uniform hazard (0.2 s)	1.629
crs	coefficient of risk (0.2 s)	0.9
ssrt	risk-targeted hazard (0.2 s)	1.466
ssd	deterministic hazard (0.2 s)	4.288
s1uh	max direction uniform hazard (1.0 s)	0.557
cr1	coefficient of risk (1.0 s)	0.896
s1rt	risk-targeted hazard (1.0 s)	0.499
s1d	deterministic hazard (1.0 s)	1.501

Type	Description	Value
Hazard Level		BSE-1N
S_{XS}	site-modified spectral response (0.2 s)	1.173
S_{X1}	site-modified spectral response (1.0 s)	0.599

Type	Description	Value
Hazard Level		BSE-2E
S_S	spectral response (0.2 s)	1.081
S_1	spectral response (1.0 s)	0.362
S_{XS}	site-modified spectral response (0.2 s)	1.297
S_{X1}	site-modified spectral response (1.0 s)	0.701
f_a	site amplification factor (0.2 s)	1.2
f_v	site amplification factor (1.0 s)	1.938

Type	Description	Value
Hazard Level		BSE-1E
S_S	spectral response (0.2 s)	0.501
S_1	spectral response (1.0 s)	0.155
S_{XS}	site-modified spectral response (0.2 s)	0.701
S_{X1}	site-modified spectral response (1.0 s)	0.355
F_a	site amplification factor (0.2 s)	1.399
F_v	site amplification factor (1.0 s)	2.29

Type	Description	Value
Hazard Level		TL Data
T-Sub-L	Long-period transition period in seconds	6

City Hall Tier 1 Evaluation
Life Safety

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Client:	City of Tukwila	Sheet:	of
Project:	Tukwila Seismic Evaluation	Design By:	MLO
	City Hall	Date:	
		Checked By:	KRB
Project No.:	262022.017	Date:	

City Hall Tier 1 Calculations - Life Safety

Building Properties			**Code Ref.**
Building Type:	C2a/W2	*Concrete & Wood Shear Walls w/ Flexible Diaphragms*	
Area:	14,030 ft^2		
Latitude:	47.463		
Longitude:	-122.256		
Site Class:	D (Default)		
No. Stories:	2		
Building Height:	25.00 ft	*(Approximate) Height of Sloped Roof*	
Risk Category:	II		
Level of Performance:	LS	*Life Safety*	ASCE 41-17 Table 2.2

Table 2-2. Scope of Assessment Required for Tier 1 and Tier 2 with the Basic Performance Objective for Existing Buildings (BPOE)

Risk Category	Tier 1 and 2[a] BSE-1E	BSE-2E
I and II	Not evaluated	Collapse Prevention Structural Performance
	Life Safety Nonstructural Performance (3-C)	Hazards Reduced Nonstructural Performance[b] (5-D)
III	Not evaluated	Limited Safety Structural Performance[c]
	Position Retention Nonstructural Performance (2-B)	Hazards Reduced Nonstructural Performance[b] (4-D)
IV	Immediate Occupancy Structural Performance	Life Safety Structural Performance[d]
	Position Retention Nonstructural Performance (1-B)	Hazards Reduced Nonstructural Performance[b] (3-D)

[a] For Tier 1 and 2 assessments of Risk Categories I–III, Structural Performance for the BSE-1E is not explicitly evaluated.
[b] Compliance with ASCE 7 provisions for new construction is deemed to comply.
[c] For Risk Category III, the Tier 1 screening checklists shall be based on the Collapse Prevention Performance Level (S-5), except that checklist statements using the Quick Check procedures of Section 4.4.3 shall be based on M_s factors taken as the average of the values for Life Safety and Collapse Prevention.
[d] For Risk Category IV, the Tier 1 screening checklists shall be based on the Collapse Prevention Performance Level (S-5), except that checklist statements using the Quick Check procedures of Section 4.4.3 shall be based on M_s factors for Life Safety.

Seismic Properties, BSE-2E			**Code Ref.**
Mapped Short Period Accel.:	S_S =	1.081 g	OSHPD Seismic Maps
Mapped One-Sec. Accel.:	S_1 =	0.362 g	OSHPD Seismic Maps
Accel. Site Coefficient:	F_a =	1.200	OSHPD Seismic Maps
Velocity Site Coefficient:	F_v =	1.938	OSHPD Seismic Maps
Design Short Period Accel.:	$S_{DS} = (2/3)*S_s*F_a$ =	0.865 g	ASCE 41-17 Eq. 2-4
Design 1-Sec. Period Accel.:	$S_{D1} = (2/3)*S_1*F_v$ =	0.468 g	ASCE 41-17 Eq. 2-5
Level of Seismicity:	High		
Seismic Hazard Level:	2E	*5% Probability of Exceedance in 50 Years for an Existing Building*	
BSE 2E Design Short Period Accel.:	S_{XS} =	1.297 g	OSHPD Seismic Maps
BSE 2E 1-Sec. Design Short Period Accel.:	S_{X1} =	0.701 g	OSHPD Seismic Maps

Design Spectral Acceleration, BSE-2E				**Code Ref.**
Period Coefficient:	C_t =	0.020	*For All Other Framing System*	ASCE 41-17 S. 4.4.2.4
Period Coefficient:	β =	0.75	*For All Other Framing System*	ASCE 41-17 S. 4.4.2.4
Fundamental Period:	$T = C_t*h_n^{\beta}$ =	0.22 s		ASCE 41-17 Eq.4-4
Spectral Acc.:	$\mathbf{S_a = S_{X1}/T}$ =	**1.297 g**	*but S_a shall not exceed* S_{XS}	ASCE 41-17 Eq.4-3

City Hall Tier 1 Calculations - Life Safety

Weight Take-Off	Code Ref.

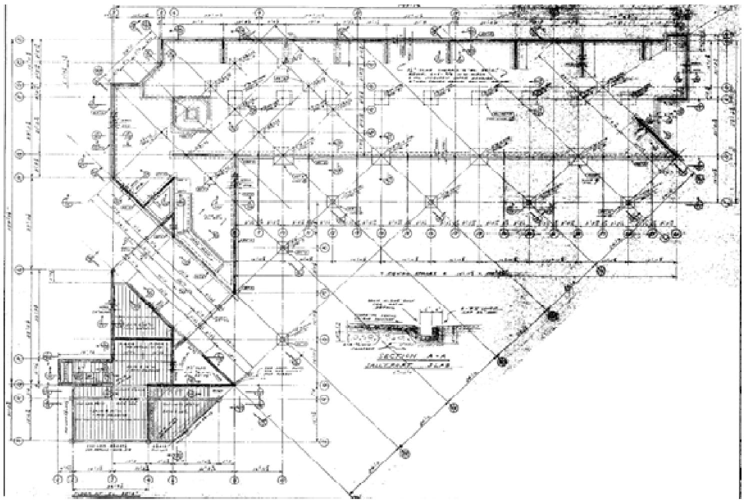

Figure 1: City Hall Foundation Plan

Ground Floor

8" Conc	228.6	kip

First Floor

2x14 @ 16" oc	2.8	psf
3/4" Plywood Shthg	3	psf
1.5" Conc topping	19	psf
Misc	5	psf
	414.2	kip

Roof

2x8 @ 24" oc	1.3	psf
GL 5.125x15 @ 12' oc	1.6	psf
1/2" plywd	2.0	psf
Misc	5.0	psf
	138.9	kip

Figure 1: City Hall Foundation Plan

Building Weight Summary		
Roof	139	kip
Level 1	643	kip
Σ	782	kip

Vertical Distribution of Psuedo-Seismic Base Shear	Code Ref.

Coefficient Exponent:	k =	1.0		ASCE 41-17 S. 4.4.2.2
Effective Seismic Building Weight:	W=	782 kips		
Modification Factor:	C =	1.2	*for CMU Buildings*	ASCE 41-17 Tbl. 4-7
Psuedo Seismic Base Shear, BSE-2E:	$V_{pseudo} = C*S_a*W =$	**1,217 kips**		

Story Shear Forces: Vertical Distribution of Pseudo Shear Forces

Floor Level [from base]	Height, h_x [ft]	Story Weight, w_x [kip]	$w_x h_x^k$ [kip*ft]	Dist. Factor C_{vx}	Lateral Force [kip]	Story Shear* [kip]
Roof	25.0	139	3,473	0.31	378	**378**
Level 2	12.0	643	7,714	0.69	839	**839**
Σ			11,187	1.0	1,217	

**Story shear will be used to check the SFRS in the structure at each respective level.*

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City Hall Tier 1 Calculations - Life Safety

Shear Stress Check - Concrete	**Code Ref.**

$$v_j^{avg} = \frac{1}{M_s}\left(\frac{V_j}{A_w}\right) \quad (4\text{-}8)$$

$A_{w,x}$ =	3363	in^2	*Horizontal cross-sectional area of all shear walls in direction x*
$A_{w,y}$ =	2784	in^2	*Horizontal cross-sectional area of all shear walls in direction y*
V_{Base} =	1,217	kip	*Max Story Shear*
M_s =	3		*Modification Factor for Shear Walls*
v_x =	120.6	psi	*Shear Stress in Walls, x-dir*
v_y =	145.7	psi	*Shear Stress in Walls, y-dir*
v_{max} =	145.7	psi	*Shear Stress in Walls*
$v_{allowable}$ =	100	psi	*Allowable Shear Stress in Walls*
DCR =	1.457	NC	*Demand Capacity Ratio*

Reinforcing Steel in Shear Walls	**Code Ref.**

City Hall	Reinforcing ratio, ρ	$\rho_{provided}$	$\rho_{required}$
Vertical	#4 @ 16" oc	0.00156	0.0012
Horizontal	#4 @ 10" oc	0.0025	0.002
Total		0.00406	0.002

Shear Stress Check - Wood	**Code Ref.**

$$v_j^{avg} = \frac{1}{M_s}\left(\frac{V_j}{A_w}\right) \quad (4\text{-}8)$$

$A_{w,NW}$ =	339	ft	*Horizontal cross-sectional area of all shear walls in direction x*
$A_{w,NE}$ =	139	ft	*Horizontal cross-sectional area of all shear walls in direction y*
V_{Base} =	377,728	lb	*Max Story Shear*
M_s =	3		*Modification Factor for Shear Walls*
v_x =	371.8	plf	*Shear Stress in Walls, x-dir*
v_y =	903.2	plf	*Shear Stress in Walls, y-dir*
v_{max} =	903.2	plf	*Shear Stress in Walls*
$v_{allowable}$ =	1000	plf	*Allowable Shear Stress in Walls*
DCR =	0.903	C	*Demand Capacity Ratio*

City Hall Tier 2 Evaluation
Life Safety

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ASCE 41-17 Linear Static Procedure (Sec. 7.4.1) - City Hall

Mapped Spectral Response Acceleration				Code Ref.
BSE-2E accel. @ short periods:	S_{S2E} =	1.081	g	OSHPD Seismic Maps
BSE-2E accel. @ a 1-sec. period:	S_{12E} =	0.362	g	OSHPD Seismic Maps
BSE-1E accel. @ short periods:	S_{S1E} =	0.501	g	OSHPD Seismic Maps
BSE-1E accel. @ a 1-sec. period:	S_{11E} =	0.155	g	OSHPD Seismic Maps
BSE-2N accel. @ short periods:	S_{S2N} =	1.466	g	OSHPD Seismic Maps
BSE-2N accel. @ a 1-sec. period:	S_{12N} =	0.499	g	OSHPD Seismic Maps
Site class:		D		
Long period transition parameter	T_L =	6	sec	
BSE-2E short period site coefficient:	F_{a2E} =	1.20		ASCE 7-16 Table 11.4-1
BSE-2E long period site coefficient:	F_{v2E} =	1.94		ASCE 7-16 Table 11.4-2
BSE-1E short period site coefficient:	F_{a1E} =	1.40		ASCE 7-16 Table 11.4-1
BSE-1E long period site coefficient:	F_{v1E} =	2.29		ASCE 7-16 Table 11.4-2
BSE-2N short period site coefficient:	F_{a2N} =	1.20		ASCE 7-16 Table 11.4-1
BSE-2N long period site coe	F_{v2N} =	1.80		ASCE 7-16 Table 11.4-2

Design Spectral Response Parameters (Sec. 2.4.1.6)				Code Ref.
BSE-2E controlling short period accel.	S_{S2E} = MIN(S_{S2E},S_{S2N}) =	1.081	g	2.4.1.3
BSE-2E controlling accel. @ T=1 s:	S_{12E} = MIN(S_{12E},S_{12N}) =	0.362	g	2.4.1.3
BSE-1E controlling short period accel.	S_{S1E} = MIN(S_{S1E},2/3*S_{S2N}) =	0.501	g	2.4.1.4
BSE-1E controlling accel. @ T=1 s:	S_{11E} = MIN(S_{11E},2/3*S_{12N}) =	0.155	g	2.4.1.4
BSE-2E design short period accel:	S_{XS2E} = F_{a2E}*S_{S2E} =	1.297	g	2.4.1.6
BSE-2E design 1 sec. period accel.:	S_{X12E} = F_{v2E}*S_{12E} =	0.702	g	2.4.1.6
BSE-1E design short period accel.:	S_{XS1E} = F_{a1E}*S_{S1E} =	0.701	g	2.4.1.6
BSE-1E design 1 sec. period accel.:	S_{X11E} = F_{v1E}*S_{11E} =	0.355	g	2.4.1.6

Level of Seismicity (Sec. 2.5)				Code Ref.
BSE-2N design short period accel:	S_{DS} = 2/3*F_{a2N}*S_{S2N} =	1.17	g	2.4.1.6
BSE-2N design 1 sec. period accel.:	S_{D1} = 2/3*F_{v2N}*S_{12N} =	0.60	g	2.4.1.6
Level of Seismicity:		**HIGH**		Table 2-4

LSP Structure Properties				Code Ref.
Building height:	h_n =	25.0	ft	
Effective damping ratio:	β =	5.00%		7.2.3.6
Lateral system:		Steel Moment Frame		7.4.1.2.2
Period coefficient:	C_t =	0.035		7.4.1.2.2
Period exponent:	β =	0.8		7.4.1.2.2
Empirical period:	T =	0.460	sec	7.4.1.2.2

Response Spectra Characteristic Periods				Code Ref.
BSE-2E spectra:	T_{S2} = S_{X12E}/S_{XS2E} =	0.54	sec	ASCE 7-16 Sec. 11.4.6
	T_{02} = 0.2*(S_{X12E}/S_{XS2E}) =	0.11	sec	ASCE 7-16 Sec. 11.4.6
BSE-1E spectra:	T_{S1} = S_{X11E}/S_{XS1E} =	0.51	sec	ASCE 7-16 Sec. 11.4.6
	T_{01} = 0.2*(S_{X11E}/S_{XS1E}) =	0.10	sec	ASCE 7-16 Sec. 11.4.6

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Project: City of Tukwila
Tukwila Seismic Evaluation
City Hall
Project No.: 262022.017

Sheet: of
Sheet: of
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Date:
Date:

ASCE 41-17 Linear Static Procedure (Sec. 7.4.1) - City Hall

Pseudo Seismic Force				Code Ref.
Building seismic weight:	$W =$	782	kip	7.4.1.3.1
Number of stories:	$n =$	2		
m_{max} @ BSE-2E:	$m_{max2} =$	3.5		7.4.1.3.1
m_{max} @ BSE-1E:	$m_{max1} =$	2.5		7.4.1.3.1
Damping coefficient:	$B_1 =$	1.00		2.4.1.7.1
BSE-2E mod. factors product:	$C_{12}C_{22} =$	1.1		Table 7-3
BSE-1E mod. factors product:	$C_{11}C_{21} =$	1.1		Table 7-3
Effective mass factor:	$C_m =$	1		Table 7-4
BSE-2E spectral acceleration:	$S_{a2} =$	1.29	g	2.4.3
BSE-1E spectral acceleration:	$S_{a1} =$	0.70	g	2.4.3
BSE-2E pseudo lateral load:	$V_{2E} = C_{12}C_{22}C_mS_{a2}W =$	1113.2	kip	7.4.1.3.1
BSE-1E pseudo lateral load:	$V_{1E} = C_{11}C_{21}C_mS_{a1}W =$	601.5	kip	7.4.1.3.1

Vertical Distribution of Seismic Forces (Sec. 7.4.1.3.2)			Code Ref.
Story force:	$F_x = w_xh_x^k/(\Sigma w_xh_x^k)*V =$	See Table Below	Eq. 7-24
Story heihgt exponent factor:	$k =$	1.00	7.4.1.3.2
Diaphragm force:	$F_{px} = V_x*w_x/W_x =$	See Table Below	Eq. 7-26

Story Name	Story Weight w_x (k)	Story Height h_x (ft)	$w_x*h_x^k$	BSE-2E Story Force F_{x2} (k)	BSE-1E Story Force F_{x1} (k)	BSE-2E Story Shear V_{x2} (k)	BSE-1E Story Shear V_{x1} (k)	Total Weight Above W_x (k)	BSE-2E Diaph. Force F_{px2} (k)	BSE-1E Diaph. Force F_{px1} (k)
Roof	139	25.0	3475	281.1	151.9	281.1	151.9	139.0	281.1	151.9
Level 2	643	16	10288	832.1	449.6	1113.2	601.5	782.0	915.3	494.6
SUM =	**782**		**13763**							

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Sheet: of
Sheet: of
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ASCE 41-17 Linear Static Procedure (Sec. 7.4.1) - City Hall

Acceleration Response Spectra

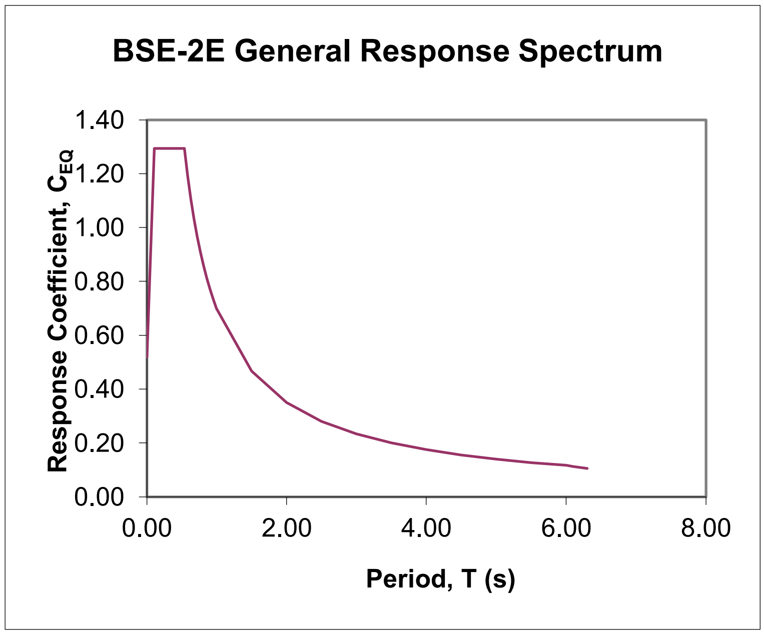
	BSE-2E		BSE-1E	
	T (sec)	C_{EQ}	T (sec)	C_{EQ}
	0.00	0.52	0.00	0.28
T_0 =	0.11	1.29	0.10	0.70
T_s =	0.54	1.29	0.51	0.70
	0.59	1.19	0.56	0.64
	0.63	1.11	0.61	0.59
	0.68	1.03	0.65	0.54
	0.72	0.97	0.70	0.50
	0.77	0.91	0.75	0.47
	0.82	0.86	0.80	0.44
	0.86	0.81	0.85	0.42
	0.91	0.77	0.90	0.39
	0.95	0.73	0.95	0.37
T_1 =	1	0.70	1	0.35
	2	0	2	0
	2	0	2	0
	3	0	3	0
	3	0	3	0
	4	0	4	0
	4	0	4	0
	5	0	5	0
	5	0	5	0
	6	0	6	0
T_L =	6	0	6	0
	6	0	6	0
	6	0	6	0
	6	0	6	0

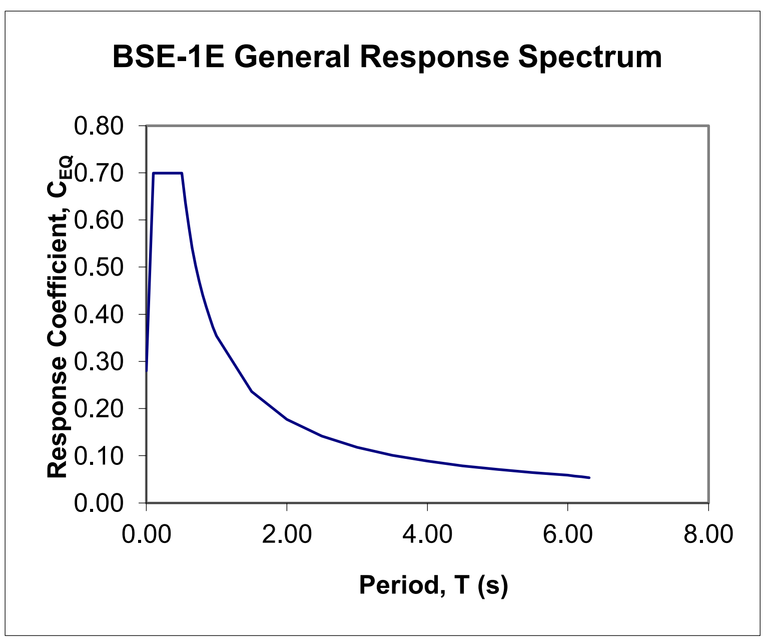
$C_{EQ} = C_1C_2C_MS_{XS}[(5/B_1\text{-}2)T/T_s+0.4$ - $\{@\ T < T_0\}$

$C_{EQ} = C_1C_2C_MS_{XS}/B_1$ - $\{@\ T_0 \le T \le T_S\}$

$C_{EQ} = C_1C_2C_MS_{X1}/(B_1{*}T)$ - $\{@\ T_s < T \le T_L\}$

$C_{EQ} = C_1C_2C_MT_LS_{X1}/(B_1{*}T^2)$ - $\{@\ T_L < T\}$





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City Hall Tier 2 Life Safety Calculations

Shear Stress Check - Concrete	**Code Ref.**

	V_{Base} =	1113.2	k	*Max Story Shear*	
	A_{Trib} =	1150	ft^2	*Tributary are to greatest stressed wall, x-direction (GL E, East wing)*	
	A_{Trib} =	870	ft^2	*Tributary are to greatest stressed wall, y-direction (GL 17, East wing)*	
	A_{Tot} =	14030	ft^2	*Total Floor Area*	
$Q_{UD,x}$ =	V_{Wall} =	91.2	k	*Tributary force to greatest stressed wall, x-direction*	
$Q_{UD,y}$ =	V_{Wall} =	69.0	k	*Tributary force to greatest stressed wall, x-direction*	
	$L_{wall,x}$ =	11.0	ft	*Wall Length, x-direction*	
	$L_{wall,y}$ =	8.5	ft	*Wall Length, y-direction*	
	$t_{Wall,x}$ =	8	in	*Wall Thickness, x-direction*	
	$t_{Wall,y}$ =	8	in	*Wall Thickness, y-direction*	
$Q_{CE,x}$ =	$V_{n,Wall,x}$ =	141.7	k	*Wall Shear Capacity, x-direction*	
$Q_{CE,y}$ =	$V_{n,Wall,y}$ =	109.5	k	*Wall Thickness, y-direction*	
	m =	2.5		*m-factor*	*ASCE 41-17 Table 10-22*
	k =	0.8		*knowledge factor*	*ASCE 41-17 Table 6-1*
	$mkQ_{CE,x}$ =	265.6			
	$mkQ_{CE,y}$ =	205.3			
	DCR =	0.343	C	*Demand Capacity Ratio*	

City Hall Tier 1 Evaluation
Collapse Prevention

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	City Hall	Date:	
		Checked By:	KRB
Project No.:	262022.017	Date:	

City Hall Tier 1 Calculations - Collapse Prevention

Building Properties

Code Ref.

Building Type:	C2a/W2	*Concrete & Wood Shear Walls w/ Flexible Diaphragms*	
Area:	14,030 ft^2		
Latitude:	47.463		
Longitude:	-122.256		
Site Class:	D (Default)		
No. Stories:	2		
Building Height:	25.00 ft	*(Approximate) Height of Sloped Roof*	
Risk Category:	II		
Level of Performance:	CP	*Collapse Prevention*	ASCE 41-17 Table 2.2

Table 2-2. Scope of Assessment Required for Tier 1 and Tier 2 with the Basic Performance Objective for Existing Buildings (BPOE)

Risk Category	Tier 1 and 2[a]: BSE-1E	Tier 1 and 2[a]: BSE-2E
I and II	Not evaluated	Collapse Prevention Structural Performance
	Life Safety Nonstructural Performance (3-C)	Hazards Reduced Nonstructural Performance[b] (5-D)
III	Not evaluated	Limited Safety Structural Performance[c]
	Position Retention Nonstructural Performance (2-B)	Hazards Reduced Nonstructural Performance[b] (4-D)
IV	Immediate Occupancy Structural Performance	Life Safety Structural Performance[d]
	Position Retention Nonstructural Performance (1-B)	Hazards Reduced Nonstructural Performance[b] (3-D)

[a] For Tier 1 and 2 assessments of Risk Categories I–III, Structural Performance for the BSE-1E is not explicitly evaluated.
[b] Compliance with ASCE 7 provisions for new construction is deemed to comply.
[c] For Risk Category III, the Tier 1 screening checklists shall be based on the Collapse Prevention Performance Level (S-5), except that checklist statements using the Quick Check procedures of Section 4.4.3 shall be based on M_s factors taken as the average of the values for Life Safety and Collapse Prevention.
[d] For Risk Category IV, the Tier 1 screening checklists shall be based on the Collapse Prevention Performance Level (S-5), except that checklist statements using the Quick Check procedures of Section 4.4.3 shall be based on M_s factors for Life Safety.

Seismic Properties, BSE-2E

			Code Ref.
Mapped Short Period Accel.:	S_S =	1.081 g	OSHPD Seismic Maps
Mapped One-Sec. Accel.:	S_1 =	0.362 g	OSHPD Seismic Maps
Accel. Site Coefficient:	F_a =	1.200	OSHPD Seismic Maps
Velocity Site Coefficient:	F_v =	1.938	OSHPD Seismic Maps
Design Short Period Accel.:	$S_{DS} = (2/3)*S_s*F_a$ =	0.865 g	ASCE 41-17 Eq. 2-4
Design 1-Sec. Period Accel.:	$S_{D1} = (2/3)*S_1*F_v$ =	0.468 g	ASCE 41-17 Eq. 2-5
Level of Seismicity:	High		
Seismic Hazard Level:	2E	*5% Probability of Exceedance in 50 Years for an Existing Building*	
BSE 1E Design Short Period Accel.:	S_{XS} =	1.297 g	OSHPD Seismic Maps
BSE 1E 1-Sec. Design Short Period Accel.:	S_{X1} =	0.701 g	OSHPD Seismic Maps

Design Spectral Acceleration, BSE-2E

				Code Ref.
Period Coefficient:	C_t =	0.020	*For All Other Framing System*	ASCE 41-17 S. 4.4.2.4
Period Coefficient:	β =	0.75	*For All Other Framing System*	ASCE 41-17 S. 4.4.2.4
Fundamental Period:	$T = C_t*h_n^{\beta}$ =	0.22 s		ASCE 41-17 Eq.4-4
Spectral Acc.:	$\mathbf{S_a = S_{X1}/T}$ =	**1.297 g**	*but S_a shall not exceed* S_{XS}	ASCE 41-17 Eq.4-3

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City Hall Tier 1 Calculations - Collapse Prevention

Weight Take-Off	Code Ref.

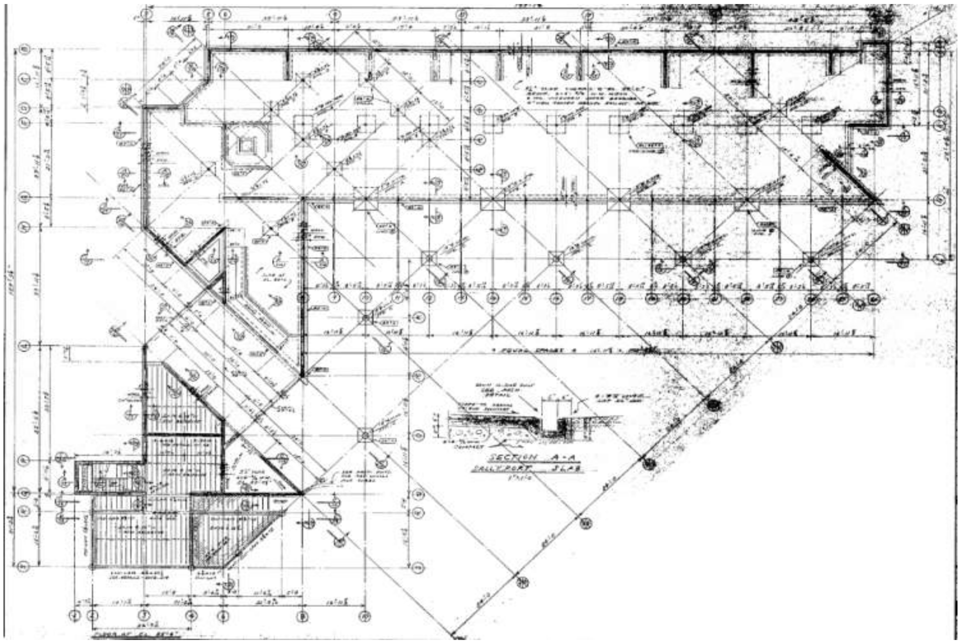

Figure 1: City Hall Foundation Plan

Building Weight Summary	
Roof	139 kip
Level 1	643 kip
Σ	782 kip

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City Hall Tier 1 Calculations - Collapse Prevention

Vertical Distribution of Psuedo-Seismic Base Shear

Code Ref.

Coefficient Exponent:	k =	1.0		ASCE 41-17 S. 4.4.2.2
Effective Seismic Building Weight:	W=	782 kips		
Modification Factor:	C =	1.2	*for CMU Buildings*	ASCE 41-17 Tbl. 4-7
Psuedo Seismic Base Shear, BSE-1E:	**V_{pseudo} = C*S_a*W =**	**1,217 kips**		

Story Shear Forces: Vertical Distribution of Pseudo Shear Forces

Floor Level [from base]	Height, h_x [ft]	Story Weight, w_x [kip]	$w_x h_x^k$ [kip*ft]	Dist. Factor C_{vx}	Lateral Force [kip]	Story Shear* [kip]
Roof	22.0	139	3,056	0.28	345	**345**
Level 2	12.0	643	7,714	0.72	871	**871**
Σ			10,770	1.0	1,217	

**Story shear will be used to check the SFRS in the structure at each respective level.*

Shear Stress Check - Concrete

Code Ref.

$$v_j^{avg} = \frac{1}{M_s}\left(\frac{V_j}{A_w}\right) \quad (4\text{-}8)$$

$A_{w,x}$ =	3363	in^2	*Horizontal cross-sectional area of all shear walls in direction*
$A_{w,y}$ =	2784	in^2	*Horizontal cross-sectional area of all shear walls in direction*
V_{roof} =	1,217	kip	*Max Story Shear*
M_s =	4.5		*Modification Factor for Shear Walls*
v_x =	80.4	psi	*Shear Stress in Walls, x-dir*
v_y =	97.1	psi	*Shear Stress in Walls, y-dir*
v_{max} =	97.1	psi	*Shear Stress in Walls*
$v_{allowable}$ =	100	psi	*Allowable Shear Stress in Walls*
DCR =	0.971	C	*Demand Capacity Ratio*

Shear Stress Check - Wood

Code Ref.

$$v_j^{avg} = \frac{1}{M_s}\left(\frac{V_j}{A_w}\right) \quad (4\text{-}8)$$

$A_{w,NW}$ =	339	ft	*Horizontal cross-sectional area of all shear walls in direction x*
$A_{w,NE}$ =	139	ft	*Horizontal cross-sectional area of all shear walls in direction y*
V_{Base} =	345,263	lb	*Max Story Shear*
M_s =	3		*Modification Factor for Shear Walls*
v_x =	339.8	plf	*Shear Stress in Walls, x-dir*
v_y =	825.6	plf	*Shear Stress in Walls, y-dir*
v_{max} =	825.6	plf	*Shear Stress in Walls*
$v_{allowable}$ =	1000	plf	*Allowable Shear Stress in Walls*
DCR =	0.826	C	*Demand Capacity Ratio*

6300 Building

17-2 Collapse Prevention Basic Configuration Checklist

Low Seismicity

Building System—General

C	NC	N/A	U	EVALUATION STATEMENT	COMMENT
X				LOAD PATH: The structure contains 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. *(Tier 2: Sec. 5.4.1.1; Commentary: Sec. A.2.1.10)*	
X				ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building is greater than 0.25% of the height of the shorter building in low seismicity, 0.5% in moderate seismicity, and 1.5% in high seismicity. *(Tier 2: Sec. 5.4.1.2; Commentary: Sec. A.2.1.2)*	
		X		MEZZANINES: Interior mezzanine levels are braced independently from the main structure or are anchored to the seismic-force-resisting elements of the main structure. *(Tier 2: Sec. 5.4.1.3; Commentary: Sec. A.2.1.3)*	

Building System—Building Configuration

C	NC	N/A	U	EVALUATION STATEMENT	COMMENT
X				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. *(Tier 2: Sec. 5.4.2.1; Commentary: Sec. A.2.2.2)*	
X				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. *(Tier 2: Sec. 5.4.2.2; Commentary: Sec. A.2.2.3)*	
	X			VERTICAL IRREGULARITIES: All vertical elements in the seismic-force-resisting system are continuous to the foundation. *(Tier 2: Sec. 5.4.2.3; Commentary: Sec. A.2.2.4)*	Wood shear walls on the upper floors are not continuous to the concrete foundation.
X				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. *(Tier 2: Sec. 5.4.2.4; Commentary: Sec. A.2.2.5)*	
X				MASS: There is no change in effective mass of more than 50% from one story to the next. Light roofs, penthouses, and mezzanines need not be considered. *(Tier 2: Sec. 5.4.2.5; Commentary: Sec. A.2.2.6)*	
		X		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. *(Tier 2: Sec. 5.4.2.6; Commentary: Sec. A.2.2.7)*	Building has a flexible diaphragm and is rectangular.

17-2 Collapse Prevention Basic Configuration Checklist

Moderate Seismicity

(Complete the Following Items in Addition to the Items for Low Seismicity)

Geologic Site Hazards

C	NC	N/A	U	EVALUATION STATEMENT	COMMENT
X				LIQUEFACTION: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance do not exist in the foundation soils at depths within 50 ft (15.2 m) under the building. *(Tier 2: Sec. 5.4.3.1; Commentary: Sec. A.6.1.1)*	Not a Site Class F site per 2008 Geotechncial report completed as part of original report.
			X	SLOPE FAILURE: The building site is located away from potential earthquake-induced slope failures or rockfalls so that it is unaffected by such failures or is capable of accommodating any predicted movements without failure. *(Tier 2: Sec. 5.4.3.1; Commentary: Sec. A.6.1.2)*	Building is located on a hillside site. Stability of the slope is unknown.
X				SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site are not anticipated. *(Tier 2: Sec. 5.4.3.1; Commentary: Sec. A.6.1.3)*	

High Seismicity

(Complete the Following Items in Addition to the Items for Low and Moderate Seismicity)

Foundation Configuration

C	NC	N/A	U	EVALUATION STATEMENT	COMMENT
	X			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 $0.6S_a$. *(Tier 2: Sec. 5.4.3.3; Commentary: Sec. A.6.2.1)*	80'/210' = 0.381 0.6Sa = 0.6(.701) = 0.421
X				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. *(Tier 2: Sec. 5.4.3.4; Commentary: Sec. A.6.2.2)*	Central columns not tied together are not part of the seismic force resisting system.

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

17-6. Collapse Prevention Structural Checklist for Building Type W2

Low and Moderate Seismicity

Seismic-Force-Resisting System

C	NC	N/A	U	EVALUATION STATEMENT	COMMENT
X				REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2. *(Tier 2: Sec. 5.5.1.1; Commentary: Sec. A.3.2.1.1)*	No shear walls in E/W direction but steel moment frames present.
	X			SHEAR STRESS CHECK: The shear stress in the shear walls, calculated using the Quick Check procedure of Section 4.4.3.3, is less than the following values: Structural panel sheathing, 1,000 lb/ft (14.6 kN/m); Diagonal sheathing, 700 lb/ft (10.2 kN/m); Straight sheathing, 100 lb/ft (1.5 kN/m); All other conditions, 100 lb/ft (1.5 kN/m). *(Tier 2: Sec. 5.5.3.1.1 ; Commentary: Sec.A.3.2.7.1)*	Shear stress check exceeds 1000 plf
X				STUCCO (EXTERIOR PLASTER) SHEAR WALLS: Multi-story buildings do not rely on exterior stucco walls as the primary seismic-force-resisting system. *(Tier 2: Sec. 5.5.3.6.1; Commentary: Sec. A.3.2.7.2)*	
X				GYPSUM WALLBOARD OR PLASTER SHEAR WALLS: Interior plaster or gypsum wallboard is not used for shear walls on buildings more than one story high with the exception of the uppermost level of a multi-story building. *(Tier 2: Sec. 5.5.3.6.1; Commentary: Sec. A.3.2.7.3)*	
X				NARROW WOOD SHEAR WALLS: Narrow wood shear walls with an aspect ratio greater than 2-to-1 are not used to resist seismic forces. *(Tier 2: Sec. 5.5.3.6.1; Commentary: Sec. A.3.2.7.4)*	
X				WALLS CONNECTED THROUGH FLOORS: Shear walls have an interconnection between stories to transfer overturning and shear forces through the floor. *(Tier 2: Sec. 5.5.3.6.2; Commentary: Sec. A.3.2.7.5)*	
		X		HILLSIDE SITE: For structures that are taller on at least one side by more than one-half story because of a sloping site, all shear walls on the downhill slope have an aspect ratio less than 1-to-2. *(Tier 2: Sec. 5.5.3.6.3; Commentary: Sec. A.3.2.7.6)*	Wood shearwalls only exist above grade.
		X		CRIPPLE WALLS: Cripple walls below first-floor-level shear walls are braced to the foundation with wood structural panels. *(Tier 2: Sec. 5.5.3.6.4; Commentary: Sec. A.3.2.7.7)*	Wood shear walls only exist above level 2
		X		OPENINGS: Walls with openings greater than 80% of the length are braced with wood structural panel shear walls with aspect ratios of not more than 1.5-to-1 or are supported by adjacent construction through positive ties capable of transferring the seismic forces. *(Tier 2: Sec. 5.5.3.6.5; Commentary: Sec. A.3.2.7.8)*	No openings in shear walls greater than 80%

Connections

C	NC	N/A	U	EVALUATION STATEMENT	COMMENT
		X		WOOD POSTS: There is a positive connection of wood posts to the foundation. *(Tier 2: Sec. 5.7.3.3; Commentary: Sec. A.5.3.3)*	Columns are 6” steel pipes
X				WOOD SILLS: All wood sills are bolted to the foundation. *(Tier 2: Sec. 5.7.3.3; Commentary: Sec. A.5.3.4)*	

X				GIRDER–COLUMN CONNECTION: There is a positive connection using plates, connection hardware, or straps between the girder and the column support. *(Tier 2: Sec. 5.7.4.1; Commentary: Sec. A.5.4.1)*	

High Seismicity

(Complete the Following Items in Addition to the Items for Low and Moderate Seismicity)

Connections

C	NC	N/A	U	EVALUATION STATEMENT	COMMENT
X				WOOD SILL BOLTS: Sill bolts are spaced at 6 ft or less with acceptable edge and end distance provided for wood and concrete. *(Tier 2: Sec. 5.7.3.3; Commentary: Sec. A.5.3.7)*	

Diaphragms

C	NC	N/A	U	EVALUATION STATEMENT	COMMENT
X				DIAPHRAGM CONTINUITY: The diaphragms are not composed of split-level floors and do not have expansion joints. *(Tier 2: Sec. 5.6.1.1; Commentary: Sec. A.4.1.1)*	
X				ROOF CHORD CONTINUITY: All chord elements are continuous, regardless of changes in roof elevation. *(Tier 2: Sec. 5.6.1.1; Commentary: Sec. A.4.1.3)*	
		X		DIAPHRAGM REINFORCEMENT AT OPENINGS: There is reinforcing around all diaphragm openings larger than 50% of the building width in either major plan dimension. *(Tier 2: Sec. 5.6.1.5; Commentary: Sec. A.4.1.8)*	No diaphragm openings larger than 50% of the building width.
		X		STRAIGHT SHEATHING: All straight-sheathed diaphragms have aspect ratios less than 1-to-1 in the direction being considered. *(Tier 2: Sec. 5.6.2; Commentary: Sec. A.4.2.1)*	Diaphragm consists of plywood sheathing
X				SPANS: All wood diaphragms with spans greater than 12 ft (3.6 m) consist of wood structural panels or diagonal sheathing. *(Tier 2: Sec. 5.6.2; Commentary: Sec. A.4.2.2)*	
		X		DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 30 ft (9.1 m) and aspect ratios less than or equal to 3-to-1. *(Tier 2: Sec. 5.6.2; Commentary: Sec. A.4.2.3)*	Diaphragm is blocked plywood sheathing.
X				OTHER DIAPHRAGMS: The diaphragms do not consist of a system other than wood, metal deck, concrete, or horizontal bracing. *(Tier 2: Sec. 5.6.5; Commentary: Sec. A.4.7.1)*	

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

17-8 Collapse Prevention Structural Checklist for Building Types S1 and S1a

Low Seismicity

Seismic-Force-Resisting System

C	NC	N/A	U	EVALUATION STATEMENT	COMMENT
	X			REDUNDANCY: The number of lines of moment frames in each principal direction is greater than or equal to 2. *(Tier 2: Sec. 5.5.1.1; Commentary: Sec. A.3.1.1.1)*	E/W direction utilized wood shear walls. N/S direction only has a single line of moment frames.
	X			DRIFT CHECK: The drift ratio of the steel moment frames, calculated using the Quick Check procedure of Section 4.4.3.1, is less than 0.030. *(Tier 2: Sec. 5.5.2.1.2; Commentary: Sec. A.3.1.3.1)*	Drift check exceeds 0.03
	X			COLUMN AXIAL STRESS CHECK: The axial stress caused by gravity loads in columns subjected to overturning forces is less than $0.10F_y$. Alternatively, the axial stress caused by overturning forces alone, calculated using the Quick Check procedure of Section 4.4.3.6, is less than $0.30F_y$. *(Tier 2: Sec. 5.5.2.1.3; Commentary: Sec. A.3.1.3.2)*	Column Axial stress exceeds 0.1Fy.
	X			FLEXURAL STRESS CHECK: The average flexural stress in the moment frame columns and beams, calculated using the Quick Check procedure of Section 4.4.3.9, is less than F_y. Columns need not be checked if the strong column– weak beam checklist item is compliant. *(Tier 2: Sec. 5.5.2.1.2; Commentary: Sec. A.3.1.3.3)*	

Connections

C	NC	N/A	U	EVALUATION STATEMENT	COMMENT
X				TRANSFER TO STEEL FRAMES: Diaphragms are connected for transfer of seismic forces to the steel frames. *(Tier 2: Sec. 5.7.2; Commentary: Sec. A.5.2.2)*	
X				STEEL COLUMNS: The columns in seismic-force-resisting frames are anchored to the building foundation. *(Tier 2: Sec. 5.7.3.1; Commentary: Sec. A.5.3.1)*	

Moderate Seismicity

(Complete the Following Items in Addition to the Items for Low Seismicity)

Seismic-Force-Resisting System

C	NC	N/A	U	EVALUATION STATEMENT	COMMENT
	X			REDUNDANCY: The number of bays of moment frames in each line is greater than or equal to 2. *(Tier 2: Sec. 5.5.1.1; Commentary: Sec. A.3.1.1.1)*	Only a single 2-bay frame in the x direction
		X		INTERFERING WALLS: All concrete and masonry infill walls placed in moment frames are isolated from structural elements. *(Tier 2: Sec. 5.5.2.1.1; Commentary: Sec. A.3.1.2.1)*	
X				MOMENT-RESISTING CONNECTIONS: All moment connections can develop the strength of the adjoining members based on the specified minimum yield stress of steel. *(Tier 2: Sec. 5.5.2.2.1; Commentary: Sec. A.3.1.3.4)*	

17-8 Collapse Prevention Structural Checklist for Building Types S1 and S1a

High Seismicity

(Complete the Following Items in Addition to the Items for Low and Moderate Seismicity)

Seismic-Force-Resisting System

C	NC	N/A	U	EVALUATION STATEMENT	COMMENT
X				MOMENT-RESISTING CONNECTIONS: All moment connections are able to develop the strength of the adjoining members or panel zones based on 110% of the expected yield stress of the steel in accordance with AISC 341, Section A3.2. *(Tier 2: Sec. 5.5.2.2.1; Commentary: Sec. A.3.1.3.4)*	
X				PANEL ZONES: All panel zones have the shear capacity to resist the shear demand required to develop 0.8 times the sum of the flexural strengths of the girders framing in at the face of the column. *(Tier 2: Sec. 5.5.2.2.2; Commentary: Sec. A.3.1.3.5)*	
		X		COLUMN SPLICES: All column splice details located in moment-resisting frames include connection of both flanges and the web. *(Tier 2: Sec. 5.5.2.2.3; Commentary: Sec. A.3.1.3.6)*	
	X			STRONG COLUMN—WEAK BEAM: The percentage of strong column–weak beam joints in each story of each line of moment frames is greater than 50%. *(Tier 2: Sec. 5.5.2.1.5; Commentary: Sec. A.3.1.3.7)*	
X				COMPACT MEMBERS: All frame elements meet section requirements in accordance with AISC 341, Table D1.1, for moderately ductile members. *(Tier 2: Sec. 5.5.2.2.4; Commentary: Sec. A.3.1.3.8)*	

Diaphragms (Stiff or Flexible)

C	NC	N/A	U	EVALUATION STATEMENT	COMMENT
		X		OPENINGS AT FRAMES: Diaphragm openings immediately adjacent to the moment frames extend less than 25% of the total frame length. *(Tier 2: Sec. 5.6.1.3; Commentary: Sec. A.4.1.5)*	

Flexible Diaphragms

C	NC	N/A	U	EVALUATION STATEMENT	COMMENT
X				CROSS TIES: There are continuous cross ties between diaphragm chords. *(Tier 2: Sec. 5.6.1.2; Commentary: Sec. A.4.1.2)*	
		X		STRAIGHT SHEATHING: All straight-sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered. *(Tier 2: Sec. 5.6.2; Commentary: Sec. A.4.2.1)*	All diaphragms are panel sheathing
X				SPANS: All wood diaphragms with spans greater than 24 ft (7.3 m) consist of wood structural panels or diagonal sheathing. *(Tier 2: Sec. 5.6.2; Commentary: Sec. A.4.2.2)*	
X				DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 ft (12.2 m) and aspect ratios less than or equal to 4-to-1. *(Tier 2: Sec. 5.6.2; Commentary: Sec. A.4.2.3)*	

17-8 Collapse Prevention Structural Checklist for Building Types S1 and S1a

C	NC	N/A	U	EVALUATION STATEMENT	COMMENT
X				OTHER DIAPHRAGMS: Diaphragms do not consist of a system other than wood, metal deck, concrete, or horizontal bracing. *(Tier 2: Sec. 5.6.5; Commentary: Sec. A.4.7.1)*	

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

17-24 Collapse Prevention Structural Checklist for Building Types C2 and C2a

Low and Moderate Seismicity

Seismic-Force-Resisting System

C	NC	N/A	U	EVALUATION STATEMENT	COMMENT
X				COMPLETE FRAMES: Steel or concrete frames classified as secondary components form a complete vertical-load-carrying system. *(Tier 2: Sec. 5.5.2.5.1; Commentary: Sec. A.3.1.6.1)*	
X				REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2. *(Tier 2: Sec.5.5.1.1; Commentary: Sec. A.3.2.1.1)*	
X				SHEAR STRESS CHECK: The shear stress in the concrete shear walls, calculated using the Quick Check procedure of Section 4.4.3.3, is less than the greater of 100 lb/in.^{2} (0.69 MPa) or $2\sqrt{f'_c}$. *(Tier 2: Sec.5.5.3.1.1; Commentary: Sec. A.3.2.2.1)*	
X				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. *(Tier 2: Sec.5.5.3.1.3; Commentary: Sec. A.3.2.2.2)*	

Connections

C	NC	N/A	U	EVALUATION STATEMENT	COMMENT
X				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 strength to resist the connection force calculated in the Quick Check procedure of Section 4.4.3.7. *(Tier 2: Sec.5.7.1.1; Commentary: Sec. A.5.1.1)*	
X				TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls. *(Tier 2: Sec.5.7.2; Commentary: Sec. A.5.2.1)*	
	X			FOUNDATION DOWELS: Wall reinforcement is doweled into the foundation with vertical bars equal in size and spacing to the vertical wall reinforcing directly above the foundation. *(Tier 2: Sec. 5.7.3.4; Commentary: Sec. A.5.3.5)*	Section A-6 on sheet S6 shows wall reinforcing shown as #6 @ unknown spacing with #3 @ 18" oc dowels.

High Seismicity

(Complete the Following Items in Addition to the Items for Low and Moderate Seismicity)

Seismic-Force-Resisting System

C	NC	N/A	U	EVALUATION STATEMENT	COMMENT
	X			DEFLECTION COMPATIBILITY: Secondary components have the shear capacity to develop the flexural strength of the components. *(Tier 2: Sec.5.5.2.5.2; Commentary: Sec. A.3.1.6.2)*	Columns do not have the shear capacity to develop their flexural strength.
		X		FLAT SLABS: Flat slabs or plates not part of the seismic-force-resisting system have continuous bottom steel through the column joints. *(Tier 2: Sec.5.5.2.5.3; Commentary: Sec. A.3.1.6.3)*	No flat slabs

17-24 Collapse Prevention Structural Checklist for Building Types C2 and C2a

C	NC	N/A	U	EVALUATION STATEMENT	COMMENT
		X		COUPLING BEAMS: The ends of both walls to which the coupling beam is attached are supported at each end to resist vertical loads caused by overturning. *(Tier 2: Sec.5.5.3.2.1; Commentary: Sec. A.3.2.2.3)*	No coupling beams

Diaphragms (Stiff or Flexible)

C	NC	N/A	U	EVALUATION STATEMENT	COMMENT
X				DIAPHRAGM CONTINUITY: The diaphragms are not composed of split-level floors and do not have expansion joints. *(Tier 2: Sec.5.6.1.1; Commentary: Sec. A.4.1.1)*	
		X		OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 25% of the wall length. *(Tier 2: Sec.5.6.1.3; Commentary: Sec. A.4.1.4)*	No diaphragm openings

Flexible Diaphragms

C	NC	N/A	U	EVALUATION STATEMENT	COMMENT
X				CROSS TIES: There are continuous cross ties between diaphragm chords. *(Tier 2: Sec.5.6.1.2; Commentary: Sec. A.4.1.2)*	
		X		STRAIGHT SHEATHING: All straight-sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered. *(Tier 2: Sec.5.6.2; Commentary: Sec. A.4.2.1)*	Diaphragm is structural panel sheathing
X				SPANS: All wood diaphragms with spans greater than 24 ft (7.3 m) consist of wood structural panels or diagonal sheathing. *(Tier 2: Sec.5.6.2; Commentary: Sec. A.4.2.2)*	
		X		DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 ft (12.2 m) and aspect ratios less than or equal to 4-to-1. *(Tier 2: Sec.5.6.2; Commentary: Sec. A.4.2.3)*	Diaphragm is structural panel sheathing
X				OTHER DIAPHRAGMS: Diaphragms do not consist of a system other than wood, metal deck, concrete, or horizontal bracing. *(Tier 2: Sec.5.6.5; Commentary: Sec. A.4.7.1)*	

Connections

C	NC	N/A	U	EVALUATION STATEMENT	COMMENT
		X		UPLIFT AT PILE CAPS: Pile caps have top reinforcement, and piles are anchored to the pile caps. *(Tier 2: Sec.5.7.3.5; Commentary: Sec. A.5.3.8)*	Building foundation does not utilize pile caps

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

Design Criteria

ReidMiddleton
728 134th St. SW, Suite 200
Everett, WA 98204
425-741-3800
www.reidmiddleton.com

DESIGN SUMMARY

The 6300 building is 3 stories on a sloped grade sloping from the second floor on the north side down to the first floor on the south side. At the first floor the building is a concrete parking garage with concrete columns and retaining and shear walls. Starting at the second floor and going up the building is wood framed. The floor is constructed of plywood supported by open web joists spanning between glulam beams running east to west. The slab between grids 1 and 2 at the north end of the building are concrete topping over hollow concrete planks. The lateral system of the building is comprised of wood shear walls in the transverse direction of the building and a 2-bay steel moment frame in the longitudinal direction. A single concrete shear wall extends up the entire height of the elevator shaft and provides lateral resistance as well.

CODES AND REFERENCES

General
- ASCE 41-17 Minimum Design Loads for Buildings and Other Structures

Concrete
- ACI 318-14 Building Code Requirements for Structural Concrete

Wood
- ANSI/AF&PA-2015 National Design Specification for Wood Construction
- AITC Timber Construction Manual, Sixth Edition

Steel
- AISC 325-11 Steel Construction Manual, 14th Edition (2011)

Catalogs and Miscellaneous
- Trus-Joist MacMillan Catalog
- Hilti Catalog
- Simpson Strong-Tie Catalog
- Red-Built Open-Web Truss Catalog
- Red-Built Red-I Joist Catalog

Tukwila City Hall

6200 Southcenter Blvd, Tukwila, WA 98188, USA

Latitude, Longitude: 47.463224, -122.2555133

Date	3/29/2022, 10:02:06 AM
Design Code Reference Document	ASCE41-17
Custom Probability	
Site Class	D - Default (See Section 11.4.3)

Type	**Description**	**Value**
Hazard Level		BSE-2N
S_S	spectral response (0.2 s)	1.466
S_1	spectral response (1.0 s)	0.499
S_{XS}	site-modified spectral response (0.2 s)	1.76
S_{X1}	site-modified spectral response (1.0 s)	0.898
F_a	site amplification factor (0.2 s)	1.2
F_v	site amplification factor (1.0 s)	1.801
ssuh	max direction uniform hazard (0.2 s)	1.629
crs	coefficient of risk (0.2 s)	0.9
ssrt	risk-targeted hazard (0.2 s)	1.466
ssd	deterministic hazard (0.2 s)	4.288
s1uh	max direction uniform hazard (1.0 s)	0.557
cr1	coefficient of risk (1.0 s)	0.896
s1rt	risk-targeted hazard (1.0 s)	0.499
s1d	deterministic hazard (1.0 s)	1.501

Type	**Description**	**Value**
Hazard Level		BSE-1N
S_{XS}	site-modified spectral response (0.2 s)	1.173
S_{X1}	site-modified spectral response (1.0 s)	0.599

Type	Description	Value
Hazard Level		BSE-2E
S_S	spectral response (0.2 s)	1.081
S_1	spectral response (1.0 s)	0.362
S_{XS}	site-modified spectral response (0.2 s)	1.297
S_{X1}	site-modified spectral response (1.0 s)	0.701
f_a	site amplification factor (0.2 s)	1.2
f_v	site amplification factor (1.0 s)	1.938

Type	Description	Value
Hazard Level		BSE-1E
S_S	spectral response (0.2 s)	0.501
S_1	spectral response (1.0 s)	0.155
S_{XS}	site-modified spectral response (0.2 s)	0.701
S_{X1}	site-modified spectral response (1.0 s)	0.355
F_a	site amplification factor (0.2 s)	1.399
F_v	site amplification factor (1.0 s)	2.29

Type	Description	Value
Hazard Level		TL Data
T-Sub-L	Long-period transition period in seconds	6

6300 Building Tier 1 Evaluation
Life Safety

ReidMiddleton

728 134th St. SW, Suite 200
Everett, WA 98204
425-741-3800
www.reidmiddleton.com

6300 Tier 1 Calculations - Life Safety

Building Properties	Code Ref.

Building Type: C2a/W2 *Concrete & Wood Shear Walls w/ Flexible Diaphragms*
Area: 16,800 ft^2
Latitude: 47.463
Longitude: -122.256
Site Class: D (Default)

No. Stories: 3
Building Height: 41.50 ft *(Approximate) Height of Sloped Roof*

Risk Category: II
Level of Performance: LS *Life Safety*

Risk Category	Scope item	Evaluation Performance Objective[2,4]	Retrofit Performance Objective[4]
I or II	Structural	Life Safety in BSE-1E	Life Safety in BSE-1N and Collapse Prevention in BSE-2N
	Nonstructural[1]	Life Safety in BSE-1E	Life Safety in BSE-1N
III	Structural	Damage Control in BSE-1E[3]	Damage Control in BSE-1N and Limited Safety in BSE-2N
	Nonstructural[1]	Life Safety in BSE-1N	Life Safety in BSE-1N
IV	Structural	Immediate Occupancy in BSE-1E	Immediate Occupancy in BSE-1N and Life Safety in BSE-2N
	Nonstructural[1]	Position Retention in BSE-1E	Operational in BSE-1N

[1] At the AHJ's discretion, the Nonstructural scope may be waived in areas of the building not affected by the project and not affecting DoD operations, safety, or post-earthquake occupancy.
[2] At the AHJ's discretion, Tier 3 evaluation at the BSE-2E hazard level may also be required, consistent with ASCE/SEI 41-13 Table 2-1.
[3] Tier 1 or Tier 2 evaluation at the Damage Control level must use the Tier 1 checklists and Tier 2 procedures for Life Safety performance, but M_s-factors and other quantitative limits must be taken as the average of Life Safety and Immediate Occupancy values.
[4] See ASCE41-13 for definitions of BSE-1E, BSE-1N, and BSE-2N.

Code Ref.: UFC 3-301-01 Section 4-2.1.1

Seismic Properties, BSE-1E			Code Ref.
Mapped Short Period Accel.:	S_S =	0.501 g	OSHPD Seismic Maps
Mapped One-Sec. Accel.:	S_1 =	0.155 g	OSHPD Seismic Maps
Accel. Site Coefficient:	F_a =	1.399	OSHPD Seismic Maps
Velocity Site Coefficient:	F_v =	2.290	OSHPD Seismic Maps
Design Short Period Accel.:	$S_{DS} = (2/3)*S_s*F_a$ =	0.467 g	ASCE 41-17 Eq. 2-4
Design 1-Sec. Period Accel.:	$S_{D1} = (2/3)*S_1*F_v$ =	0.237 g	ASCE 41-17 Eq. 2-5
Level of Seismicity:	High		
Seismic Hazard Level:	1E	*20% Probability of Exceedance in 50 Years for an Existing Building*	
BSE 2E Design Short Period Accel.:	S_{XS} =	0.701 g	OSHPD Seismic Maps
BSE 2E 1-Sec. Design Short Period Accel.:	S_{X1} =	0.355 g	OSHPD Seismic Maps

Design Spectral Acceleration, BSE-1E				Code Ref.
Period Coefficient:	C_t =	0.020	*For All Other Framing System*	ASCE 41-17 S. 4.4.2.4
Period Coefficient:	β =	0.75	*For All Other Framing System*	ASCE 41-17 S. 4.4.2.4
Fundamental Period:	$T = C_t*h_n^{\beta}$ =	0.33 s		ASCE 41-17 Eq.4-4
Spectral Acc.:	$\mathbf{S_a = S_{X1}/T}$ =	**0.701 g**	*but S_a shall not exceed* S_{XS}	ASCE 41-17 Eq.4-3

6300 Tier 1 Calculations - Life Safety

Weight Take-Off	**Code Ref.**

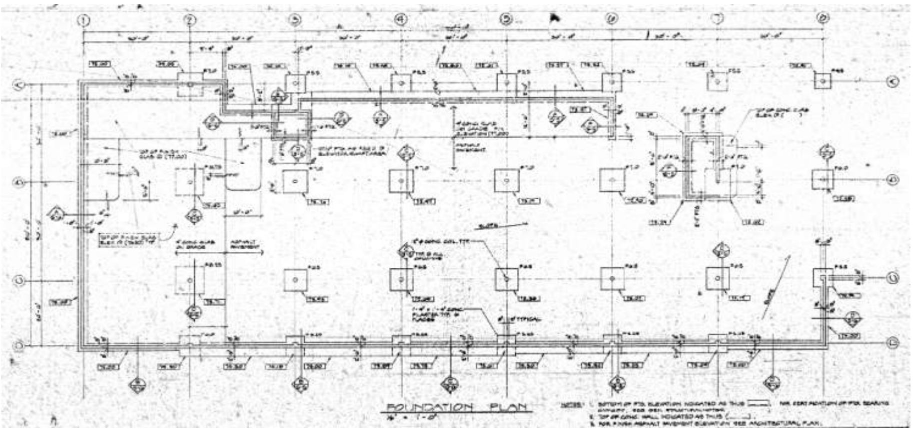

Figure 1: Building 6300 Foundation Plan

Ground Floor

8" Conc Wall	330.1	kip				
12" Conc Col	12.2	kip				

First Floor

32" TJI 50 @ 2' oc	9.0	psf	14400	sf	129.6	kip
3/4" Plywd	3	psf	14400	sf	43.2	kip
1.5" Lt Wt Conc topping	13	psf	14400	sf	180	kip
12" spandeck	124	psf	2400	sf	297	kip
2" topping	25	psf	2400	sf	60	kip
Misc	5	psf	16800	sf	84	kip
	793.8	kip				

Second Floor

32" TJI 50 @ 2' oc	9.0	psf	14400	sf	129.6	kip
3/4" Plywd	3	psf	14400	sf	43.2	kip
1.5" Lt Wt Conc topping	13	psf	14400	sf	180	kip
8" spandeck	83	psf	2400	sf	198	kip
2" topping	25	psf	2400	sf	60	kip
Misc	5	psf	16800	sf	84	kip
	694.8	kip				

Roof

28" TJL @ 48" oc	3	psf
3/4" Plywd	3	psf
GL 6.75x25.5	1.4	psf
Misc	5.0	psf
	208.9	kip

Building Weight Summary		
Roof	209	kip
Level 2	695	kip
Level 1	1,136	kip
Σ	2,040	kip

Vertical Distribution of Psuedo-Seismic Base Shear	**Code Ref.**

Coefficient Exponent:	k =	1.0			ASCE 41-17 S. 4.4.2.2
Effective Seismic Building Weight:	W=	2,040	kips		
Modification Factor:	C =	1.1		*for Shear walls*	ASCE 41-17 Tbl. 4-7
Psuedo Seismic Base Shear, BSE-2E:	$V_{pseudo} = C{*}S_a{*}W =$	**1,573**	**kips**		

Story Shear Forces: Vertical Distribution of Psuedo Shear Forces

ReidMiddleton
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Client: City of Tukwila
Project: Tukwila Seismic Evaluation
6300 Building
Project No.: 262022.017

Sheet: of
Design By: MLO
Date:
Checked By: KRB
Date:

6300 Tier 1 Calculations - Life Safety

Floor Level [from base]	Height, h_x [ft]	Story Weight, w_x [kip]	$w_x h_x^k$ [kip*ft]	Dist. Factor C_{vx}	Lateral Force [kip]	Story Shear* [kip]
Roof	41.5	209	8,669	0.21	334	**334**
Level 2	27.5	695	19,107	0.47	736	**736**
Level 1	11.5	1,136	13,064	0.32	503	**503**
Σ		2,040	40,841	1.0	1,573	

**Story shear will be used to check the SFRS in the structure at each respective level.*

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6300 Tier 1 Calculations - Life Safety

Shear Stress Check - Concrete | **Code Ref.**

$$v_j^{avg} = \frac{1}{M_s}\left(\frac{V_j}{A_w}\right) \qquad (4\text{-}8)$$

$A_{w,x}$ =	34560	in^2	*Horizontal cross-sectional area of all shear walls in direction x*
$A_{w,y}$ =	16672	in^2	*Horizontal cross-sectional area of all shear walls in direction y*
V_{Base} =	1,573	kip	*Max Story Shear*
M_s =	3		*Modification Factor for Shear Walls*
v_x =	15.2	psi	*Shear Stress in Walls, x-dir*
v_y =	31.4	psi	*Shear Stress in Walls, y-dir*
v_{max} =	31.4	psi	*Shear Stress in Walls*
$v_{allowable}$ =	100	psi	*Allowable Shear Stress in Walls*
DCR =	0.314	C	*Demand Capacity Ratio*

Reinforcing Steel in Shear Walls | **Code Ref.**

City Hall	Reinforcing ratio, ρ	$\rho_{provided}$	$\rho_{required}$
Vertical	#5 @ 12" oc	0.00323	0.0012
Horizontal	#4 @ 12" oc	0.00208	0.002
Total		0.00531	0.002

Wall Anchorage Check | **Code Ref.**

$$T_c = \psi S_{XS} w_p A_p \qquad (4\text{-}12)$$

ψ =	1.3		*CP = 1.0; LS = 1.3; IO = 1.8*
S_{XS} =	0.701	g	*Spectral Response Acceleration*
w_p =	100	psf	*Unit Weight of Wall*
A_p =	24	ft^2	*Area of Wall Tributary to Connection*
T_c =	2187	lb	*Connection Demand*
T_n =	12000	lb	*Connection Capacity (#4 @ 12" oc)*
DCR =	0.182	C	

Shear Stress Check - Wood | **Code Ref.**

$$v_j^{avg} = \frac{1}{M_s}\left(\frac{V_j}{A_w}\right) \qquad (4\text{-}8)$$

$A_{w,y}$ =	124	ft	*Horizontal cross-sectional area of all shear walls in direction y*
V_{Floor} =	333,867	lb	*Max Story Shear*
M_s =	3		*Modification Factor for Shear Walls*
v_y =	639.8	plf	*Service Level Shear Stress in Walls, y-dir*
v_{max} =	639.8	plf	*Service Level Shear Stress in Walls*
$v_{allowable}$ =	1000	plf	*Allowable Shear Stress in Walls*
DCR =	0.640	C	*Demand Capacity Ratio*

Drift Check | **Code Ref.**

$$D_r = \left(\frac{k_b + k_c}{k_b k_c}\right)\left(\frac{h}{12E}\right)V_c \qquad (4\text{-}6)$$

D_r =	0.07255	*Drift ratio*
k_b =	1.08611	*I/L for the representative beam*
k_c =	2.32738	*I/h for the representative column*

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6300 Tier 1 Calculations - Life Safety

I =	391 in^4	W12x50	*Column moment of Inertia (in^4)*
I =	391 in^4	W12x50	*Beam moment of Inertia (in^4)*
L =	360 in		*Beam length*
h =	168 in		*Story Height*
E =	29000 ksi		*Modulus of elasticity (ksi)*
V_c =	111.289 kip		*Shear in the column*

$D_{r,\ Allowable}$ = 0.03

DCR_{Drift} 2.41834

Column Axial Stress Check — Code Ref.

$$p_{ot} = \frac{1}{M_s}\left(\frac{2}{3}\right)\left(\frac{Vh_n}{Ln_f}\right)\left(\frac{1}{A_{col}}\right) \qquad (4\text{-}11)$$

M_s =	1.5	*System modification factor (CP = 2.5, LS = 1.5, IO = 1.0)*
V =	334 kip	*Pseudo seismic force*
h_n =	30 ft	*Height above the base to roof*
L =	60 ft	*Total Length of the frame*
N_f =	1	*Number of frames in the direction of loading*
A_{col} =	14.6 in^2	*Area of end column of the frame*
p_{ot} =	5.08169	*Axial stress of columns*

$0.1F_y$ = 3.6 ksi

DCR = 1.41

Frame Flexural Stress — Code Ref.

$$f_j^{avg} = V_j \frac{1}{M_s}\left(\frac{n_c}{n_c - n_f}\right)\left(\frac{h}{2}\right)\frac{1}{Z} \qquad (4\text{-}14)$$

V_j =	334 kip	*Story Shear*
M_s =	6	*System modification factor (CP = 9, LS = 6, IO = 2.5)*
n_c =	30	*Number of frame Columns*
n_f =	1	*Number of frames*
h =	168 in	*Story Height*
Z =	71.9 in^3	*Plastic Section of Beams*
f_j^{avg} =	67.3 ksi	*Axial stress of columns*

F_y = 36 ksi — *Beam Yield Stress*

DCR = 1.87

6300 Building Tier 1 Evaluation
Collapse Prevention

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Tukwila Seismic Evaluation

City of Tukwila

6300 Building Tier 2 Evaluation

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6300 Building Tier 1 Calculations - Collapse Prevention

Building Properties			Code Ref.
Building Type:	C2a/W2	*Concrete & Wood Shear Walls w/ Flexible Diaphragms*	
Area:	14,030 ft^2		
Latitude:	47.463		
Longitude:	-122.256		
Site Class:	D (Default)		
No. Stories:	2		
Building Height:	25.00 ft	*(Approximate) Height of Sloped Roof*	
Risk Category:	II		
Level of Performance:	CP	*Collapse Prevention*	UFC 3-301-01 Section 4-2.1.1

Risk Category	Scope item	Evaluation Performance Objective[2,4]	Retrofit Performance Objective[4]
I or II	Structural	Life Safety in BSE-1E	Life Safety in BSE-1N and Collapse Prevention in BSE-2N
	Nonstructural[1]	Life Safety in BSE-1E	Life Safety in BSE-1N
III	Structural	Damage Control in BSE-1E[3]	Damage Control in BSE-1N and Limited Safety in BSE-2N
	Nonstructural[1]	Life Safety in BSE-1N	Life Safety in BSE-1N
IV	Structural	Immediate Occupancy in BSE-1E	Immediate Occupancy in BSE-1N and Life Safety in BSE-2N
	Nonstructural[1]	Position Retention in BSE-1E	Operational in BSE-1N

[1] At the AHJ's discretion, the Nonstructural scope may be waived in areas of the building not affected by the project and not affecting DoD operations, safety, or post-earthquake occupancy.
[2] At the AHJ's discretion, Tier 3 evaluation at the BSE-2E hazard level may also be required, consistent with ASCE/SEI 41-13 Table 2-1.
[3] Tier 1 or Tier 2 evaluation at the Damage Control level must use the Tier 1 checklists and Tier 2 procedures for Life Safety performance, but M_s-factors and other quantitative limits must be taken as the average of Life Safety and Immediate Occupancy values.
[4] See ASCE41-13 for definitions of BSE-1E, BSE-1N, and BSE-2N.

Seismic Properties, BSE-2E				Code Ref.
Mapped Short Period Accel.:		S_S =	1.081 g	OSHPD Seismic Maps
Mapped One-Sec. Accel.:		S_1 =	0.362 g	OSHPD Seismic Maps
Accel. Site Coefficient:		F_a =	1.200	OSHPD Seismic Maps
Velocity Site Coefficient:		F_v =	1.938	OSHPD Seismic Maps
Design Short Period Accel.:		$S_{DS} = (2/3)*S_s*F_a$ =	0.865 g	ASCE 41-17 Eq. 2-4
Design 1-Sec. Period Accel.:		$S_{D1} = (2/3)*S_1*F_v$ =	0.468 g	ASCE 41-17 Eq. 2-5
Level of Seismicity:	High			
Seismic Hazard Level:	2E	*5% Probability of Exceedance in 50 Years for an Existing Building*		
BSE 1E Design Short Period Accel.:		S_{XS} =	1.297 g	OSHPD Seismic Maps
BSE 1E 1-Sec. Design Short Period Accel.:		S_{X1} =	0.701 g	OSHPD Seismic Maps

Design Spectral Acceleration, BSE-2E				Code Ref.
Period Coefficient:	C_t =	0.020	*For All Other Framing System*	ASCE 41-17 S. 4.4.2.4
Period Coefficient:	β =	0.75	*For All Other Framing System*	ASCE 41-17 S. 4.4.2.4
Fundamental Period:	$T = C_t*h_n^{\beta}$ =	0.22 s		ASCE 41-17 Eq.4-4
Spectral Acc.:	$\mathbf{S_a = S_{X1}/T}$ =	**1.297 g**	*but S_a shall not exceed* S_{XS}	ASCE 41-17 Eq.4-3

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Client: City of Tukwila
Project: Tukwila Seismic Evaluation
6300 Building
Project No.: 262022.017

Sheet: of
Design By: MLO
Date:
Checked By: KRB
Date:

6300 Building Tier 1 Calculations - Collapse Prevention

Weight Take-Off	Code Ref.

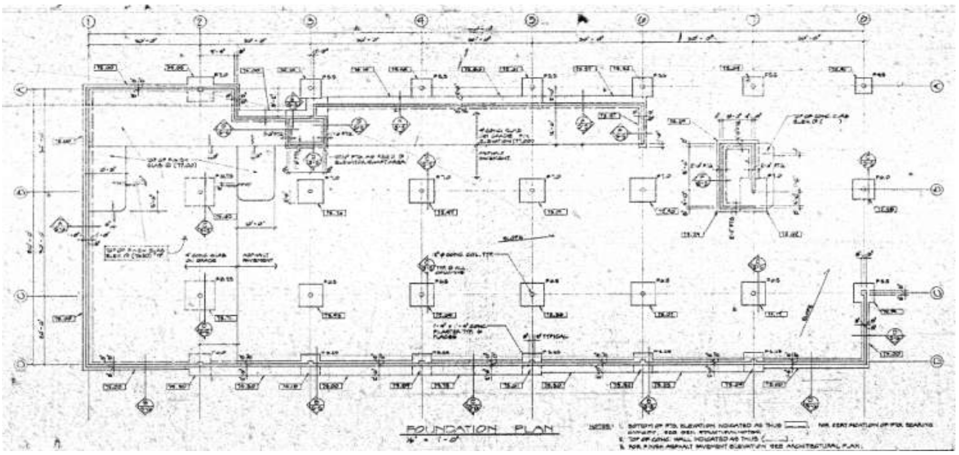

Figure 1: 6300 Building Foundation Plan

Building Weight Summary	
Roof	209 kip
Level 2	695 kip
Level 1	1,136 kip
Σ	2,040 kip

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6300 Building Tier 1 Calculations - Collapse Prevention

Vertical Distribution of Psuedo-Seismic Base Shear — Code Ref.

				Code Ref.
Coefficient Exponent:	k =	1.0		ASCE 41-17 S. 4.4.2.2
Effective Seismic Building Weight:	W=	2,040 kips		
Modification Factor:	C =	1.1	*for Shear walls*	ASCE 41-17 Tbl. 4-7
Psuedo Seismic Base Shear, BSE-1E:	V_{pseudo} = C*S_a*W =	**2,910 kips**		

Story Shear Forces: Vertical Distribution of Pseudo Shear Forces						
Floor Level [from base]	Height, h_x [ft]	Story Weight, w_x [kip]	$w_x h_x^k$ [kip*ft]	Dist. Factor C_{vx}	Lateral Force [kip]	Story Shear* [kip]
Roof	41.5	209	8,669	0.21	618	**618**
Level 2	27.5	695	19,107	0.47	1,361	**1,361**
Level 1	11.5	1,136	13,064	0.32	931	**931**
Σ			40,841	1.0	2,910	

**Story shear will be used to check the SFRS in the structure at each respective level.*

Shear Stress Check - Concrete — Code Ref.

$$v_j^{avg} = \frac{1}{M_s}\left(\frac{V_j}{A_w}\right) \qquad (4\text{-}8)$$

$A_{w,x}$ =	34560 in^2	*Horizontal cross-sectional area of all shear walls in direction*
$A_{w,y}$ =	16672 in^2	*Horizontal cross-sectional area of all shear walls in direction*
V_{Base} =	2,910 kip	*Max Story Shear*
M_s =	4.5	*Modification Factor for Shear Walls*
v_x =	18.7 psi	*Shear Stress in Walls, x-dir*
v_y =	38.8 psi	*Shear Stress in Walls, y-dir*
v_{max} =	38.8	*Shear Stress in Walls*
$v_{allowable}$ =	100	*Allowable Shear Stress in Walls*
DCR =	0.388 C	*Demand Capacity Ratio*

6300 Building Tier 1 Calculations - Collapse Prevention

Wall Anchorage Check	Code Ref.

$$T_c = \psi S_{XS} w_p A_p \qquad (4\text{-}12)$$

ψ =	1		*CP = 1.0; LS = 1.3; IO = 1.8*
S_{XS} =	1.297	g	*Spectral Response Acceleration*
w_p =	100	psf	*Unit Weight of Wall*
A_p =	24	ft^2	*Area of Wall Tributary to Connection*
T_c =	3113	lb	*Connection Demand*
T_n =	12000	lb	*Connection Capacity (#4 @ 12" oc)*
DCR =	0.259	C	

Shear Stress Check - Wood	Code Ref.

$$v_j^{avg} = \frac{1}{M_s}\left(\frac{V_j}{A_w}\right) \qquad (4\text{-}8)$$

$A_{w,y}$ =	124	ft	*Horizontal cross-sectional area of all shear walls in direction y*
V_{Floor} =	617,725	lb	*Max Story Shear*
M_s =	3		*Modification Factor for Shear Walls*
v_y =	1183.7	plf	*Service Level Shear Stress in Walls, y-dir*
v_{max} =	1183.7	plf	*Service Level Shear Stress in Walls*
$v_{allowable}$ =	1000	plf	*Allowable Shear Stress in Walls*
DCR =	1.184	NC	*Demand Capacity Ratio*

Drift Check	Code Ref.

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Project: City of Tukwila
Tukwila Seismic Evaluation
6300 Building
Project No.: 262022.017

Sheet: of
Sheet: of
Design By: MLO
Date:
Date:

ASCE 41-17 Linear Static Procedure (Sec. 7.4.1) - 6300 Building

Mapped Spectral Response Acceleration				**Code Ref.**
BSE-2E accel. @ short periods:	S_{S2E} =	1.081	g	OSHPD Seismic Maps
BSE-2E accel. @ a 1-sec. period:	S_{12E} =	0.362	g	OSHPD Seismic Maps
BSE-1E accel. @ short periods:	S_{S1E} =	0.501	g	OSHPD Seismic Maps
BSE-1E accel. @ a 1-sec. period:	S_{11E} =	0.155	g	OSHPD Seismic Maps
BSE-2N accel. @ short periods:	S_{S2N} =	1.466	g	OSHPD Seismic Maps
BSE-2N accel. @ a 1-sec. period:	S_{12N} =	0.499	g	OSHPD Seismic Maps
Site class:		D		
Long period transition parameter	T_L =	6	sec	
BSE-2E short period site coefficient:	F_{a2E} =	1.20		ASCE 7-16 Table 11.4-1
BSE-2E long period site coefficient:	F_{v2E} =	1.94		ASCE 7-16 Table 11.4-2
BSE-1E short period site coefficient:	F_{a1E} =	1.40		ASCE 7-16 Table 11.4-1
BSE-1E long period site coefficient:	F_{v1E} =	2.29		ASCE 7-16 Table 11.4-2
BSE-2N short period site coefficient:	F_{a2N} =	1.20		ASCE 7-16 Table 11.4-1
BSE-2N long period site coe	F_{v2N} =	1.80		ASCE 7-16 Table 11.4-2

Design Spectral Response Parameters (Sec. 2.4.1.6)				**Code Ref.**
BSE-2E controlling short period accel	S_{S2E} = MIN(S_{S2E},S_{S2N}) =	1.081	g	2.4.1.3
BSE-2E controlling accel. @ T=1 s:	S_{12E} = MIN(S_{12E},S_{12N}) =	0.362	g	2.4.1.3
BSE-1E controlling short period accel	S_{S1E} = MIN(S_{S1E},2/3*S_{S2N}) =	0.501	g	2.4.1.4
BSE-1E controlling accel. @ T=1 s:	S_{11E} = MIN(S_{11E},2/3*S_{12N}) =	0.155	g	2.4.1.4
BSE-2E design short period accel:	S_{XS2E} = F_{a2E}*S_{S2E} =	1.297	g	2.4.1.6
BSE-2E design 1 sec. period accel.:	S_{X12E} = F_{v2E}*S_{12E} =	0.702	g	2.4.1.6
BSE-1E design short period accel.:	S_{XS1E} = F_{a1E}*S_{S1E} =	0.701	g	2.4.1.6
BSE-1E design 1 sec. period accel.:	S_{X11E} = F_{v1E}*S_{11E} =	0.355	g	2.4.1.6

Level of Seismicity (Sec. 2.5)				**Code Ref.**
BSE-2N design short period accel:	S_{DS} = 2/3*F_{a2N}*S_{S2N} =	1.17	g	2.4.1.6
BSE-2N design 1 sec. period accel.:	S_{D1} = 2/3*F_{v2N}*S_{12N} =	0.60	g	2.4.1.6
Level of Seismicity:		**HIGH**		Table 2-4

LSP Structure Properties				**Code Ref.**
Building height:	h_n =	25.0	ft	
Effective damping ratio:	β =	5.00%		7.2.3.6
Lateral system:		Concrete Shear Wall		7.4.1.2.2
Period coefficient:	C_t =	0.02		7.4.1.2.2
Period exponent:	β =	0.75		7.4.1.2.2
Empirical period:	T =	0.224	sec	7.4.1.2.2

Response Spectra Characteristic Periods				**Code Ref.**
BSE-2E spectra:	T_{S2} = S_{X12E}/S_{XS2E} =	0.54	sec	ASCE 7-16 Sec. 11.4.6
	T_{02} = 0.2*(S_{X12E}/S_{XS2E}) =	0.11	sec	ASCE 7-16 Sec. 11.4.6
BSE-1E spectra:	T_{S1} = S_{X11E}/S_{XS1E} =	0.51	sec	ASCE 7-16 Sec. 11.4.6
	T_{01} = 0.2*(S_{X11E}/S_{XS1E}) =	0.10	sec	ASCE 7-16 Sec. 11.4.6

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ASCE 41-17 Linear Static Procedure (Sec. 7.4.1) - 6300 Building

Pseudo Seismic Force				Code Ref.
Building seismic weight:	W =	2,040	kip	7.4.1.3.1
Number of stories:	n =	3		
m_{max} @ BSE-2E:	m_{max2} =	3.5		7.4.1.3.1
m_{max} @ BSE-1E:	m_{max1} =	2.5		7.4.1.3.1
Damping coefficient:	B_1 =	1.00		2.4.1.7.1
BSE-2E mod. factors product:	$C_{12}C_{22}$ =	1.4		Table 7-3
BSE-1E mod. factors product:	$C_{11}C_{21}$ =	1.4		Table 7-3
Effective mass factor:	C_m =	0.8		Table 7-4
BSE-2E spectral acceleration:	S_{a2} =	1.29	g	2.4.3
BSE-1E spectral acceleration:	S_{a1} =	0.70	g	2.4.3
BSE-2E pseudo lateral load:	$V_{2E} = C_{12}C_{22}C_mS_{a2}W$ =	2956.8	kip	7.4.1.3.1
BSE-1E pseudo lateral load:	$V_{1E} = C_{11}C_{21}C_mS_{a1}W$ =	1597.6	kip	7.4.1.3.1

Vertical Distribution of Seismic Forces (Sec. 7.4.1.3.2)			Code Ref.
Story force:	$F_x = w_xh_x^k/(\Sigma w_xh_x^k)*V$ =	See Table Below	Eq. 7-24
Story heihgt exponent factor:	k =	1.00	7.4.1.3.2
Diaphragm force:	$F_{px} = V_x*w_x/W_x$ =	See Table Below	Eq. 7-26

Story Name	Story Weight w_x (k)	Story Height h_x (ft)	$w_x*h_x^k$	BSE-2E Story Force F_{x2} (k)	BSE-1E Story Force F_{x1} (k)	BSE-2E Story Shear V_{x2} (k)	BSE-1E Story Shear V_{x1} (k)	Total Weight Above W_x (k)	BSE-2E Diaph. Force F_{px2} (k)	BSE-1E Diaph. Force F_{px1} (k)	
Roof	209	41.5	8673.5	627.8	339.2	627.8	339.2	209.0	627.8	339.2	14.0
Level 2	695	27.5	19113	1383.4	747.5	2011.2	1086.7	904.0	1546.2	835.5	16.0
Level 1	1136	11.5	13064	945.6	510.9	2956.8	1597.6	2040.0	1646.6	889.7	
SUM =	**2040**		**40850**								

ASCE 41-17 Linear Static Procedure (Sec. 7.4.1) - 6300 Building

Acceleration Response Spectra

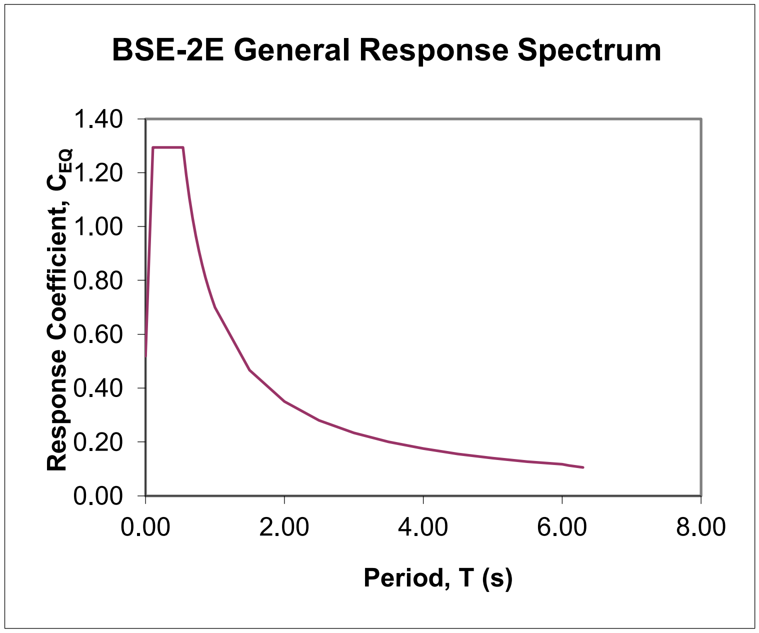
	BSE-2E		BSE-1E	
	T (sec)	C_{EQ}	T (sec)	C_{EQ}
	0.00	0.52	0.00	0.28
T_0 =	0.11	1.29	0.10	0.70
T_s =	0.54	1.29	0.51	0.70
	0.59	1.19	0.56	0.64
	0.63	1.11	0.61	0.59
	0.68	1.03	0.65	0.54
	0.72	0.97	0.70	0.50
	0.77	0.91	0.75	0.47
	0.82	0.86	0.80	0.44
	0.86	0.81	0.85	0.42
	0.91	0.77	0.90	0.39
	0.95	0.73	0.95	0.37
T_1 =	1	0.70	1	0.35
	2	0	2	0
	2	0	2	0
	3	0	3	0
	3	0	3	0
	4	0	4	0
	4	0	4	0
	5	0	5	0
	5	0	5	0
	6	0	6	0
T_L =	6	0	6	0
	6	0	6	0
	6	0	6	0
	6	0	6	0

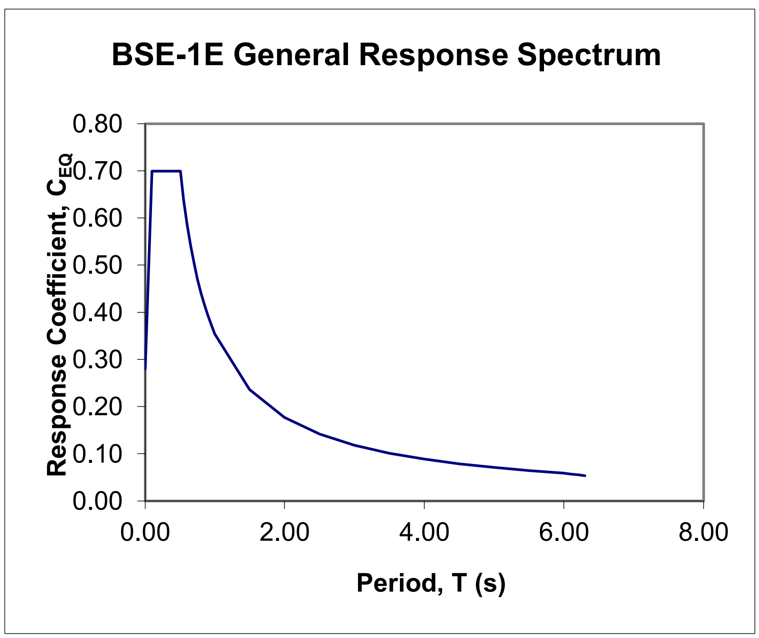
$C_{EQ} = C_1C_2C_MS_{XS}[(5/B_1\text{-}2)T/T_s+0.4$ - { @ $T < T_0$ }

$C_{EQ} = C_1C_2C_MS_{XS}/B_1$ - { @ $T_0 \le T \le T_S$ }

$C_{EQ} = C_1C_2C_MS_{X1}/(B_1{*}T)$ - { @ $T_s < T \le T_L$ }

$C_{EQ} = C_1C_2C_MT_LS_{X1}/(B_1{*}T^2)$ - { @ $T_L < T$ }





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Client:	City of Tukwila	Sheet:	of
Project:	City of Tukwila	Sheet:	of
	Tukwila Seismic Evaluation	Design By:	MLO
	6300 Building	Date:	
Project No.:	262022.017	Date:	

Tukwila 6300 Building Tier 2 Life Safety Calculations

Overturning | **Code Ref.**

	V_{roof} =	339.2	kip	*Roof story force*
	$V_{level\ 2}$ =	747.5	kip	*Level 2 story force*
	$V_{level\ 1}$ =	510.9	kip	*Level 1 story force*
	h_{roof} =	42	ft	*Roof Height*
	$h_{level\ 2}$ =	28	ft	*Level 2 height*
	$h_{level\ 1}$ =	12	ft	*Level 1 height*
Q_{UD} =	M_{OT} =	40509	k-ft	*Overturning moment due to seismic*
	$q_{bearing}$ =	4000	psf	*Allowable soil bearing pressure*
	$A_{footings}$ =	569	sf	*Total area of footings along East edge*
	P_{OT} =	2277	kip	*Allowable bearing resistance*
	Building width =	80	ft	*Bearing moment arm*
Q_{CE} =	M_R =	182131	k-ft	*Overturning resistance*
	m =	1		*m-factor*
	k =	0.90		*Knowledge factor*
	mkQ_{CE} =	163918	k-ft	*Overturning resistance*
	DCR =	0.247	C	

Foundation Dowels | **Code Ref.**

Wall Demands

	$L_{w,x}$ =	360	ft	*Horizontal cross-sectional area of all shear walls in direction X*
	$L_{w,y}$ =	174	ft	*Horizontal cross-sectional area of all shear walls in direction Y*
	V_{Base} =	1,598	kip	*Max Story Shear*
Q_{UD} =	v_x =	4.4	kip/ft	*Shear Stress in Walls, x-dir*
Q_{UD} =	v_y =	9.2	kip/ft	*Shear Stress in Walls, y-dir*

Dowel Shear Capacity (#3 @ 18" oc)

	A_s =	0.11	in^2	*Shear reinforcing at footing interface*
	f_y =	60	ksi	*Reinforcing yield strength*
	s =	18	in	*Reinforcing spacing*
Q_{CE} =	V_s =	4.4	kip/ft	*Shear Stress in Walls, y-dir*
	T_e =	0.34		*Effective fundamental period of the building*
	C_1C_2 =	1.1		*Modification factors for force controlled*
	J =	2		*Force delivery reduction factor*
	k =	0.9		*Knowledge factor*
	$(C_1C_2J)kQ_{CE}$ =	8.7		*Psuedo capacity for dowel reinforcing*
	DCR =	1.056	NC	

Deflection Compatibility			Code Ref.

Level 1 Drift

V =	1597.6	kip	*Building Seismic Force*
F =	426.0	kip	*Seismic Force tributary to wall @ GL 6*

Building is more flexible in the E/W direction. Wall @ GL 6 was chosen b/c it is in-line with columns and is the shortest wall

E =	3605	ksi	*Modulus of elasticity for concrete*
t =	8	in	*Wall thickness*
h =	11.5	ft	*Wall height*
l =	11.8	ft	*Wall length*
k =	7445	k/in	*Wall stiffness*
Δ =	0.057	in	*Wall deflection*

Shear Demand

	k_{Column} =	4.211	k/in	*Column stiffness*
Q_{UD} =	F_{Column} =	0.241	kip	*Column Force*

Shear Strength of Columns

	A_s =	0.4	in_2	*Column reinforcing (4 #9 Vert)*
	f_y =	60	ksi	*Steel yield Stress*
	d =	10	in	*Column depth*
	b =	12	in	*Column width*
	s =	12	in	*Reinforcing spacing*
	f'_c =	4000	psi	*Concrete compressive strength*
	V_s =	20.0	kip	*Steel shear strength*
	V_c =	15.2	kip	*Concrete shear strength*
Q_{CE} =	V_n =	35.2	kip	*Column shear strength*
	m =	2.1		*m-factor (LS = 2.1, CP = 2.5)*
	k =	0.9		*Knowledge Factor*
	mkQ_{CE} =	66.5	kip	*Column psuedo shear capacity*
	DCR =	0.004	C	

Shear Stress Check - Wood			Code Ref.

V_{roof} = 339.2 kip

Location	Length	Trib. Area (sf)	Trib. Force (k)	Force/Length (plf)	DCR	
GL 2	30	3380	68.6	2286	0.98	C
GL 2.8	21.5	1920	39.0	1812	0.78	C
GL 3.5	20.75	4680	94.9	4576	1.97	NC
GL 6.8	20	4040	82.0	4098	1.77	NC
GL 7	23	2700	54.8	2382	1.03	NC
		16720	339.219			

Nominal Shear Capacity =	950	plf	
m =	3.8		*m-factor*
k =	0.9		*knowledge factor*
Shear Strength Capacity =	2320.714286	plf	*Nominal capacity converted to allowable capacity by dividing by 2 per Sec. 4.3.3 of 2015 SDPWS*
			Allowable capacity converted to strength capacity by divded by 0.7

Axial/Flexural Stress/Redundancy/Compact Members/Strong Column-Weak Beam (SC-WB) (Cont.)	Code Ref.

W12x50 Column

Column Dimensions

d_d =	12.2	in	*Column Depth*	b_f =	8.08	in	*Flange Width*	
t_f =	0.64	in	*Flange Thickness*	t_w =	0.37	in	*Web Thickness*	

Material Properties

E =	29000	ksi	*Modulus of Elasticity*	
F_y =	36	ksi	*Yield Strength*	
F_{ye} =	39.6	ksi	*Expected Yield Strength.* $F_{ye} = 1.1F_y$	ASCE 41-17 T. 9-3

Demands from RISA 3D using BSE-1E and 2E Seismic Loads

$P_{u,dead}$ =	0.0	kip	$P_{u,live}$ =	0.0	kip	RISA Output
$V_{u,dead}$ =	0.0	kip	$V_{u,live}$ =	0.0	kip	RISA Output
$M_{u,dead}$ =	0.0	k*ft	$M_{u,live}$ =	0.0	k*ft	RISA Output

Framing spans parallel to moment frames. Negligible loads would be applied to frame members.

$P_{u,EQ,1E}$ =	355.4	kip	$P_{u,EQ,2E}$ =	657.8	kip	RISA Output
$V_{u,EQ,1E}$ =	281.3	kip	$V_{u,EQ,2E}$ =	520.6	kip	RISA Output
$M_{u,EQ,1E}$ =	4500.3	k*ft	$M_{u,EQ,2E}$ =	8329.0	k*ft	RISA Output

$Q_G = 1.1(Q_D + Q_L + Q_S)$ Use $Q_G = 0.9Q_D$ if gravity and seismic loads are counteracting — ASCE 41-17 Eq. 7-1,2

$Q_{G,P}$ =	0.0	kip	*Force Due to Gravity Loads*	ASCE 41-17 Eq. 7-1,2
$Q_{G,M}$ =	0.0	k*ft	*Moment Due to Gravity Loads*	ASCE 41-17 Eq. 7-1,2
$Q_{G,V}$ =	0.0	kip	*Shear Due to Gravity Loads*	ASCE 41-17 Eq. 7-1,2

Force Controlled Demands

$$Q_{UF} = Q_G \pm \frac{\chi Q_E}{C_1 C_2 J}$$ ASCE 41-17 7.5.2.1.2.2

X_{1E} =	1.0			*Adjustment Factor for Collapse Prevention*	ASCE 41-17 7.5.2.1.2.2
X_{2E} =	1.3			*Adjustment Factor for Life Safety*	
J =	2.0			*Reduction Factor for High Level of Seismicity*	ASCE 41-17 7.5.2.1.2.2
$C_1C_{2,1E}$ =	1.4			*1E Alternative Modification Factor*	ASCE 41-17 Tier 2 LSP
$C_1C_{2,2E}$ =	1.4			*2E Alternative Modification Factor*	ASCE 41-17 Tier 2 LSP
$Q_{UF,P,1E}$ =	$P_{UF,1E}$ =	126.9	kip	*1E Force Controlled Axial Force*	ASCE 41-17 Eq. 7-35
$Q_{UF,P,2E}$ =	$P_{UF,2E}$ =	305.4	kip	*2E Force Controlled Axial Force*	ASCE 41-17 Eq. 7-35

Deformation Controlled Demands

$Q_{UD} = Q_G + Q_E$ ASCE 41-17 Eq. 7-34

$Q_{UD,M,1E}$ =	$M_{UD,1E}$ =	4500	k*ft	*1E Deformation Controlled Moment*	ASCE 41-17 Eq. 7-34
$Q_{UD,V,1E}$ =	$V_{UD,1E}$ =	281	kip	*1E Deformation Controlled Shear*	ASCE 41-17 Eq. 7-34
$Q_{UD,M,2E}$ =	$M_{UD,2E}$ =	8329	k*ft	*2E Deformation Controlled Moment*	ASCE 41-17 Eq. 7-34
$Q_{UD,V,2E}$ =	$V_{UD,2E}$ =	521	kip	*2E Deformation Controlled Shear*	ASCE 41-17 Eq. 7-34

Axial/Flexural Stress/Redundancy/Compact Members/Strong Column-Weak Beam (SC-WB) (Cont.)	Code Ref.

Member Capacity

Capacities were calculated using Enercalc using F_y and F_{ye}

$0.9\ P_{n,Fy}$ =	285	kip	$0.9\ P_{n,Fye}$ =	299	kip	*Axial Capacity*	Enercalc Output
$V_{n,Fy}$ =	108	kip	$V_{n,Fye}$ =	119	kip	*Shear Capacity*	Enercalc Output
$0.9\ M_{nx,Fy}$ =	194	k*ft	$0.9\ M_{nx,Fye}$ =	214	k*ft	*Moment Capacity*	Enercalc Output
$P_{CL} = P_n$ =	317	kip	$P_{ye} = P_n$ =	332	kip	*Adjusted Axial Capacity*	
$V_{CL} = V_n$ =	108	kip	$V_{CE} = V_n$ =	119	kip	*Adjusted Shear Capacity*	
$M_{CL} = M_{nx}$ =	216	k*ft	$M_{CE} = M_{nx}$ =	237	k*ft	*Adjusted Moment Capacity*	

m-Factor

L.S. 1E m-factor

$P_{UF,1E}/P_{ye}$ = 0.38 < 0.1 ? *False. Force Controlled!* — ASCE 41-17 T. 9-6

C.P. 2E m-factor

$P_{UF,2E}/P_{CL}$ = 0.92 < 0.1 ? *False. Force Controlled!* — ASCE 41-17 T. 9-6

Acceptance Criteria

Flexure is the controlling demand since the column is treated as a beam-column.

L.S. 1E DCR's

Flexural DCR

$$DCR = \frac{Q_{UD,M,1E}}{C_1 C_2 * J * M_{CE}}$$

DCR = 6.78 **NC** — ASCE 41-17 Eq.7-36

C.P. 2E DCR's

$$DCR = \frac{Q_{UD,M,2E}}{C_1 C_2 * J * M_{CE}}$$

DCR = 12.54 **NC** — ASCE 41-17 Eq.7-36

Axial/Flexural Stress/Redundancy/Compact Members/Strong Column-Weak Beam (SC-WB) (Cont.)	Code Ref.

W12x50 Beam

Column Dimensions

d_d =	12.2	in	*Column Depth*	b_f =	8.08	in	*Flange Width*
t_f =	0.64	in	*Flange Thickness*	t_w =	0.37	in	*Web Thickness*

Material Properties

E =	29000	ksi	*Modulus of Elasticity*	
F_y =	36	ksi	*Yield Strength*	ASCE 41-17 T. 9-1
F_{ye} =	39.6	ksi	*Expected Yield Strength. F_{ye} = 1.1F_y*	ASCE 41-17 T. 9-3

Demands from RISA 3D using BSE-1E and 2E Seismic Loads

$P_{u,dead}$ =	0.0	kip	$P_{u,live}$ =	0.0	kip	RISA Output
$V_{u,dead}$ =	0.0	kip	$V_{u,live}$ =	0.0	kip	RISA Output
$M_{u,dead}$ =	0.0	k*ft	$M_{u,live}$ =	0.0	k*ft	RISA Output

Framing spans parallel to moment frames. Negligible loads would be applied to frame members.

$P_{u,EQ,1E}$ =	365.7	kip	$P_{u,EQ,2E}$ =	676.8	kip	RISA Output
$V_{u,EQ,1E}$ =	270.6	kip	$V_{u,EQ,2E}$ =	500.8	kip	RISA Output
$M_{u,EQ,1E}$ =	4085.4	k*ft	$M_{u,EQ,2E}$ =	7561.1	k*ft	RISA Output

$Q_G = 1.1(Q_D + Q_L + Q_S)$ Use $Q_G = 0.9Q_D$ if gravity and seismic loads are counteracting — ASCE 41-17 Eq. 7-1,2

$Q_{G,P}$ =	0.0	kip	*Force Due to Gravity Loads*	ASCE 41-17 Eq. 7-1,2
$Q_{G,M}$ =	0.0	k*ft	*Moment Due to Gravity Loads*	ASCE 41-17 Eq. 7-1,2
$Q_{G,V}$ =	0.0	kip	*Shear Due to Gravity Loads*	ASCE 41-17 Eq. 7-1,2

Force Controlled Demands

$$Q_{UF} = Q_G \pm \frac{\chi Q_E}{C_1 C_2 J}$$

ASCE 41-17 7.5.2.1.2.2

X_{1E} =	1.0				*Adjustment Factor for Collapse Prevention*	ASCE 41-17 7.5.2.1.2.2
X_{2E} =	1.3				*Adjustment Factor for Life Safety*	
J =	2.0				*Reduction Factor for High Level of Seismicity*	ASCE 41-17 7.5.2.1.2.2
$C_1C_{2,1E}$ =	1.4				*1E Alternative Modification Factor*	ASCE 41-17 Tier 2 LSP
$C_1C_{2,2E}$ =	1.4				*2E Alternative Modification Factor*	ASCE 41-17 Tier 2 LSP
$Q_{UF,P,1E}$ =		$P_{UF,1E}$ =	130.6	kip	*1E Force Controlled Axial Force*	ASCE 41-17 Eq. 7-35
$Q_{UF,P,2E}$ =		$P_{UF,2E}$ =	314.2	kip	*2E Force Controlled Axial Force*	ASCE 41-17 Eq. 7-35

Deformation Controlled Demands

$$Q_{UD} = Q_G + Q_E$$

ASCE 41-17 Eq. 7-34

$Q_{UD,M,1E}$ =	$M_{UD,1E}$ =	4085	k*ft	*1E Deformation Controlled Moment*	ASCE 41-17 Eq. 7-34
$Q_{UD,V,1E}$ =	$V_{UD,1E}$ =	271	kip	*1E Deformation Controlled Shear*	ASCE 41-17 Eq. 7-34
$Q_{UD,M,2E}$ =	$M_{UD,2E}$ =	7561	k*ft	*2E Deformation Controlled Moment*	ASCE 41-17 Eq. 7-34
$Q_{UD,V,2E}$ =	$V_{UD,2E}$ =	501	kip	*2E Deformation Controlled Shear*	ASCE 41-17 Eq. 7-34

Axial/Flexural Stress/Redundancy/Compact Members/Strong Column-Weak Beam (SC-WB) (Cont.)	Code Ref.

Member Capacity

Capacities were calculated using Enercalc using F_y and F_{ye}

$0.9\ P_{n,Fy}$ =	285	kip	$0.9\ P_{n,Fye}$ =	299	kip	*Axial Capacity*	Enercalc Output
$V_{n,Fy}$ =	108	kip	$V_{n,Fye}$ =	119	kip	*Shear Capacity*	Enercalc Output
$0.9\ M_{nx,Fy}$ =	194	k*ft	$0.9\ M_{nx,Fye}$ =	214	k*ft	*Moment Capacity*	Enercalc Output
$P_{CL} = P_n$ =	317	kip	$P_{ye} = P_n$ =	332	kip	*Adjusted Axial Capacity*	
$V_{CL} = V_n$ =	108	kip	$V_{CE} = V_n$ =	119	kip	*Adjusted Shear Capacity*	
$M_{CL} = M_{nx}$ =	216	k*ft	$M_{CE} = M_{nx}$ =	237	k*ft	*Adjusted Moment Capacity*	

m-Factor

L.S. 1E m-factor

$P_{UF,1E}/P_{ye}$ =	0.394	< 0.1 ?	*False. Force Controlled!*	ASCE 41-17 T. 9-6

C.P. 2E m-factor

$P_{UF,2E}/P_{CL}$ =	0.947	< 0.1 ?	*False. Force Controlled!*	ASCE 41-17 T. 9-6

Acceptance Criteria

L.S. 1E DCR's

Flexural DCR

$$DCR = \frac{Q_{UD,M,1E}}{C_1 C_2 * J * M_{CE}}$$

DCR = 6.15 — NC — ASCE 41-17 Eq.7-36

C.P. 2E DCR's

$$DCR = \frac{Q_{UD,M,2E}}{C_1 C_2 * J * M_{CE}}$$

DCR = 11.38 — NC — ASCE 41-17 Eq.7-36

Tukwila 6300 Building Tier 2 Collapse Prevention Calculations

Overturning				Code Ref.
	V_{roof} =	627.8 kip	*Roof story force*	
	$V_{level\ 2}$ =	1383.4 kip	*Level 2 story force*	
	$V_{level\ 1}$ =	945.6 kip	*Level 1 story force*	
	h_{roof} =	42 ft	*Roof Height*	
	$h_{level\ 2}$ =	28 ft	*Level 2 height*	
	$h_{level\ 1}$ =	12 ft	*Level 1 height*	
Q_{UD} =	M_{OT} =	74973 k-ft	*Overturning moment due to seismic*	
	$q_{bearing}$ =	4000 psf	*Allowable soil bearing pressure*	
	$A_{footings}$ =	569 sf	*Total area of footings along East edge*	
	P_{OT} =	2277 kip	*Allowable bearing resistance*	
	Building width =	80 ft	*Bearing moment arm*	
Q_{CE} =	M_R =	182131 k-ft	*Overturning resistance*	
	m =	1	*m-factor*	
	k =	0.90	*Knowledge factor*	
	mkQ_{CE} =	163918 k-ft	*Overturning resistance*	
	DCR =	0.457 C		

Foundation Dowels				Code Ref.
Wall Demands				
	$L_{w,x}$ =	360 ft	*Horizontal cross-sectional area of all shear walls in direction X*	
	$L_{w,y}$ =	174 ft	*Horizontal cross-sectional area of all shear walls in direction Y*	
	V_{Base} =	2,957 kip	*Max Story Shear*	
Q_{UD} =	v_x =	8.2 kip/ft	*Shear Stress in Walls, x-dir*	
Q_{UD} =	v_y =	17.0 kip/ft	*Shear Stress in Walls, y-dir*	
Wall Shear Capacity (#3 @ 18" oc)				
	b =	8 in	*Concrete wall width*	
	f'_c =	4000 psi	*Concrete compressive strength*	
	V_c =	1.0 kip/ft	*Concrete shear capacity*	
	A_s =	0.11 in^2	*Shear reinforcing at footing interface*	
	f_y =	60 ksi	*Reinforcing yield strength*	
	s =	18 in	*Reinforcing spacing*	
	V_s =	4.4 kip/ft	*Shear Stress in Walls, y-dir*	
Q_{CE} =	V_n =	5.4 kip/ft	*Shear Stress in Walls, y-dir*	
	T_e =	0.34	*Effective fundamental period of the building*	
	C_1C_2 =	1.1	*Modification factors for force controlled*	
	J =	1	*Force delivery reduction factor*	
	k =	0.9	*Knowledge factor*	
	$(C_1C_2J)kQ_{CE}$ =	5.4	*Psuedo capacity for dowel reinforcing*	
	DCR =	3.178 NC		
		10.6 kip/ft	*Additional shear required*	

Deflection Compatibility | Code Ref.

Level 1 Drift

V =	2956.8	kip	*Building Seismic Force*
F =	788.5	kip	*Seismic Force tributary to wall @ GL 6*

Building is more flexible in the E/W direction. Wall @ GL 6 was chosen b/c it is in-line with columns and is the shortest wall

E =	3605	ksi	*Modulus of elasticity for concrete*
t =	8	in	*Wall thickness*
h =	11.5	ft	*Wall height*
l =	11.8	ft	*Wall length*
k =	7445	k/in	*Wall stiffness*
Δ =	0.106	in	*Wall deflection*

Shear Demand

	k_{Column} =	4.211	k/in	*Column stiffness*
Q_{UD} =	F_{Column} =	0.446	kip	*Column Force*

Shear Strength of Columns

	A_s =	0.4	in_2	*Column reinforcing (4 #9 Vert)*
	f_y =	60	ksi	*Steel yield Stress*
	d =	10	in	*Column depth*
	b =	12	in	*Column width*
	s =	12	in	*Reinforcing spacing*
	f'_c =	4000	psi	*Concrete compressive strength*
	V_s =	20.0	kip	*Steel shear strength*
	V_c =	15.2	kip	*Concrete shear strength*
Q_{CE} =	V_n =	35.2	kip	*Column shear strength*
	m =	2.1		*m-factor (LS = 2.1, CP = 2.5)*
	k =	0.9		*Knowledge Factor*
	mkQ_{CE} =	66.5	kip	*Column psuedo shear capacity*

DCR = 0.007 C

Shear Stress Check - Wood | Code Ref.

V_{roof} = 627.8 kip

Location	Length	Trib. Area (	Trib. Force	Force/Length (plf)	DCR	
GL 2	30	3380	126.9	4230	1.54	NC
GL 2.8	21.5	1920	72.1	3353	1.22	NC
GL 3.5	20.75	4680	175.7	8469	3.08	NC
GL 6.8	20	4040	151.7	7585	2.76	NC
GL 7	23	2700	101.4	4408	1.60	NC
		16720	627.815			

Nominal Shear Capacity =	950	plf	
m =	4.5		*m-factor*
k =	0.9		*knowledge factor*
Shear Strength Capacity =	2748.214	plf	*Nominal capacity converted to allowable capacity by dividing by 2 per Sec. 4.3.3 of 2015 SDPWS*
			Allowable capacity converted to strength capacity by divded by 0.7

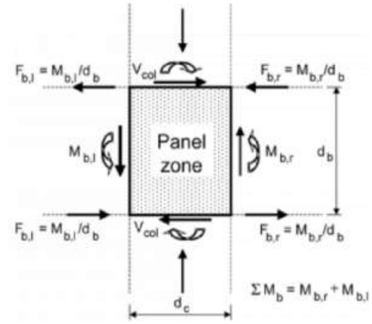
Panel Zone Shear	Code Ref.

Assumptions:

1. The effect of inelastic panel-zone deformation on the local frame stability is not accounted for in the Tier 1 analysis.
2. Column demand is 20% of the compression capacity.
3. A representative frame on the second story with a W12x96 shall be analyzed.

Panel Zone Shear Strength

$$R_n = 0.60 F_y d_c t_w$$



F_y =	36	ksi
d_c =	12.2	in
t_w =	0.37	in
R_n =	97.5	kip

Column Shear Capacity

$$V_n = 0.6 F_y A_w C_v$$

h/t_w =	26.8	<	$2.24^*(E/F_y)^{0.5}$ =	63.6
φ_v =	1.00			
C_v =	1.00			
A_w =	4.514 in^2			
V_{col} =	97.5 kip			

Z_{b_left} =	71.9	in^3	*Left Beam Plastic Modulus*
Z_{b_right} =	71.9	in^3	*Right Beam Plastic Modulus*
F_y =	36	ksi	*Yield Stress*
M_{br_left} =	2588	k-in	*Default Lower-Bound Material Yield Strength*
M_{br_right} =	2588.4	k-in	*Default Lower-Bound Material Yield Strength*
d_{b_left} =	12.2	in	*Left Beam Depth*
d_{b_right} =	12.2	in	*Right Beam Depth*

$$V_{pz} = \Sigma M_b / d_b - V_{col}$$

$$V_{pz} = 0.8 \frac{\Sigma M_{br}}{d_b} - V_{col}$$

NEHRP NIST GCR 09-917-3 Section 5.4.3

V_{pz} = 242 kip *Panel Zone Demand*

m =	11		*m-factor (LS = 8, CP = 11)*	ASCE 41 -17, Table 9-6
k =	0.9			
$mk(\varphi R_n)$ =	965	kip	0.25 *Available Panel Zone Shear Strength*	

DCR = 0.251 C

{Note: 80% of the Panel Zone Strength defined in NEHRP NIST GCR 09-917-3 Section 5.4.3
is defined in ASCE 41-17 Section A.3.1.3.5}

17-3 Immediate Occupancy Basic Configuration Checklist

Very Low Seismicity

Building System - General

C	NC	N/A	U	EVALUATION STATEMENT	COMMENT
X				LOAD PATH: The structure contains 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. *(Tier 2: Sec. 5.4.1.1; Commentary: Sec. A.2.1.1)*	
X				ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building is greater than 0.5% of the height of the shorter building in low seismicity, 1.0% in moderate seismici ty, and 3.0% in high seismicity. (Tier 2: Sec. 5.4.1.2; Commentary: Sec. A.2.1.2)	
		X		MEZZANINES: Interior mezzanine levels are braced independently from the main structure or are anchored to the seismic-force-resisting elements of the main structure. *(Tier 2: Sec. 5.4.1.3; Commentary: Sec. A.2.1.3)*	

Building System – Building Configuration

C	NC	N/A	U	EVALUATION STATEMENT	COMMENT
		X		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. *(Tier 2: Sec. 5.4.2.1; Commentary: Sec. A.2.2.2)*	
		X		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. *(Tier 2: Sec. 5.4.2.2; Commentary: Sec. A.2.2.3)*	
		X		VERTICAL IRREGULARITIES: All vertical elements in the seismic-force- resisting system are continuous to the foundation. *(Tier 2: Sec. 5.4.2.3; Commentary: Sec. A.2.2.4)*	
		X		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. *(Tier 2: Sec. 5.4.2.4; Commentary: Sec. A.2.2.5)*	
		X		MASS: There is no change in effective mass of more than 50% from one story to the next. Light roofs, penthouses, and mezzanines need not be considered. *(Tier 2: Sec. 5.4.2.5; Commentary: Sec. A.2.2.6)*	
		X		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. *(Tier 2: Sec. 5.4.2.6; Commentary: Sec. A.2.2.7)*	

17-3 Immediate Occupancy Basic Configuration Checklist

Low Seismicity

(Complete the Following Items in Addition to the Items for Very Low Seismicity)

Geologic Site Hazards

C	NC	N/A	U	EVALUATION STATEMENT	COMMENT
	X			LIQUEFACTION: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance do not exist in the foundation soils at depths within 50 ft (15.2 m) under the building. *(Tier 2: Sec. 5.4.3.1; Commentary: Sec. A.6.1.1)*	Soils around Green River in Tukwila tend to be liquefiable. Site Class F site per 2008 Geotechncial report completed as part of original report.
		X		SLOPE FAILURE: The building site is located away from potential earthquake-induced slope failures or rockfalls so that it is unaffected by such failures or is capable of accommodating any predicted movements without failure. *(Tier 2: Sec. 5.4.3.1; Commentary: Sec. A.6.1.2)*	
X				SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site are not anticipated. *(Tier 2: Sec. 5.4.3.1 ; Commentary: Sec.A.6.1.3)*	

Moderate and High Seismicity

(Complete the Following Items in Addition to the Items for Low Seismicity)

Foundation Configuration

C	NC	N/A	U	EVALUATION STATEMENT	COMMENT
X				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 $0.6S_a$. *(Tier 2: Sec. 5.4.3.3; Commentary: Sec. A.6.2.1)*	
	X			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. *(Tier 2: Sec. 5.4.3.4; Commentary: Sec. A.6.2.2)*	No beams/slabs/soils classified as Site Class A, B, or C between shallow foundation elements.

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

17-6. Collapse Prevention Structural Checklist for Building Type W2

Low and Moderate Seismicity

Seismic-Force-Resisting System

C	NC	N/A	U	EVALUATION STATEMENT	COMMENT
X				REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2. *(Tier 2: Sec. 5.5.1.1; Commentary: Sec. A.3.2.1.1)*	
	X			SHEAR STRESS CHECK: The shear stress in the shear walls, calculated using the Quick Check procedure of Section 4.4.3.3, is less than the following values: Structural panel sheathing, 1,000 lb/ft (14.6 kN/m); Diagonal sheathing, 700 lb/ft (10.2 kN/m); Straight sheathing, 100 lb/ft (1.5 kN/m); All other conditions, 100 lb/ft (1.5 kN/m). *(Tier 2: Sec. 5.5.3.1.1 ; Commentary: Sec.A.3.2.7.1)*	Shear stress check exceeds 1000 plf
X				STUCCO (EXTERIOR PLASTER) SHEAR WALLS: Multi-story buildings do not rely on exterior stucco walls as the primary seismic-force-resisting system. *(Tier 2: Sec. 5.5.3.6.1; Commentary: Sec. A.3.2.7.2)*	
X				GYPSUM WALLBOARD OR PLASTER SHEAR WALLS: Interior plaster or gypsum wallboard is not used for shear walls on buildings more than one story high with the exception of the uppermost level of a multi-story building. *(Tier 2: Sec. 5.5.3.6.1; Commentary: Sec. A.3.2.7.3)*	Interior walls have gypsum wallboard, but the structure is only one story.
	X			NARROW WOOD SHEAR WALLS: Narrow wood shear walls with an aspect ratio greater than 2-to-1 are not used to resist seismic forces. *(Tier 2: Sec. 5.5.3.6.1; Commentary: Sec. A.3.2.7.4)*	Several of the walls have an aspect ratio above 2-1.
		X		WALLS CONNECTED THROUGH FLOORS: Shear walls have an interconnection between stories to transfer overturning and shear forces through the floor. *(Tier 2: Sec. 5.5.3.6.2; Commentary: Sec. A.3.2.7.5)*	
		X		HILLSIDE SITE: For structures that are taller on at least one side by more than one-half story because of a sloping site, all shear walls on the downhill slope have an aspect ratio less than 1-to-2. *(Tier 2: Sec. 5.5.3.6.3; Commentary: Sec. A.3.2.7.6)*	Wood shearwalls only exist above grade.
		X		CRIPPLE WALLS: Cripple walls below first-floor-level shear walls are braced to the foundation with wood structural panels. *(Tier 2: Sec. 5.5.3.6.4; Commentary: Sec. A.3.2.7.7)*	
	X			OPENINGS: Walls with openings greater than 80% of the length are braced with wood structural panel shear walls with aspect ratios of not more than 1.5-to-1 or are supported by adjacent construction through positive ties capable of transferring the seismic forces. *(Tier 2: Sec. 5.5.3.6.5; Commentary: Sec. A.3.2.7.8)*	Various locations have shear walls with larger aspect ratios.

Connections

C	NC	N/A	U	EVALUATION STATEMENT	COMMENT
X				WOOD POSTS: There is a positive connection of wood posts to the foundation. *(Tier 2: Sec. 5.7.3.3; Commentary: Sec. A.5.3.3)*	

X				WOOD SILLS: All wood sills are bolted to the foundation. *(Tier 2: Sec. 5.7.3.3; Commentary: Sec. A.5.3.4)*	
X				GIRDER–COLUMN CONNECTION: There is a positive connection using plates, connection hardware, or straps between the girder and the column support. *(Tier 2: Sec. 5.7.4.1; Commentary: Sec. A.5.4.1)*	

High Seismicity

(Complete the Following Items in Addition to the Items for Low and Moderate Seismicity)

Connections

C	NC	N/A	U	EVALUATION STATEMENT	COMMENT
X				WOOD SILL BOLTS: Sill bolts are spaced at 6 ft or less with acceptable edge and end distance provided for wood and concrete. *(Tier 2: Sec. 5.7.3.3; Commentary: Sec. A.5.3.7)*	

Diaphragms

C	NC	N/A	U	EVALUATION STATEMENT	COMMENT
X				DIAPHRAGM CONTINUITY: The diaphragms are not composed of split-level floors and do not have expansion joints. *(Tier 2: Sec. 5.6.1.1; Commentary: Sec. A.4.1.1)*	
	X			ROOF CHORD CONTINUITY: All chord elements are continuous, regardless of changes in roof elevation. *(Tier 2: Sec. 5.6.1.1; Commentary: Sec. A.4.1.3)*	
		X		DIAPHRAGM REINFORCEMENT AT OPENINGS: There is reinforcing around all diaphragm openings larger than 50% of the building width in either major plan dimension. *(Tier 2: Sec. 5.6.1.5; Commentary: Sec. A.4.1.8)*	No diaphragm openings larger than 50% of the building width.
		X		STRAIGHT SHEATHING: All straight-sheathed diaphragms have aspect ratios less than 1-to-1 in the direction being considered. *(Tier 2: Sec. 5.6.2; Commentary: Sec. A.4.2.1)*	Diaphragm consists of plywood sheathing
X				SPANS: All wood diaphragms with spans greater than 12 ft (3.6 m) consist of wood structural panels or diagonal sheathing. *(Tier 2: Sec. 5.6.2; Commentary: Sec. A.4.2.2)*	
		X		DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 30 ft (9.1 m) and aspect ratios less than or equal to 3-to-1. *(Tier 2: Sec. 5.6.2; Commentary: Sec. A.4.2.3)*	Diaphragm is blocked plywood sheathing.
		X		OTHER DIAPHRAGMS: The diaphragms do not consist of a system other than wood, metal deck, concrete, or horizontal bracing. *(Tier 2: Sec. 5.6.5; Commentary: Sec. A.4.7.1)*	

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

17-7. Immediate Occupancy Checklist for Building Type W2

Very Low Seismicity

Seismic-Force-Resisting System

C	NC	N/A	U	EVALUATION STATEMENT	COMMENT
X				REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2. *(Tier 2: Sec. 5.5.1.1; Commentary: Sec. A.3.2.1.1)*	
	X			SHEAR STRESS CHECK: The shear stress in the shear walls, calculated using the Quick Check procedure of Section 4.4.3.3, is less than the following values: Structural panel sheathing, 1,000 lb/ft (14.6 kN/m); Diagonal sheathing, 700 lb/ft (10.2 kN/m); Straight sheathing, 100 lb/ft (1.5 kN/m); All other conditions, 100 lb/ft (1.5 kN/m). *(Tier 2: Sec. 5.5.3.1.1 ; Commentary: Sec.A.3.2.7.1)*	Shear stress check exceeds 1000 plf
X				STUCCO (EXTERIOR PLASTER) SHEAR WALLS: Multi-story buildings do not rely on exterior stucco walls as the primary seismic-force-resisting system. *(Tier 2: Sec. 5.5.3.6.1; Commentary: Sec. A.3.2.7.2)*	
X				GYPSUM WALLBOARD OR PLASTER SHEAR WALLS: Interior plaster or gypsum wallboard is not used for shear walls on buildings more than one story high with the exception of the uppermost level of a multi-story building. *(Tier 2: Sec. 5.5.3.6.1; Commentary: Sec. A.3.2.7.3)*	Interior walls have gypsum wallboard, but the structure is only one story.
	X			NARROW WOOD SHEAR WALLS: Narrow wood shear walls with an aspect ratio greater than 2-to-1 are not used to resist seismic forces. *(Tier 2: Sec. 5.5.3.6.1; Commentary: Sec. A.3.2.7.4)*	Several of the walls have an aspect ratio above 2-1.
		X		WALLS CONNECTED THROUGH FLOORS: Shear walls have an interconnection between stories to transfer overturning and shear forces through the floor. *(Tier 2: Sec. 5.5.3.6.2; Commentary: Sec. A.3.2.7.5)*	
		X		HILLSIDE SITE: For structures that are taller on at least one side by more than one-half story because of a sloping site, all shear walls on the downhill slope have an aspect ratio less than 1-to-2. *(Tier 2: Sec. 5.5.3.6.3; Commentary: Sec. A.3.2.7.6)*	Wood shearwalls only exist above grade.
		X		CRIPPLE WALLS: Cripple walls below first-floor-level shear walls are braced to the foundation with wood structural panels. *(Tier 2: Sec. 5.5.3.6.4; Commentary: Sec. A.3.2.7.7)*	
	X			OPENINGS: Walls with openings greater than 80% of the length are braced with wood structural panel shear walls with aspect ratios of not more than 1.5-to-1 or are supported by adjacent construction through positive ties capable of transferring the seismic forces. *(Tier 2: Sec. 5.5.3.6.5; Commentary: Sec. A.3.2.7.8)*	Various locations have shear walls with larger aspect ratios.
	X			HOLD-DOWN ANCHORS: All shear walls have hold-down anchors attached to the end studs constructed in accordance with acceptable construction practices. *(Tier 2: Sec. 5.5.3.6.6; Commentary: Sec. A.3.2.7.9)*	Not compliant at all shear wall locations.

17-7. Immediate Occupancy Checklist for Building Type W2

Connections

C	NC	N/A	U	EVALUATION STATEMENT	COMMENT
X				WOOD POSTS: There is a positive connection of wood posts to the foundation. *(Tier 2: Sec. 5.7.3.3; Commentary: Sec. A.5.3.3)*	
X				WOOD SILLS: All wood sills are bolted to the foundation. *(Tier 2: Sec. 5.7.3.3; Commentary: Sec. A.5.3.4)*	
X				GIRDER–COLUMN CONNECTION: There is a positive connection using plates, connection hardware, or straps between the girder and the column support. *(Tier 2: Sec. 5.7.4.1; Commentary: Sec. A.5.4.1)*	

Foundation System

C	NC	N/A	U	EVALUATION STATEMENT	COMMENT
		X		DEEP FOUNDATIONS: Piles and piers are capable of transferring the lateral forces between the structure and the soil. *(Commentary: Sec. A.6.2.3)*	Foundations are speard footings, not pile and piers.
		X		SLOPING SITES: The difference in foundation embedment depth from one side of the building to another does not exceed one story. *(Commentary: A.6.2.4)*	

Low, Moderate, and High Seismicity

(Complete the Following Items in Addition to the Items for Very Low Seismicity)

Seismic-Force-Resisting System

C	NC	N/A	U	EVALUATION STATEMENT	COMMENT
	X			NARROW WOOD SHEAR WALLS: Narrow wood shear walls with an aspect ratio greater than 1.5-to-1 are not used to resist seismic forces. *(Tier 2: Sec. 5.5.3.6.1; Commentary: Sec. A.3.2.7.4)*	Various locations have shear walls with larger aspect ratios.

Diaphragms

C	NC	N/A	U	EVALUATION STATEMENT	COMMENT
X				DIAPHRAGM CONTINUITY: The diaphragms are not composed of split-level floors and do not have expansion joints. *(Tier 2: Sec. 5.6.1.1; Commentary: Sec. A.4.1.1)*	
	X			ROOF CHORD CONTINUITY: All chord elements are continuous, regardless of changes in roof elevation. *(Tier 2: Sec. 5.6.1.1; Commentary: Sec. A.4.1.3)*	
		X		DIAPHRAGM REINFORCEMENT AT OPENINGS: There is reinforcing around all diaphragm openings larger than 50% of the building width in either major plan dimension. *(Tier 2: Sec. 5.6.1.5; Commentary: Sec. A.4.1.8)*	No diaphragm openings larger than 50% of the building width.
		X		STRAIGHT SHEATHING: All straight-sheathed diaphragms have aspect ratios less than 1-to-1 in the direction being considered. *(Tier 2: Sec. 5.6.2; Commentary: Sec. A.4.2.1)*	Diaphragm consists of plywood sheathing

17-7. Immediate Occupancy Checklist for Building Type W2

C	NC	N/A	U	EVALUATION STATEMENT	COMMENT
X				SPANS: All wood diaphragms with spans greater than 12 ft (3.6 m) consist of wood structural panels or diagonal sheathing. *(Tier 2: Sec. 5.6.2; Commentary: Sec. A.4.2.2)*	
		X		DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 30 ft (9.1 m) and aspect ratios less than or equal to 3-to-1. *(Tier 2: Sec. 5.6.2; Commentary: Sec. A.4.2.3)*	Diaphragm is blocked plywood sheathing.
		X		OTHER DIAPHRAGMS: The diaphragms do not consist of a system other than wood, metal deck, concrete, or horizontal bracing. *(Tier 2: Sec. 5.6.5; Commentary: Sec. A.4.7.1)*	

Connections

C	NC	N/A	U	EVALUATION STATEMENT	COMMENT
X				WOOD SILL BOLTS: Sill bolts are spaced at 4 ft or less with acceptable edge and end distance provided for wood and concrete. *(Tier 2: Sec. 5.7.3.3; Commentary: Sec. A.5.3.7)*	

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

17-34 Collapse Prevention Structural Checklist for Building Types RM1 and RM2

Low and Moderate Seismicity

Seismic-Force-Resisting System

C	NC	N/A	U	EVALUATION STATEMENT	COMMENT
X				REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2. *(Tier 2: Sec. 5.5.1.1; Commentary: Sec. A.3.2.1.1)*	
	X			SHEAR STRESS CHECK: The shear stress in the reinforced masonry shear walls, calculated using the Quick Check procedure of Section 4.4.3.3, is less than 70 lb/in.^{2} (0.48 MPa). *(Tier 2: Sec. 5.5.3.1.1; Commentary: Sec. A.3.2.4.1)*	Shear stress is 222psi for masonry shear walls. Interaction with the wood shear wall portion of the building not considered.
X				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. (1220 mm), and all vertical bars extend to the top of the walls. *(Tier 2: Sec. 5.5.3.1.3; Commentary: Sec. A.3.2.4.2)*	

Stiff Diaphragms

C	NC	N/A	U	EVALUATION STATEMENT	COMMENT
		X		TOPPING SLAB: Precast concrete diaphragm elements are interconnected by a continuous reinforced concrete topping slab. *(Tier 2: Sec. 5.6.4; Commentary: Sec. A.4.5.1)*	This is a flexible diapharagm.

Connections

C	NC	N/A	U	EVALUATION STATEMENT	COMMENT
	X			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 have strength to resist the connection force calculated in the Quick Check procedure of Section 4.4.3.7. *(Tier 2: Sec. 5.7.1.1; Commentary: Sec. A.5.1.1)*	Detail 9/S3.3 shows for wall out of plane bracing. Connection is not adequate.
X				WOOD LEDGERS: The connection between the wall panels and the diaphragm does not induce cross-grain bending or tension in the wood ledgers. *(Tier 2: Sec. 5.7.1.3; Commentary: Sec. A.5.1.2)*	
	X			TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls. *(Tier 2: Sec. 5.7.2; Commentary: Sec. A.5.2.1)*	
		X		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. *(Tier 2: Sec. 5.7.2; Commentary: Sec. A.5.2.)*	
X				FOUNDATION DOWELS: Wall reinforcement is doweled into the foundation. *(Tier 2: Sec. 5.7.3.4; Commentary: Sec. A.5.3.5)*	

17-34 Collapse Prevention Structural Checklist for Building Types RM1 and RM2

C	NC	N/A	U	EVALUATION STATEMENT	COMMENT
X				GIRDER–COLUMN CONNECTION: There is a positive connection using plates, connection hardware, or straps between the girder and the column support. *(Tier 2: Sec. 5.7.4.1; Commentary: Sec. A.5.4.1)*	

High Seismicity

(Complete the Following Items in Addition to the Items for Low and Moderate Seismicity)

Stiff Diaphragms

C	NC	N/A	U	EVALUATION STATEMENT	COMMENT
		X		OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 25% of the wall length. *(Tier 2: Sec. 5.6.1.3; Commentary: Sec. A.4.1.4)*	
		X		OPENINGS AT EXTERIOR MASONRY SHEAR WALLS: Diaphragm openings immediately adjacent to exterior masonry shear walls are not greater than 8 ft (2.4 m) long. *(Tier 2: Sec. 5.6.1.3; Commentary: Sec. A.4.1.6)*	

Flexible Diaphragms

C	NC	N/A	U	EVALUATION STATEMENT	COMMENT
	X			CROSS TIES: There are continuous cross ties between diaphragm chords. *(Tier 2: Sec. 5.6.1.2; Commentary: Sec. A.4.1.2)*	
		X		OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 25% of the wall length. *(Tier 2: Sec. 5.6.1.3; Commentary: Sec. A.4.1.4)*	
		X		OPENINGS AT EXTERIOR MASONRY SHEAR WALLS: Diaphragm openings immediately adjacent to exterior masonry shear walls are not greater than 8 ft (2.4 m) long. *(Tier 2: Sec. 5.6.1.3; Commentary: Sec. A.4.1.6)*	
		X		STRAIGHT SHEATHING: All straight-sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered. *(Tier 2: Sec.* 5.6.2*; Commentary: Sec.* A.4.2.1)	
		X		SPANS: All wood diaphragms with spans greater than 24 ft (7.3 m) consist of wood structural panels or diagonal sheathing. *(Tier 2: Sec. 5.6.2; Commentary: Sec. A.4.2.2)*	
		X		DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 ft (12.2 m) and aspect ratios less than or equal to 4-to-1. *(Tier 2: Sec. 5.6.2; Commentary: Sec. A.4.2.3)*	
X				OTHER DIAPHRAGMS: Diaphragms do not consist of a system other than wood, metal deck, concrete, or horizontal bracing. *(Tier 2: Sec. 5.6.5; Commentary: Sec. A.4.7.1)*	

17-34 Collapse Prevention Structural Checklist for Building Types RM1 and RM2

Connections

C	NC	N/A	U	EVALUATION STATEMENT	COMMENT
X				STIFFNESS OF WALL ANCHORS: 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. (3 mm) before engagement of the anchors. *(Tier 2: Sec. 5.7.1.2; Commentary: Sec. A.5.1.4)*	

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

17-35. Immediate Occupancy Structural Checklist for Building Types RM1 and RM2

Very Low Seismicity

Seismic-Force-Resisting System

C	NC	N/A	U	EVALUATION STATEMENT	COMMENT
X				REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2. *(Tier 2: Sec. 5.5.1.1; Commentary: Sec. A.3.2.1.1)*	
	X			SHEAR STRESS CHECK: The shear stress in the reinforced masonry shear walls, calculated using the Quick Check procedure of Section 4.4.3.3, is less than 70 lb/in^2. (4.83 MPa). *(Tier 2: Sec. 5.5.3.1.1; Commentary: Sec. A.3.2.4.1)*	Shear stress is 222psi for masonry shear walls. Interaction with the wood shear wall portion of the building not considered.
X				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. *(Tier 2: Sec. 5.5.3.1.3; Commentary: Sec. A.3.2.4.2)*	

Connections

C	NC	N/A	U	EVALUATION STATEMENT	COMMENT
	X			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 have strength to resist the connection force calculated in the Quick Check procedure of Section 4.4.3.7. *(Tier 2: Sec. 5.7.1.1; Commentary: Sec. A.5.1.1)*	Detail 9/S3.3 shows for wall out of plane bracing. Connection is not adequate.
X				WOOD LEDGERS: The connection between the wall panels and the diaphragm does not induce cross-grain bending or tension in the wood ledgers. *(Tier 2: Sec. 5.7.1.3; Commentary: Sec. A.5.1.2)*	
	X			TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls, and the connections are able to develop the lesser of the shear strength of the walls or diaphragms. *(Tier 2: Sec. 5.7.2; Commentary: Sec. A.5.2.1)*	
X				FOUNDATION DOWELS: Wall reinforcement is doweled into the foundation, and the dowels are able to develop the lesser of the strength of the walls or the uplift capacity of the foundation. *(Tier 2: Sec. 5.7.3.4; Commentary: Sec. A.5.3.5)*	
X				GIRDER–COLUMN CONNECTION: There is a positive connection using plates, connection hardware, or straps between the girder and the column support. *(Tier 2: Sec. 5.7.4.1; Commentary: Sec. A.5.4.1)*	

17-35. Immediate Occupancy Structural Checklist for Building Types RM1 and RM2

Stiff Diapghragms

C	NC	N/A	U	EVALUATION STATEMENT	COMMENT
		X		TOPPING SLAB: Precast concrete diaphragm elements are interconnected by a continuous reinforced concrete topping slab. *(Tier 2: Sec. 5.6.4; Commentary: Sec. A.4.5.1)*	
		X		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. *(Tier 2: Sec. 5.7.2; Commentary: Sec. A.5.2.3)*	

Foundation System

C	NC	N/A	U	EVALUATION STATEMENT	COMMENT
		X		DEEP FOUNDATIONS: Piles and piers are capable of transferring the lateral forces between the structure and the soil. *(Commentary: Sec.* A.6.2.3*)*	Foundations are speard footings, not pile and piers.
		X		SLOPING SITES: The difference in foundation embedment depth from one side of the building to another does not exceed one story. *(Commentary: Sec. A.6.2.4)*	

Low, Moderate, and High Seismicity

(Complete the Following Items in Addition to the Items for Very Low Seismicity)

Seismic-Force-Resisting System

C	NC	N/A	U	EVALUATION STATEMENT	COMMENT
X				REINFORCING AT WALL OPENINGS: All wall openings that interrupt rebar have trim reinforcing on all sides. *(Tier 2: Sec. 5.5.3.1.5; Commentary: Sec. A.3.2.4.3)*	
	X			PROPORTIONS: The height-to-thickness ratio of the shear walls at each story is less than 30. *(Tier 2: Sec. 5.5.3.1.2; Commentary: Sec. A.3.2.4.4)*	

Diapghragms (Stiff or Flexible)

C	NC	N/A	U	EVALUATION STATEMENT	COMMENT
		X		OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 15% of the wall length. *(Tier 2: Sec. 5.6.1.3; Commentary: Sec. A.4.1.4)*	
		X		OPENINGS AT EXTERIOR MASONRY SHEAR WALLS: Diaphragm openings immediately adjacent to exterior masonry shear walls are not greater than 4 ft (1.2 m) long. *(Tier 2: Sec. 5.6.1.3; Commentary: Sec. A.4.1.6)*	
		X		PLAN IRREGULARITIES: There is tensile capacity to develop the strength of the diaphragm at reentrant corners or other locations of plan irregularities. *(Tier 2: Sec. 5.6.1.4; Commentary: Sec. A.4.1.7)*	

17-35. Immediate Occupancy Structural Checklist for Building Types RM1 and RM2

C	NC	N/A	U	EVALUATION STATEMENT	COMMENT
		X		DIAPHRAGM REINFORCEMENT AT OPENINGS: There is reinforcing around all diaphragm openings larger than 50% of the building width in either major plan dimension. *(Tier 2: Sec. 5.6.1.5; Commentary: Sec. A.4.1.8)*	This could not be observed, and is not detailed in the existing drawings.

Flexible Diapghragms

C	NC	N/A	U	EVALUATION STATEMENT	COMMENT
	X			CROSS TIES: There are continuous cross ties between diaphragm chords. *(Tier 2: Sec. 5.6.1.2; Commentary: Sec. A.4.1.2)*	
		X		STRAIGHT SHEATHING: All straight-sheathed diaphragms have aspect ratios less than 1-to-1 in the direction being considered. *(Tier 2: Sec. 5.6.2; Commentary: Sec. A.4.2.1)*	
		X		SPANS: All wood diaphragms with spans greater than 12 ft (3.6 m) consist of wood structural panels or diagonal sheathing. *(Tier 2: Sec. 5.6.2; Commentary: Sec. A.4.2.2)*	
		X		DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 30 ft (9.2 m) and aspect ratios less than or equal to 3-to-1. *(Tier 2: Sec. 5.6.2; Commentary: Sec. A.4.2.3)*	
	X			NONCONCRETE FILLED DIAPHRAGMS: Untopped metal deck diaphragms or metal deck diaphragms with fill other than concrete consist of horizontal spans of less than 40 ft (12.2 m) and have aspect ratios less than 4-to-1. *(Tier 2: Sec. 5.6.3; Commentary: Sec. A.4.3.1)*	Spans greater than 40 ft.
X				OTHER DIAPHRAGMS: Diaphragms do not consist of a system other than wood, metal deck, concrete, or horizontal bracing. *(Tier 2: Sec. 5.6.5; Commentary: Sec. A.4.7.1)*	

Connections

C	NC	N/A	U	EVALUATION STATEMENT	COMMENT
	X			STIFFNESS OF WALL ANCHORS: 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. *(Tier 2: Sec. 5.7.1.2; Commentary: Sec. A.5.1.4)*	Detail 9/S3.3 shows for wall out of plane bracing. Connection is not adequate.

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

COMMUNITY CENTER WEIGHT TAKE-OFF

ROTUNDA SECTION

ROOF | SKYLIGHT SYSTEM

SKYLIGHT (ASSUMED)	=	8.0 psf
FRAMING (ASSUMED)	=	5.0 psf
AREA OF SKYLIGHT	=	1072 ft^2

LOBBY ROOF

METAL ROOFING (ASSUMED 1½" 20GA)	=	2.9 psf
SHEATHING (5/8" PLYWOOD ASSUMED)	=	1.8 psf
R-30 BATT INSULATION	=	3.8 psf
GYPSUM W.B. (5/8" ASSUMED)	=	2.8 psf
WOOD PANEL ACOUSTIC CEILING	=	1.5 psf
STEEL FRAMING	=	30.0 psf
WOOD FRAMING	=	18.0 psf
MISC. MECHANICAL	=	5.0 psf
AREA OF LOBBY ROOF	=	1910 ft^2

LOBBY WALL

BRICK VENEER (ASSUMED)	=	12.0 psf
SHEATHING - ½" PLYWOOD	=	1.5 psf *
WOOD FRAMING - 2x6 STUDS @ 16" O.C.	=	1.7 psf
R-19 INSULATION	=	2.8 psf
GYPSUM W.B. (5/8" PLYWOOD ASSUMED)	=	1.8 psf
AREA OF LOBBY WALL	=	3695 ft^2

$\boxed{\text{TOTAL WEIGHT OF ROTUNDA} = 214k}$

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Client: CITY OF TUKWILA
Project: TUKWILA SEISMIC SCREENING COMMUNITY CENTER
Project No. 262007.023
Sheet ___ of ___
Design by SRK
Date 9/07
Checked by CMH
Date ___

WEST SECTION

ROOF SYSTEM

METAL ROOFING (1½" 20 GA ASSUMED)	=	2.9 psf
SHEATHING - ½" PLYWOOD	=	1.5 psf
14" TJI/35C @ 24" o.c.	=	2.0 psf
R-30 BATT. INSULATION	=	3.8 psf
5/8" GYPSUM W.B.	=	2.8 psf
ACOUSTICAL TILE	=	0.8 psf
STEEL & WOOD FRAMING (ASSUMED)	=	5.0 psf
MISC. MECHANICAL	=	5.0 psf
INTERIOR PARTITION	=	10.0 psf
AREA OF WEST ROOF	=	19774 ft^2

WALL SYSTEM (h = 9.5 ft, L = 574 ft)

BRICK VENEER (ASSUMED)	=	12.0 psf
SHEATHING - ½" PLYWOOD	=	1.5 psf
WOOD FRAMING - 2x6 @ 16" o.c.	=	1.7 psf
R-19 BATT INSULATION	=	2.8 psf
5/8" GYPSUM W.B.	=	2.8 psf
AREA OF WALL	=	2727 ft^2

$\boxed{\text{TOTAL WEIGHT OF WEST SECTION} = 725 k}$

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Client CITY OF TUKWILA
Project TUKWILA SEISMIC SCREENING COMMUNITY CENTER
Project No. 262007.023
Sheet ___ of ___
Design by SRK
Date 9/07
Checked by CMH
Date ___

GYMNASIUM

ROOF SYSTEM

METAL ROOFING (1½" 18 GA)	=	2.9 psf
COVER BOARD (ASSUME ½" PLYWOOD)	=	1.5 psf
RIGID INSULATION (ASSUME R-30)	=	14.0 psf
STEEL TRUSSES	=	3.0 psf
MISC. MECHANICAL	=	5.0 psf
AREA OF GYM ROOF	=	13309 ft^2

WALL SYSTEM

WOOD SIDING	=	3.0 psf
SHEATHING (ASSUME ½")	=	1.5 psf
RIGID INSULATION (ASSUME R-19)	=	4.7 psf
12" CMU	=	75.0 psf
AREA OF GYM WALL	=	7581 ft^2

$\boxed{\text{TOTAL WEIGHT OF GYM} = 990\text{ k}}$

Client	CITY OF TUKWILA	Sheet ___ of ___
Project	TUKWILA SEISMIC SCREENING COMMUNITY CENTER	Design by SRK
		Date 9/07
		Checked by CMH
Project No.	262007.023	Date

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EAST SECTION

LOCKER ROOM ROOF

METAL ROOFING (1½" 18 GA)	=	2.9 psf
COVER BOARD (ASSUME ½")	=	1.5 psf
SHEATHING (ASSUME ½" PLYWOOD)	=	1.5 psf
FRAMING (14" TJI/35C @ 24" O.C.)	=	2.0 psf
R-30 BATT INSULATION	=	3.8 psf
5/8" GYPSUM W.B.	=	2.8 psf
MISC. MECHANICAL	=	5.0 psf
AREA OF L.R. ROOF	=	2581 ft^2

BAY ROOF

BUILT-UP ROOF	=	6.5 psf
COVER BOARD (ASSUME ½" PLYWOOD)	=	1.5 psf
TAPERED RIGID INSULATION R-30	=	4.7 psf
SHEATHING (ASSUME ½")	=	1.5 psf
WOOD DECK (2x T&G)	=	4.3 psf
GLU-LAM & STEEL BEAMS (ASSUMED)	=	5.0 psf
MISC. MECHANICAL	=	5.0 psf
AREA OF BAY ROOF	=	6873 ft^2

EXTERIOR WALL

WOOD SIDING	=	3.0 psf
SHEATHING ½" PLYWOOD	=	1.5 psf
2x6 WOOD STUDS @16" O.C.	=	1.7 psf
R-19 BATT. INSULATION	=	2.8 psf
5/8" GYPSUM W.B.	=	2.8 psf
AREA OF EXTERIOR WALL	=	2922 ft^2

INTERIOR PARTIONS = 10.0 psf

$\boxed{\text{TOTAL WEIGHT OF EAST SECTION} = 375\text{ k}}$

Client: CITY OF TUKWILA
Project: TUKWILA SEISMIC SCREENING COMMUNITY CENTER
Project No. 262007.023

Sheet ____ of ____
Design by SRK
Date 9/07
Checked by ____
Date ____

COMMUNITY CENTER LENGTH OF SHEARWALLS

WEST SECTION

X-DIRECTION:

$L_x = 4.5' + 6.5' + 6.5' + 4.5' + 6' + 6' + 4' + 7' + 7' + 9.5' + 16.5' + 5.5' + 5.5' + 4' + 5.5' + 5.5' + 4' + 3.5' + 19.5' + 8' + 7' + 7' + 11.5' + 7' + 4.5'$

$L_x = 176\text{ft}$

Y-DIRECTION:

$L_y = 2.5' + 5' + 5.5' + 5' + 5' + 5.5' + 2.5' + 3.5' + 6.5' + 5.5' + 3' + 9' + 9' + 8' + 6' + 11.5' + 26' + 8' + 19' + 13' + 27' + 27'$

$L_y = 213\text{ft}$

EAST SECTION

X-DIRECTION:

$L_x = 2.5' + 2.5' + 2.5' + 40' + 23' + 4' + 4' + 4' + 4' + 2.5'$

$L_x = 89\text{ft}$

$L_y = 27' + 27'$

$L_y = 54\text{ft}$

*NOTE: X & Y DIRECTION IS RELATIVE TO THE PLAN SHEETS.

$\boxed{L_{x,tot} = 265\text{ft} \quad L_{y,tot} = 267\text{ft}}$

Client	City of Tukwila
Project	Seismic Screening Community Center
Project No.	262022.017
Sheet	of
Design by	JDJ
Date	4-5-22
Checked by	
Date	

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Community Center - Wood Shear Wall Sections

Shear Stress Check

$$V_j^{AVG} = \frac{1}{m}\left(\frac{V_j}{L_w}\right)$$

- X - DIRECTION $\quad V_{j\,IO} = 1018k \quad V_{j\,LS} = 2423k$

$m_{IO} = 1.7 \quad m_{LS} = 3.8$

$V_{j\,IO} = \frac{1}{1.7}\left(\frac{1018k}{265ft}\right) = 2260\,plf$ ∴ NOT COMPLIANT

$V_{j\,LS} = \frac{1}{3.8}\left(\frac{2423k}{265ft}\right) = 2406\,plf$ ∴ NOT COMPLIANT

SW1 EXPECTED	480plf x 0.9 x 1.5 =	648plf	DCR = 3.71
SW2 EXPECTED	700plf x 0.9 x 1.5 =	945plf	DCR = 2.55
SW3 EXPECTED	900plf x 0.9 x 1.5 =	1215plf	DCR = 1.98

- Y - DIRECTION

$V_{j\,IO} = \frac{1}{1.7}\left(\frac{1018k}{267ft}\right) = 2243\,plf$ ∴ NOT COMPLIANT

$V_{j\,LS} = \frac{1}{3.8}\left(\frac{2423k}{267ft}\right) = 2388\,plf$ ∴ NOT COMPLIANT

SW1 EXPECTED	648 plf	DCR = 3.69
SW2 EXPECTED	945 plf	DCR = 2.53
SW3 EXPECTED	1215 plf	DCR = 1.97

Client: CITY OF TUKWILA
Project: SEISMIC SCREENING COMMUNITY CENTER
Project No. 262022.017

Sheet ___ of ___
Design by JDJ
Date 4-5-22
Checked by ___
Date ___

COMMUNITY CENTER - CMU SECTION

SHEAR STRESS CHECK - 12" CMU

$v_j^{AVG} = \frac{1}{m}\left(\frac{V_j}{A_w}\right)$

$A_w = 54\,in^2/ft \times 38\,ft = 2052\,in^2$

$V_{j\,IO} = 697k$ $\quad V_{j\,LS} = 1434k$ $\quad m_{IO} = 1.5$ $\quad m_{LS} = 3.0$

$v_{j\,IO} = \frac{1}{1.5}\left(\frac{697k}{2052\,in^2}\right) = 226\,psi$ $\quad \therefore$ NOT COMPLIANT

$v_{j\,LS} = \frac{1}{3.0}\left(\frac{1434k}{2052\,in^2}\right) = 233\,psi$ $\quad \therefore$ NOT COMPLIANT

EXPECTED = 70psi $\quad$ DCR = 3.33

SHEAR STRESS CHECK - 8" CMU

$A_w = 41\,in^2/ft \times 38\,ft = 1558\,in^2$

$V_{j\,IO} = 697k$ $\quad V_{j\,LS} = 1434k$ $\quad m_{IO} = 1.5$ $\quad m_{LS} = 3.0$

$v_{j\,IO} = \frac{1}{1.5}\left(\frac{697k}{1558\,in^2}\right) = 298\,psi$ $\quad \therefore$ NOT COMPLIANT

$v_{j\,LS} = \frac{1}{3.0}\left(\frac{1434k}{1558\,in^2}\right) = 307\,psi$ $\quad \therefore$ NOT COMPLIANT

EXPECTED = 70psi $\quad$ DCR = 4.39

Client CITY OF TUKWILA — Sheet ___ of ___
Project SEISMIC SCREENING COMMUNITY CENTER — Design by SRK — Date 9/07
Checked by PNC — Date ___
Project No. 262007.023

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REINFORCEMENT CHECK - 8" CMU

AREA OF CMU = 41 in^2/ft

AREA OF REIN. = 0.0775 in^2/ft VERT #5@48"
= 0.1 in^2/ft HORIZ. (2)#4@48"

$$\rho_V = \frac{0.0775\ in^2/ft}{41\ in^2/ft} = 0.0019 \quad \therefore \text{COMPLIANT}$$

$$\rho_H = \frac{0.1\ in^2/ft}{41\ in^2/ft} = 0.0024 \quad \therefore \text{COMPLIANT}$$

∴ TOTAL HORIZ. & VERT REINFORCEMENT IS COMPLIANT

REINFORCEMENT CHECK - 12" CMU

AREA OF CMU = 72.1 in^2/ft

AREA OF REIN. = 0.31 in^2/ft VERT (2)#5@24"
= 0.155 in^2/ft HORIZ (2)#5@48"

$$\rho_V = \frac{0.31\ in^2/ft}{72.1\ in^2/ft} = 0.0043 \quad \therefore \text{COMPLIANT}$$

$$\rho_H = \frac{0.155\ in^2/ft}{72.1\ in^2/ft} = 0.0021 \quad \therefore \text{COMPIANT}$$

∴ TOTAL HORIZ. & VERT. REINFORCEMENT IS COMPLIANT

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Client	City of Tukwila	Sheet	of
Project	Community Center - Wood Portion	Design by	JDJ
		Date	4/5/2022
		Checked	
Project No.	262022.017	Date	

ASCE 41-17: LINEAR STATIC PROCEDURE (SEC. 7.4.1)

I.D.:

MAPPED SPECTRAL RESPONSE ACCELERATION:				Ref:
BSE-2E mapped short period accel.:E	S_{S2M} =	1.10 g		2.4.1.3
BSE-2E mapped accel. @ T=1 s:	S_{12M} =	0.37 g		2.4.1.3
BSE-1E mapped short period accel.:	S_{S1M} =	0.51 g		2.4.1.4
BSE-1E mapped accel. @ T=1 s:	S_{11M} =	0.16 g		2.4.1..4
BSE-2N mapped short period accel.:	S_{S2NM} =	1.51 g		2.4.1.1
BSE-2N mapped accel. @ T=1 s:	S_{12NM} =	0.51 g		2.4.1.1
BSE-2E controlling short period accel.:	S_{S2} = MIN(S_{S2M},S_{S2NM}) =		1.1 g	2.4.1.3
BSE-2E controlling accel. @ T=1 s:	S_{12} = MIN(S_{12M},S_{12NM}) =		0.37 g	2.4.1.3
BSE-1E controlling short period accel.:	S_{S1} = MIN(S_{S1M},2/3*S_{S2NM}) =		0.506 g	2.4.1.4
BSE-1E controlling accel. @ T=1 s:	S_{11} = MIN(S_{11M},2/3*S_{12NM}) =		0.157 g	2.4.1.4

MODIFIED SPECTRAL RESPONSE PARAMETERS:				Ref:
Site class:		D		2.4.1.6
BSE-2E acceleration site coefficient:	F_{a2} =	1.20		Table 2-3
BSE-2E velocity site coefficient:	F_{v2} =	1.93		Table 2-4
BSE-1E acceleration site coefficient:	F_{a1} =	1.40		Table 2-3
BSE-1E velocity site coefficient:	F_{v1} =	2.29		Table 2-4
BSE-2N acceleration site coefficient:	F_{a2N} =	1.00		2.5/2.4.1.6
BSE-2N velocity site coefficient:	F_{v2N} =	1.50		2.5/2.4.1.6
BSE-2E design short period accel.:	S_{XS2} = F_{a2}*S_{S2} =		1.32 g	2.4.1.6
BSE-2E design 1 sec. period accel.:	S_{X12} = F_{v2}*S_{12} =		0.71 g	2.4.1.6
BSE-1E design short period accel.:	S_{XS1} = F_{a1}*S_{S1} =		0.71 g	2.4.1.6
BSE-1E design 1 sec. period accel.:	S_{X11} = F_{v1}*S_{11} =		0.36 g	2.4.1.6
ASCE 7 design short period accel:	S_{DS} = 2/3*F_{a2N}*S_{S2NM} =		1.01 g	2.5
ASCE 7 design 1 sec. period accel:	S_{D1} = 2/3*F_{v1N}*S_{12NM} =		0.51 g	2.5
Seismicity zone:	Zone of seismicity is		**HIGH**	2.5

RESPONSE SPECTRA CHARACTERISTIC PERIODS:			Ref:
BSE-2E spectra:	T_{S2} = S_{X12}/(S_{XS2}) =	0.54 s	2.4.1.7.1
	T_{02} = 0.2*T_{S2} =	0.11 s	2.4.1.7.1
BSE-1E spectra:	T_{S1} = S_{X11}/(S_{XS1}) =	0.51 s	2.4.1.7.1
	T_{01} = 0.2*T_{S1} =	0.10 s	2.4.1.7.1

STRUCTURE DYNAMIC PROPERTIES:				Ref:
Building seismic weight:	W =	1,314 k		7.4.1.3
Number of stories:	n =	1		7.4.1.3
Effective damping ratio:	β =	5 %		7.2.3.6
Damping coefficients:	B_1 =	1.0		2.4.1.7.1
Lateral system:	Wood buildings			7.4.1.2.2
Period coefficient:	C_t =	0.020		7.4.1.2.2
Period exponent:	β =	0.75		7.4.1.2.2
Building height:	h_n =	14.5 ft		7.4.1.2.2
Calculated period	T_c =	s		7.4.1.2.1
Empirical period:	T_e = C_t*h_n^{β} =		0.15 s	7.4.1.2.2
Fundamental period:	T =	0.15 s		7.4.1.2.2
m_{max} @ BSE-2E:	m_{max2}	3.8		7.4.1.3.1
m_{max} @ BSE-1E:	m_{max1}	1.7		7.4.1.3.1

PSEUDO-LATERAL LOAD:					Ref:
BSE-2E spectral acceleration:	S_{a2} =	1.317 g			2.4.1.7.1
BSE-1E spectral acceleration:	S_{a1} =	0.704 g			2.4.1.7.1
Effective mass factor:	C_m =	1.0			7.4.1.3.1
BSE-2E mod. factors product	C_{12}*C_{22} =	1.40			7.4.1.3.1
BSE-1E mod. factors product	C_{11}*C_{21} =	1.10			7.4.1.3.1
BSE-2E pseudo lateral load:	**V_2 =** $C_{12}C_{22}C_mS_{a2}W$ =		**1.8436 W =**	**2423 k**	7.4.1.3.1
BSE-1E pseudo lateral load:	**V_1 =** $C_{11}C_{21}C_mS_{a1}W$ =		**0.7746 W =**	**1018 k**	7.4.1.3.1

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Client	City of Tukwila	Sheet	of
Project	Community Center - Wood Portion	Design by	JDJ
		Date	4/5/2022
		Checked	
Project No.	262022.017	Date	

ASCE 41-17: LINEAR STATIC PROCEDURE (SEC. 7.4.1)

I.D.:

FORCE DISTRIBUTION CALCULATIONS:			Ref:
Story force:	$F_x = w_x{*}h_x^k/(\Sigma w_x{*}h_x^k){*}V =$	see table	7.4.1.3.2
	$k =$ IF(T<=0.5,1,IF(T>=2.5,2,1+(T-0.5)/2)) =	1.000	7.4.1.3.2
	$\Sigma w_x{*}h_x^k =$	19053	7.4.1.3.2
Diaphragm force:	$F_{px} = V_x{*}w_x/W_x =$	see table	7.4.1.3.4

Story Name	Story Weight w_x (k)	Story Height h_x (ft)	$w_x{*}h_x^k$	BSE-2E Story Force F_{x2} (k)	BSE-1E Story Force F_{x1} (k)	BSE-2E Story Shear V_{x2} (k)	BSE-1E Story Shear V_{x1} (k)	Total Weight Above W_x (k)	BSE-2E Diaph. Force F_{px2} (k)	BSE-1E Diaph. Force F_{px1} (k)
Roof	1,314	14.5	19053	2423	1018	2423	1018	1314	2423	1018

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Project	Community Center - Wood Portion	Design by	JDJ
		Date	4/5/2022
		Checked	
Project No.	262022.017	Date	

ASCE 41-17: LINEAR STATIC PROCEDURE (SEC. 7.4.1)

I.D.:

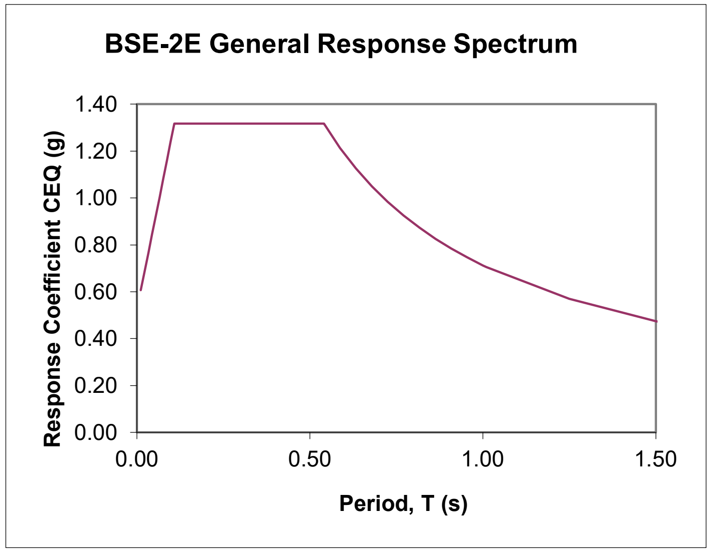
ACCELERATION RESPONSE SPECTRA:

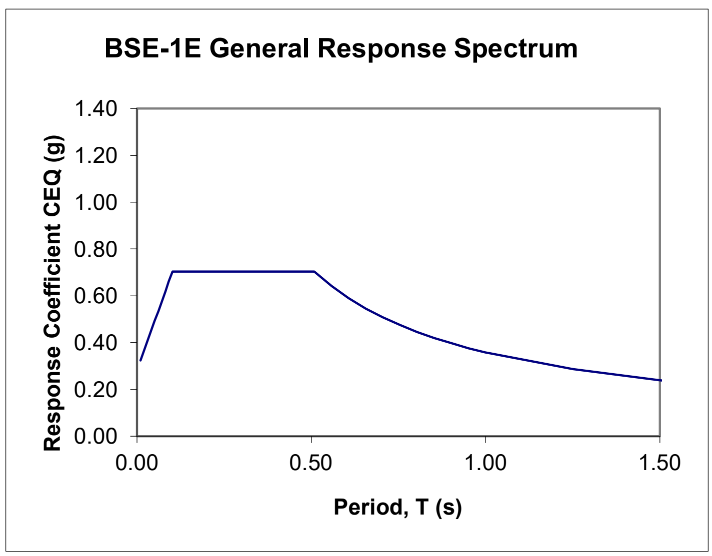
$C_{EQ} = C_1C_2C_MS_{XS}[(5/B_1\text{-}2)^*T/T_S+0.4]$ @ $T<= T_0$

$C_{EQ} = C_1C_2C_MS_{XS}/B_1$ @ $T_0<T<= T_S$

$C_{EQ} = C_1C_2C_MS_{X1}/(B_1^*T)$ @ $T>T_S$

	BSE-2E		BSE-1E	
	T (s)	C_{EQ} (g)	T (s)	C_{EQ} (g)
	0.01	0.61	0.01	0.32
	0.02	0.69	0.02	0.37
	0.03	0.76	0.03	0.41
	0.04	0.84	0.04	0.45
	0.05	0.92	0.05	0.49
	0.06	1.00	0.06	0.54
	0.08	1.08	0.07	0.58
	0.09	1.16	0.08	0.62
	0.10	1.24	0.09	0.66
T_0 =	0.11	1.32	0.10	0.70
	0.16	1.32	0.15	0.70
	0.22	1.32	0.20	0.70
	0.27	1.32	0.25	0.70
	0.32	1.32	0.31	0.70
	0.38	1.32	0.36	0.70
	0.43	1.32	0.41	0.70
	0.49	1.32	0.46	0.70
T_S =	0.54	1.32	0.51	0.70
	0.587	1.21	0.56	0.64
	0.633	1.13	0.61	0.59
	0.679	1.05	0.66	0.55
	0.725	0.98	0.71	0.51
	0.770	0.92	0.75	0.47
	0.816	0.87	0.80	0.45
	0.862	0.83	0.85	0.42
	0.908	0.78	0.90	0.40
	0.954	0.75	0.95	0.38
	1.00	0.71	1.00	0.36
	1.01	0.71	1.01	0.35
	1.25	0.57	1.25	0.29
	1.50	0.47	1.50	0.24
	1.75	0.41	1.75	0.20
	2.00	0.36	2.00	0.18
	3.00	0.24	3.00	0.12
	4.00	0.18	4.00	0.09





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Client	City of Tukwila	Sheet	of
Project	Community Center - Masonry Portion	Design by	JDJ
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		Checked	
Project No.	262022.017	Date	

ASCE 41-17: LINEAR STATIC PROCEDURE (SEC. 7.4.1)

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MAPPED SPECTRAL RESPONSE ACCELERATION:				**Ref:**
BSE-2E mapped short period accel.:E	S_{S2M} =	1.10 g		2.4.1.3
BSE-2E mapped accel. @ T=1 s:	S_{12M} =	0.37 g		2.4.1.3
BSE-1E mapped short period accel.:	S_{S1M} =	0.51 g		2.4.1.4
BSE-1E mapped accel. @ T=1 s:	S_{11M} =	0.16 g		2.4.1..4
BSE-2N mapped short period accel.:	S_{S2NM} =	1.51 g		2.4.1.1
BSE-2N mapped accel. @ T=1 s:	S_{12NM} =	0.51 g		2.4.1.1
BSE-2E controlling short period accel.:	S_{S2} =	MIN(S_{S2M},S_{S2NM}) =	1.1 g	2.4.1.3
BSE-2E controlling accel. @ T=1 s:	S_{12} =	MIN(S_{12M},S_{12NM}) =	0.37 g	2.4.1.3
BSE-1E controlling short period accel.:	S_{S1} =	MIN(S_{S1M},2/3*S_{S2NM}) =	0.506 g	2.4.1.4
BSE-1E controlling accel. @ T=1 s:	S_{11} =	MIN(S_{11M},2/3*S_{12NM}) =	0.157 g	2.4.1.4
MODIFIED SPECTRAL RESPONSE PARAMETERS:				**Ref:**
Site class:		D		2.4.1.6
BSE-2E acceleration site coefficient:	F_{a2} =	1.20		Table 2-3
BSE-2E velocity site coefficient:	F_{v2} =	1.93		Table 2-4
BSE-1E acceleration site coefficient:	F_{a1} =	1.40		Table 2-3
BSE-1E velocity site coefficient:	F_{v1} =	2.29		Table 2-4
BSE-2N acceleration site coefficient:	F_{a2N} =	1.00		2.5/2.4.1.6
BSE-2N velocity site coefficient:	F_{v2N} =	1.50		2.5/2.4.1.6
BSE-2E design short period accel.:	S_{XS2} =	F_{a2}*S_{S2} =	1.32 g	2.4.1.6
BSE-2E design 1 sec. period accel.:	S_{X12} =	F_{v2}*S_{12} =	0.71 g	2.4.1.6
BSE-1E design short period accel.:	S_{XS1} =	F_{a1}*S_{S1} =	0.71 g	2.4.1.6
BSE-1E design 1 sec. period accel.:	S_{X11} =	F_{v1}*S_{11} =	0.36 g	2.4.1.6
ASCE 7 design short period accel:	S_{DS} =	2/3*F_{a2N}*S_{S2NM} =	1.01 g	2.5
ASCE 7 design 1 sec. period accel:	S_{D1} =	2/3*F_{v1N}*S_{12NM} =	0.51 g	2.5
Seismicity zone:		Zone of seismicity is	**HIGH**	2.5
RESPONSE SPECTRA CHARACTERISTIC PERIODS:				**Ref:**
BSE-2E spectra:	T_{S2} =	S_{X12}/(S_{XS2}) =	0.54 s	2.4.1.7.1
	T_{02} =	0.2*T_{S2} =	0.11 s	2.4.1.7.1
BSE-1E spectra:	T_{S1} =	S_{X11}/(S_{XS1}) =	0.51 s	2.4.1.7.1
	T_{01} =	0.2*T_{S1} =	0.10 s	2.4.1.7.1
STRUCTURE DYNAMIC PROPERTIES:				**Ref:**
Building seismic weight:	W =	990 k		7.4.1.3
Number of stories:	n =	1		7.4.1.3
Effective damping ratio:	β =	5 %		7.2.3.6
Damping coefficients:	B_1 =	1.0		2.4.1.7.1
Lateral system:		Other		7.4.1.2.2
Period coefficient:	C_t =	0.020		7.4.1.2.2
Period exponent:	β =	0.75		7.4.1.2.2
Building height:	h_n =	38 ft		7.4.1.2.2
Calculated period	T_c =	s		7.4.1.2.1
Empirical period:	T_e =	C_t*h_n^{β} =	0.31 s	7.4.1.2.2
Fundamental period:	T =	0.31 s		7.4.1.2.2
m_{max} @ BSE-2E:	m_{max2}	3.0		7.4.1.3.1
m_{max} @ BSE-1E:	m_{max1}	1.5		7.4.1.3.1
PSEUDO-LATERAL LOAD:				**Ref:**
BSE-2E spectral acceleration:	S_{a2} =	1.317 g		2.4.1.7.1
BSE-1E spectral acceleration:	S_{a1} =	0.704 g		2.4.1.7.1
Effective mass factor:	C_m =	1.0		7.4.1.3.1
BSE-2E mod. factors product	C_{12}*C_{22} =	1.10		7.4.1.3.1
BSE-1E mod. factors product	C_{11}*C_{21} =	1.00		7.4.1.3.1
BSE-2E pseudo lateral load:	**V_2 =**	$C_{12}C_{22}C_mS_{a2}W$ =	**1.4486 W = 1434 k**	7.4.1.3.1
BSE-1E pseudo lateral load:	**V_1 =**	$C_{11}C_{21}C_mS_{a1}W$ =	**0.7042 W = 697 k**	7.4.1.3.1

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Phone: 425-741-3800

ASCE 41-17: LINEAR STATIC PROCEDURE (SEC. 7.4.1)

I.D.:

FORCE DISTRIBUTION CALCULATIONS:			**Ref:**
Story force:	$F_x = w_x*h_x^k/(\Sigma w_x*h_x^k)*V =$	see table	7.4.1.3.2
	$k =$ IF(T<=0.5,1,IF(T>=2.5,2,1+(T-0.5)/2)) =	1.000	7.4.1.3.2
	$\Sigma w_x*h_x^k =$	37620	7.4.1.3.2
Diaphragm force:	$F_{px} = V_x*w_x/W_x =$	see table	7.4.1.3.4

Story Name	Story Weight w_x (k)	Story Height h_x (ft)	$w_x*h_x^k$	BSE-2E Story Force F_{x2} (k)	BSE-1E Story Force F_{x1} (k)	BSE-2E Story Shear V_{x2} (k)	BSE-1E Story Shear V_{x1} (k)	Total Weight Above W_x (k)	BSE-2E Diaph. Force F_{px2} (k)	BSE-1E Diaph. Force F_{px1} (k)
Roof	990	38	37620	1434	697	1434	697	990	1434	697

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Suite 200, Everett, WA
Phone: 425-741-3800

Client	City of Tukwila	Sheet	of
Project	Community Center - Masonry Portion	Design by	JDJ
		Date	4/5/2022
		Checked	
Project No.	262022.017	Date	

ASCE 41-17: LINEAR STATIC PROCEDURE (SEC. 7.4.1)

I.D.:

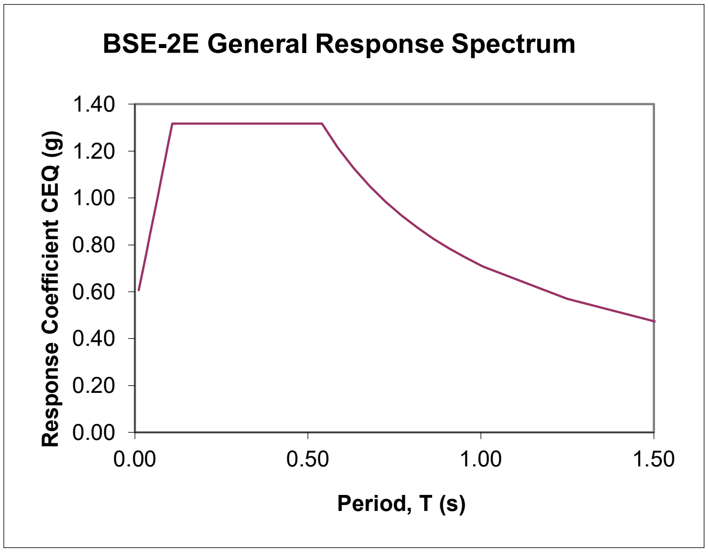
ACCELERATION RESPONSE SPECTRA:

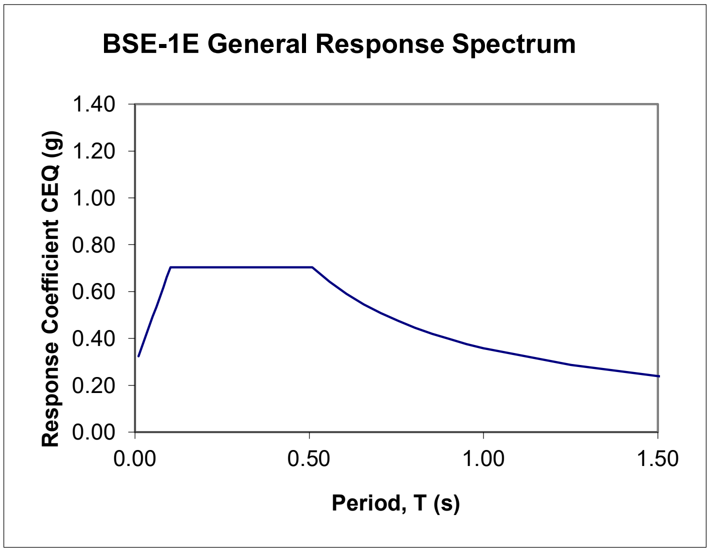
	BSE-2E		BSE-1E	
	T (s)	C_{EQ} (g)	T (s)	C_{EQ} (g)
	0.01	0.61	0.01	0.32
	0.02	0.69	0.02	0.37
	0.03	0.76	0.03	0.41
	0.04	0.84	0.04	0.45
	0.05	0.92	0.05	0.49
	0.06	1.00	0.06	0.54
	0.08	1.08	0.07	0.58
	0.09	1.16	0.08	0.62
	0.10	1.24	0.09	0.66
T_0 =	0.11	1.32	0.10	0.70
	0.16	1.32	0.15	0.70
	0.22	1.32	0.20	0.70
	0.27	1.32	0.25	0.70
	0.32	1.32	0.31	0.70
	0.38	1.32	0.36	0.70
	0.43	1.32	0.41	0.70
	0.49	1.32	0.46	0.70
T_S =	0.54	1.32	0.51	0.70
	0.587	1.21	0.56	0.64
	0.633	1.13	0.61	0.59
	0.679	1.05	0.66	0.55
	0.725	0.98	0.71	0.51
	0.770	0.92	0.75	0.47
	0.816	0.87	0.80	0.45
	0.862	0.83	0.85	0.42
	0.908	0.78	0.90	0.40
	0.954	0.75	0.95	0.38
	1.00	0.71	1.00	0.36
	1.01	0.71	1.01	0.35
	1.25	0.57	1.25	0.29
	1.50	0.47	1.50	0.24
	1.75	0.41	1.75	0.20
	2.00	0.36	2.00	0.18
	3.00	0.24	3.00	0.12
	4.00	0.18	4.00	0.09

$C_{EQ} = C_1C_2C_MS_{XS}[(5/B_1\text{-}2)*T/T_S+0.4]$ @ $T \le T_0$

$C_{EQ} = C_1C_2C_MS_{XS}/B_1$ @ $T_0 < T \le T_S$

$C_{EQ} = C_1C_2C_MS_{X1}/(B_1*T)$ @ $T > T_S$





Appendix B
Cost Estimate

City Hall
Collapse Prevention
Tukwila Seismic Improvement Program

Tukwila, WA

Conceptual Cost Estimate

June 20, 2022

Prepared for:

Reid Middleton

728 134th Street SW
Suite 200
Everett, WA 98204

PRODIMS
PROJECT MANAGEMENT SERVICES

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fax: (425) 828-0700
www.prodims.com

Estimate Summary

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Name:	**City Hall - Collapse Prevention**
Second Name:	**Tukwila Seismic Improvement Program**
Location:	**Tukwila, WA**
Design Phase:	**Concept Cost Estimate**
Date of Estimate:	**June 20, 2022**
Date of Revision:	
Month of Cost Basis:	**April, 2022**

Estimate Summary

	Percentage of Previous Subtotal		Amount			
				Subtotal Direct Cost	**$**	**2,976,204**
				Subtotal	$	2,976,204
Scope Contingency	15.0%	$	446,431			
				Subtotal	$	3,422,635
General Conditions	16.0%	$	547,622			
				Subtotal	$	3,970,257
Home Office Overhead	6.0%	$	238,215			
				Subtotal	$	4,208,472
Profit	6.0%	$	252,508			
				Subtotal	$	4,460,980
Escalation - Not Included - See Escalation in Table Below	0.00%	$	-			
				Subtotal	$	4,460,980

TOTAL ESTIMATED CONSTRUCTION COST in April, 2022 Dollars → $ 4,460,980

Escalation Table

Cost Estimate in April, 2022 Dollars from Above -> $ 4,460,980

Escalation to:	Out How Many Years	Rate at 6% per year		Escalation Total:	Mid-point of Construction Allowance:		Escalated Total:	Date:
April, 2023	1 year	6.00%	$	267,659	April, 2023	**$**	**4,728,639**	**April, 2023**
April, 2024	2 years	12.36%	$	551,377	April, 2024	**$**	**5,012,358**	**April, 2024**
April, 2025	3 years	19.10%	$	852,119	April, 2025	**$**	**5,313,099**	**April, 2025**
April, 2026	4 years	26.25%	$	1,170,905	April, 2026	**$**	**5,631,885**	**April, 2026**
April, 2027	5 years	33.82%	$	1,508,818	April, 2027	**$**	**5,969,798**	**April, 2027**

Estimate Assumptions:

This estimate is based on the As Built Markups and narrative information received by 6-10-22
This estimate is based on the working in an unoccupied building with no phasing or 2nd, 3rd shift work.
This estimate is based on the mechanical, electrical, plumbing and fire protection (M/E/P/FP) systems being modified to accommodate seismic work but M/E/P/FP systems are Not upgraded to latest building codes for these systems.
This estimate does not include any Hazardous Material Abatement Costs as it is not defined.
All soft costs are the owner's responsibility to determine and verify. The Soft costs estimate has been excluded from the construction cost estimate.
Escalation is allowed in the above table for 1 to 5 years out to the mid-point of construction as the construction schedule is still to be determined.

Estimate Qualifications:

The estimate is not be relied on solely for proforma development and financial decisions.
Additional Studies of additional systems impacted by the seismic scope of work should be performed before setting construction and project budgets.
Summary sheet markups are cumulative, not additive. Percentages are added to the previous subtotal rather than the direct cost subtotal.
Estimated labor is based on an 8 hour per day shift 5 days a week. Accelerated schedule work of overtime has not been included.
Estimated construction cost is for the entire project. This estimate is not intended to be used for other projects.
Division 0/ Division 1 specifications are presumed to have normal ranges for liquidated damages, construction schedule and terms & conditions. These divisions are typically written after the final estimate. Please contact the cost estimator for a review, if desired.
Please consult the cost estimator for any modifications to this estimate. Unilaterally adding and deleting markups, scope of work, schedule, specifications, plans and bid forms could incorrectly restate the project construction cost.
The construction cost estimate does not include an estimate of owner soft costs such as taxes, A/E fees, owner contingencies and permit fees.
Construction reserve contingency for change orders is not included in the estimate.
Any modifications to the plans via addendums and code review for permits will cause cost increases and are not included in this estimate.
Sole source supply of materials and/ or installers typically results in a 40% to 100% premium on costs over open specifications.
Imposition of tariffs and market instability of resources such as fuel, insurance and labor occurring after estimate date are not included.
Contractors imposing different bidding conditions from plans and specifications on subcontractors are not bidding from the plans and specifications.
Modifications to the proposed construction schedule and modifying the phasing plans after this estimate will affect construction cost and are not included.
The estimate includes a reasonable construction escalation that can be determined based on market conditions for up to the next 6 months.
Since this project has a midpoint of construction further than 6 months, increases in escalation are not included beyond the rate shown in the estimate.

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Name: **City Hall - Collapse Prevention**
Second Name: **Tukwila Seismic Improvement Program**
Location: **Tukwila, WA**
Design Phase: **Concept Cost Estimate**
Date of Estimate: **June 20, 2022**
Date of Revision:
Month of Cost Basis: **April, 2022**

Area	sqft
1st Floor	12,000
2nd Floor	14,000
	26,000

WBS	Description	Quantity	U of M	Labor	Labor Total	Material	Material Total	Equipment	Equipment Total	Total $/U of M	Direct Cost	Direct $/SQFT
A- Substructure												
A10- Foundations												
A1010- Standard Foundations												
	Enlarge Column Footings with Concrete, Drilled in Rebar Dowels, Formwork, Excavation and Backfill. Remove Restore Surface Treatment	9	each	$ 1,775.00	$ 15,975.00	$ 725.00	$ 6,525.00	$ 150.00	$ 1,350.00	$ 2,650.00	$ 23,850.00	
A1020- Special Foundations												
	Compaction Grouting for Ground Improvement for Seismic Mitigation - Work is performed under and outside of building and includes restoration of all building and site elements removed to perform the work.	26,250	sqft	$ 20.48	$ 537,600.00	$ 11.52	$ 302,400.00	$ 1.92	$ 50,400.00	$ 33.92	$ 890,400.00	
Totals	**A10- Foundations**										**$ 914,250.00**	**$ 35.16**
B- Shell												
B10- Superstructure												
B1010- Upper Floor												
	Steel - Braces for Continuous Load Path for Shear Walls Above	520	lnft	$ 85.00	$ 44,200.00	$ 40.00	$ 20,800.00	$ 7.50	$ 3,900.00	$ 132.50	$ 68,900.00	
	Upgrade Beam for Shearwall Above	1	each	$ 1,275.00	$ 1,275.00	$ 225.00	$ 225.00	$ 90.00	$ 90.00	$ 1,590.00	$ 1,590.00	
	Shear Walls - New and Retrofit Existing Walls - 2x Wood Framing, Sheathing Each Side	3,105	sqft	$ 7.80	$ 24,219.00	$ 4.20	$ 13,041.00	$ 0.72	$ 2,235.60	$ 12.72	$ 39,495.60	
	Seismic Straps Across Beam Line at Floor Joists at 4' o.c.	68	each	$ 101.40	$ 6,895.20	$ 28.60	$ 1,944.80	$ 7.80	$ 530.40	$ 137.80	$ 9,370.40	
	A35 Clip - Install from 2X Rim to 2X Plate	30	each	$ 24.70	$ 741.00	$ 13.30	$ 399.00	$ 2.28	$ 68.40	$ 40.28	$ 1,208.40	
B1020- Roof												
	Seismic Straps Between Roof Beams	90	lnft	$ 16.38	$ 1,474.20	$ 4.62	$ 415.80	$ 1.26	$ 113.40	$ 22.26	$ 2,003.40	
	Steel Bracing at Windows - X-Braces	146	lnft	$ 68.25	$ 9,964.50	$ 36.75	$ 5,365.50	$ 6.30	$ 919.80	$ 111.30	$ 16,249.80	
	Steel Drag Strut	24	lnft	$ 146.25	$ 3,510.00	$ 78.75	$ 1,890.00	$ 13.50	$ 324.00	$ 238.50	$ 5,724.00	
Totals	**B10- Superstructure**										**$ 144,541.60**	**$ 5.56**

WBS	Description	Quantity	U of M	Labor	Labor Total	Material	Material Total	Equipment	Equipment Total	Total $/U of M	Direct Cost	Direct $/SQFT
B20- Exterior Closure												
B2010- Exterior Walls												
	Remove and Replace Exterior Closure System to Install New Shear Wall System - Allowance	3,044	sqft	$ 13.20	$ 40,180.80	$ 10.80	$ 32,875.20	$ 1.44	$ 4,383.36	$ 25.44	$ 77,439.36	
B2020- Exterior Windows												
	Insulated Glazing "Storefront" Window System Remove and Replace to Install New Steel Bracing	1,168	sqft	$ 35.67	$ 41,662.56	$ 51.33	$ 59,953.44	$ 5.22	$ 6,096.96	$ 92.22	$ 107,712.96	
Totals	**B20- Exterior Closure**										**$ 185,152.32**	**$ 7.12**
B30- Roofing												
B3010- Roof Coverings												
	New Roofing System - Asphalt Composition Roofing System, Underlayment, Batt Insulation, Sheet Metal Flashing and Trim. Demo Existing Roofing System.	14,840	sqft	$ 13.34	$ 197,965.60	$ 9.66	$ 143,354.40	$ 1.38	$ 20,479.20	$ 24.38	$ 361,799.20	
B3020- Roof Openings												
	Install New Skylight System and Curb - 3'-4" x 7'-3" and Remove Existing Skylights	4	each	$ 933.68	$ 3,734.72	$ 1,188.32	$ 4,753.28	$ 127.32	$ 509.28	$ 2,249.32	$ 8,997.28	
Totals	**B30- Roofing**										**$ 370,796.48**	**$ 14.26**
C- Interiors												
C10- Interior Construction												
C1010- Interior Partitions												
	Remove and Reinstall Walls, Doors, Specialties and Casework as required for New Structural Seismic Work	26,000	sqft	$ 3.48	$ 90,402.00	$ 2.22	$ 57,798.00	$ 0.34	$ 8,892.00	$ 6.04	$ 157,092.00	
Totals	**C10- Interior Construction**										**$ 157,092.00**	**$ 6.04**
C30- Interior Finishes												
C3010- Interior Wall Finishes												
	Restore Wall Finishes-Including Painting, Tile, Bases and Specialty Finishes as required for New Structural Seismic Work	26,000	sqft	$ 2.41	$ 62,647.00	$ 1.54	$ 40,053.00	$ 0.24	$ 6,162.00	$ 4.19	$ 108,862.00	
C3020- Interior Floor Finishes												
	Restore Floor Finishes-Including Carpet, Tile, LVT and Specialty Finishes as required for New Structural Seismic Work	26,000	sqft	$ 1.80	$ 46,787.00	$ 1.15	$ 29,913.00	$ 0.18	$ 4,602.00	$ 3.13	$ 81,302.00	
C3030- Interior Ceiling Finishes												
	Restore Ceiling Finishes - Including ACT, GWB and Specialty Finishes as required for New Structural Seismic Work	26,000	sqft	$ 3.02	$ 78,444.60	$ 1.43	$ 37,255.40	$ 0.27	$ 6,942.00	$ 4.72	$ 122,642.00	
Totals	**C30- Interior Finishes**										**$ 312,806.00**	**$ 12.03**

WBS	Description	Quantity	U of M	Labor	Labor Total	Material	Material Total	Equipment	Equipment Total	Total $/U of M	Direct Cost	Direct $/SQFT
D- Services												
D20- Plumbing Systems												
D2010- Plumbing System												
	Allowance For Modifications to Plumbing Systems as required for New Structural Seismic Work	26,000	sqft	$ 1.80	$ 46,787.00	$ 1.15	$ 29,913.00	$ 0.18	$ 4,602.00	$ 3.13	$ 81,302.00	
Totals	**D20- Plumbing Systems**										**$ 81,302.00**	**$ 3.13**
D30- HVAC Systems												
D3020- HVAC System												
	Allowance for HVAC work as required for New Structural Seismic Work	26,000	sqft	$ 10.89	$ 283,101.00	$ 6.96	$ 180,999.00	$ 1.07	$ 27,846.00	$ 18.92	$ 491,946.00	
Totals	**D30- HVAC Systems**										**$ 491,946.00**	**$ 18.92**
D40- Fire Protection Systems												
D4010- Fire Sprinkler System												
	Allowance for Fire Protection work as required for New Structural Seismic Work	26,000	sqft	$ 1.28	$ 33,306.00	$ 0.82	$ 21,294.00	$ 0.13	$ 3,276.00	$ 2.23	$ 57,876.00	
Totals	**D40- Fire Protection Systems**										**$ 57,876.00**	**$ 2.23**
D50- Electrical Systems												
D5020- Lighting and Branch Wiring												
	Allowance for Electrical Power and Lighting work as required for New Structural Seismic Work	26,000	sqft	$ 6.14	$ 159,705.00	$ 3.31	$ 85,995.00	$ 0.57	$ 14,742.00	$ 10.02	$ 260,442.00	
Totals	**D50- Electrical Systems**										**$ 260,442.00**	**$ 10.02**
										Total Direct Costs ->	**$ 2,976,204**	**$ 114.47**

City Hall
Life Safety
Tukwila Seismic Improvement Program

Tukwila, WA

Conceptual Cost Estimate

April 22, 2022
Revised June 20, 2022

Prepared for:

Reid Middleton

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Estimate Summary

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Name:	**City Hall - Life Safety**
Second Name:	**Tukwila Seismic Improvement Program**
Location:	**Tukwila, WA**
Design Phase:	**Concept Cost Estimate**
Date of Estimate:	**April 22, 2022**
Date of Revision:	**June 20, 2022**
Month of Cost Basis:	**April, 2022**

Estimate Summary

	Percentage of Previous Subtotal		Amount			
				Subtotal Direct Cost	**$**	**3,053,122**
				Subtotal	$	3,053,122
Scope Contingency	15.0%	$	457,968			
				Subtotal	$	3,511,091
General Conditions	16.0%	$	561,774			
				Subtotal	$	4,072,865
Home Office Overhead	6.0%	$	244,372			
				Subtotal	$	4,317,237
Profit	6.0%	$	259,034			
				Subtotal	$	4,576,271
Escalation - Not Included - See Escalation in Table Below	0.00%	$	-			
				Subtotal	$	4,576,271

TOTAL ESTIMATED CONSTRUCTION COST in April, 2022 Dollars → $ 4,576,271

Escalation Table

Cost Estimate in April, 2022 Dollars from Above -> $ 4,576,271

Escalation to:	Out How Many Years	Rate at 6% per year		Escalation Total:	Mid-point of Construction Allowance:		Escalated Total:	Date:
April, 2023	1 year	6.00%	$	274,576	April, 2023	**$**	**4,850,847**	**April, 2023**
April, 2024	2 years	12.36%	$	565,627	April, 2024	**$**	**5,141,898**	**April, 2024**
April, 2025	3 years	19.10%	$	874,141	April, 2025	**$**	**5,450,412**	**April, 2025**
April, 2026	4 years	26.25%	$	1,201,166	April, 2026	**$**	**5,777,437**	**April, 2026**
April, 2027	5 years	33.82%	$	1,547,812	April, 2027	**$**	**6,124,083**	**April, 2027**

Estimate Assumptions:

This estimate is based on the As Built Markups and narrative information received by 4-8-22 and Revisions by 6-10-22
This estimate is based on the working in an unoccupied building with no phasing or 2nd, 3rd shift work.
This estimate is based on the mechanical, electrical, plumbing and fire protection (M/E/P/FP) systems being modified to accommodate seismic work but M/E/P/FP systems are Not upgraded to latest building codes for these systems.
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Division 0/ Division 1 specifications are presumed to have normal ranges for liquidated damages, construction schedule and terms & conditions. These divisions are typically written after the final estimate. Please contact the cost estimator for a review, if desired.
Please consult the cost estimator for any modifications to this estimate. Unilaterally adding and deleting markups, scope of work, schedule, specifications, plans and bid forms could incorrectly restate the project construction cost.
The construction cost estimate does not include an estimate of owner soft costs such as taxes, A/E fees, owner contingencies and permit fees.
Construction reserve contingency for change orders is not included in the estimate.
Any modifications to the plans via addendums and code review for permits will cause cost increases and are not included in this estimate.
Sole source supply of materials and/ or installers typically results in a 40% to 100% premium on costs over open specifications.
Imposition of tariffs and market instability of resources such as fuel, insurance and labor occurring after estimate date are not included.
Contractors imposing different bidding conditions from plans and specifications on subcontractors are not bidding from the plans and specifications.
Modifications to the proposed construction schedule and modifying the phasing plans after this estimate will affect construction cost and are not included.
The estimate includes a reasonable construction escalation that can be determined based on market conditions for up to the next 6 months.
Since this project has a midpoint of construction further than 6 months, increases in escalation are not included beyond the rate shown in the estimate.

Estimate Detail

PRODIMS
520 Kirkland Way, Suite 301
Kirkland, WA 98033
Phone: 425-828-0500 Fax: 425-828-0700
www.prodims.com

Name: **City Hall - Life Safety**
Second Name: **Tukwila Seismic Improvement Program**
Location: **Tukwila, WA**
Design Phase: **Concept Cost Estimate**
Date of Estimate: **April 22, 2022**
Date of Revision: **June 20, 2022**
Month of Cost Basis: **April, 2022**

Area	sqft
1st Floor	12,000
2nd Floor	14,000
	26,000

WBS	Description	Quantity	U of M	Labor	Labor Total	Material	Material Total	Equipment	Equipment Total	Total $/U of M	Direct Cost	Direct $/SQFT
A- Substructure												
A10- Foundations												
A1010- Standard Foundations												
	Enlarge Column Footings with Concrete, Drilled in Rebar Dowels, Formwork, Excavation and Backfill. Remove Restore Surface Treatment	11	each	$ 1,775.00	$ 19,525.00	$ 725.00	$ 7,975.00	$ 150.00	$ 1,650.00	$ 2,650.00	$ 29,150.00	
	New Column Footing at Shear Wall with Concrete, Rebar Dowels, Formwork, Excavation and Backfill. Remove Restore Concrete Slab	1	each	$ 1,430.00	$ 1,430.00	$ 770.00	$ 770.00	$ 132.00	$ 132.00	$ 2,332.00	$ 2,332.00	
A1020- Special Foundations												
	Compaction Grouting for Ground Improvement for Seismic Mitigation - Work is performed under and outside of building and includes restoration of all building and site elements removed to perform the work.	26,250	sqft	$ 20.48	$ 537,600.00	$ 11.52	$ 302,400.00	$ 1.92	$ 50,400.00	$ 33.92	$ 890,400.00	
Totals	**A10- Foundations**										**$ 921,882.00**	**$ 35.46**
B- Shell												
B10- Superstructure												
B1010- Upper Floor												
	Steel - Braces for Continuous Load Path for Shear Walls Above	520	lnft	$ 85.00	$ 44,200.00	$ 40.00	$ 20,800.00	$ 7.50	$ 3,900.00	$ 132.50	$ 68,900.00	
	Upgrade Beam for Shearwall Above	1	each	$ 1,275.00	$ 1,275.00	$ 225.00	$ 225.00	$ 90.00	$ 90.00	$ 1,590.00	$ 1,590.00	
	Shear Walls - New and Retrofit Existing Walls - 2x Wood Framing, Sheathing Each Side	4,550	sqft	$ 7.80	$ 35,490.00	$ 4.20	$ 19,110.00	$ 0.72	$ 3,276.00	$ 12.72	$ 57,876.00	
	Seismic Straps Across Beam Line at Floor Joists at 4' o.c.	74	each	$ 101.40	$ 7,503.60	$ 28.60	$ 2,116.40	$ 7.80	$ 577.20	$ 137.80	$ 10,197.20	
	A35 Clip - Install from 2X Rim to 2X Plate	30	each	$ 24.70	$ 741.00	$ 13.30	$ 399.00	$ 2.28	$ 68.40	$ 40.28	$ 1,208.40	
B1020- Roof												
	Seismic Straps Between Roof Beams	90	lnft	$ 16.38	$ 1,474.20	$ 4.62	$ 415.80	$ 1.26	$ 113.40	$ 22.26	$ 2,003.40	
	Steel Bracing at Windows - X-Braces	146	lnft	$ 68.25	$ 9,964.50	$ 36.75	$ 5,365.50	$ 6.30	$ 919.80	$ 111.30	$ 16,249.80	
	Steel Drag Strut	24	lnft	$ 146.25	$ 3,510.00	$ 78.75	$ 1,890.00	$ 13.50	$ 324.00	$ 238.50	$ 5,724.00	
Totals	**B10- Superstructure**										**$ 163,748.80**	**$ 6.30**

WBS	Description	Quantity	U of M	Labor	Labor Total	Material	Material Total	Equipment	Equipment Total	Total $/U of M	Direct Cost	Direct $/SQFT
B20- Exterior Closure												
B2010- Exterior Walls												
	Remove and Replace Exterior Closure System to Install New Shear Wall System - Allowance	4,200	sqft	$ 13.20	$ 55,440.00	$ 10.80	$ 45,360.00	$ 1.44	$ 6,048.00	$ 25.44	$ 106,848.00	
B2020- Exterior Windows												
	Insulated Glazing "Storefront" Window System Remove and Replace to Install New Steel Bracing	1,168	sqft	$ 35.67	$ 41,662.56	$ 51.33	$ 59,953.44	$ 5.22	$ 6,096.96	$ 92.22	$ 107,712.96	
Totals	**B20- Exterior Closure**										**$ 214,560.96**	**$ 8.25**
B30- Roofing												
B3010- Roof Coverings												
	New Roofing System - Asphalt Composition Roofing System, Underlayment, Batt Insulation, Sheet Metal Flashing and Trim. Demo Existing Roofing System.	14,840	sqft	$ 13.34	$ 197,965.60	$ 9.66	$ 143,354.40	$ 1.38	$ 20,479.20	$ 24.38	$ 361,799.20	
B3020- Roof Openings												
	Install New Skylight System and Curb - 3'-4" x 7'-3" and Remove Existing Skylights	4	each	$ 933.68	$ 3,734.72	$ 1,188.32	$ 4,753.28	$ 127.32	$ 509.28	$ 2,249.32	$ 8,997.28	
Totals	**B30- Roofing**										**$ 370,796.48**	**$ 14.26**
C- Interiors												
C10- Interior Construction												
C1010- Interior Partitions												
	Remove and Reinstall Walls, Doors, Specialties and Casework as required for New Structural Seismic Work	26,000	sqft	$ 3.66	$ 95,160.00	$ 2.34	$ 60,840.00	$ 0.36	$ 9,360.00	$ 6.36	$ 165,360.00	
Totals	**C10- Interior Construction**										**$ 165,360.00**	**$ 6.36**
C30- Interior Finishes												
C3010- Interior Wall Finishes												
	Restore Wall Finishes-Including Painting, Tile, Bases and Specialty Finishes as required for New Structural Seismic Work	26,000	sqft	$ 2.44	$ 63,440.00	$ 1.56	$ 40,560.00	$ 0.24	$ 6,240.00	$ 4.24	$ 110,240.00	
C3020- Interior Floor Finishes												
	Restore Floor Finishes-Including Carpet, Tile, LVT and Specialty Finishes as required for New Structural Seismic Work	26,000	sqft	$ 1.83	$ 47,580.00	$ 1.17	$ 30,420.00	$ 0.18	$ 4,680.00	$ 3.18	$ 82,680.00	
C3030- Interior Ceiling Finishes												
	Restore Ceiling Finishes - Including ACT, GWB and Specialty Finishes as required for New Structural Seismic Work	26,000	sqft	$ 3.05	$ 79,326.00	$ 1.45	$ 37,674.00	$ 0.27	$ 7,020.00	$ 4.77	$ 124,020.00	
Totals	**C30- Interior Finishes**										**$ 316,940.00**	**$ 12.19**

WBS	Description	Quantity	U of M	Labor	Labor Total	Material	Material Total	Equipment	Equipment Total	Total $/U of M	Direct Cost	Direct $/SQFT
D- Services												
D20- Plumbing Systems												
D2010- Plumbing System												
	Allowance For Modifications to Plumbing Systems as required for New Structural Seismic Work	26,000	sqft	$ 1.83	$ 47,580.00	$ 1.17	$ 30,420.00	$ 0.18	$ 4,680.00	$ 3.18	$ 82,680.00	
Totals	**D20- Plumbing Systems**										**$ 82,680.00**	**$ 3.18**
D30- HVAC Systems												
D3020- HVAC System												
	Allowance for HVAC work as required for New Structural Seismic Work	26,000	sqft	$ 10.98	$ 285,480.00	$ 7.02	$ 182,520.00	$ 1.08	$ 28,080.00	$ 19.08	$ 496,080.00	
Totals	**D30- HVAC Systems**										**$ 496,080.00**	**$ 19.08**
D40- Fire Protection Systems												
D4010- Fire Sprinkler System												
	Allowance for Fire Protection work as required for New Structural Seismic Work	26,000	sqft	$ 1.31	$ 34,099.00	$ 0.84	$ 21,801.00	$ 0.13	$ 3,354.00	$ 2.28	$ 59,254.00	
Totals	**D40- Fire Protection Systems**										**$ 59,254.00**	**$ 2.28**
D50- Electrical Systems												
D5020- Lighting and Branch Wiring												
	Allowance for Electrical Power and Lighting work as required for New Structural Seismic Work	26,000	sqft	$ 6.18	$ 160,550.00	$ 3.33	$ 86,450.00	$ 0.57	$ 14,820.00	$ 10.07	$ 261,820.00	
Totals	**D50- Electrical Systems**										**$ 261,820.00**	**$ 10.07**
										Total Direct Costs ->	**$ 3,053,122**	**$ 117.43**

6300 Building
Tukwila Seismic Improvement Program

Tukwila, WA

Conceptual Cost Estimate

April 22, 2022
Revised June 20, 2022

Prepared for:

Reid Middleton

728 134th Street SW
Suite 200
Everett, WA 98204

PD | PRODIMS
PROJECT MANAGEMENT SERVICES

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Estimate Summary

PRODIMS PROJECT MANAGEMENT SERVICES
520 Kirkland Way, Suite 301
Kirkland, WA 98033
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fax: (425) 828-0700
www.prodims.com

Name:	**6300 Building - Life Safety**
Second Name:	**Tukwila Seismic Improvement Program**
Location:	**Tukwila, WA**
Design Phase:	**Concept Cost Estimate**
Date of Estimate:	**April 22, 2022**
Date of Revision:	**June 20, 2022**
Month of Cost Basis:	**April, 2022**

Estimate Summary

	Percentage of Previous Subtotal		Amount			
				Subtotal Direct Cost	**$**	**2,057,461**
				Subtotal	$	2,057,461
Scope Contingency	15.0%	$	308,619			
				Subtotal	$	2,366,080
General Conditions	16.0%	$	378,573			
				Subtotal	$	2,744,653
Home Office Overhead	6.0%	$	164,679			
				Subtotal	$	2,909,332
Profit	6.0%	$	174,560			
				Subtotal	$	3,083,892
Escalation - Not Included - See Escalation in Table Below	0.00%	$	-			
				Subtotal	$	3,083,892

TOTAL ESTIMATED CONSTRUCTION COST in April, 2022 Dollars → $ 3,083,892

Escalation Table

Cost Estimate in April, 2022 Dollars from Above -> $ 3,083,892

Escalation to:	Out How Many Years	Rate at 6% per year		Escalation Total:	Mid-point of Construction Allowance:		Escalated Total:	Date:
April, 2023	1 year	6.00%	$	185,034	April, 2023	**$**	**3,268,926**	**April, 2023**
April, 2024	2 years	12.36%	$	381,169	April, 2024	**$**	**3,465,061**	**April, 2024**
April, 2025	3 years	19.10%	$	589,073	April, 2025	**$**	**3,672,965**	**April, 2025**
April, 2026	4 years	26.25%	$	809,451	April, 2026	**$**	**3,893,343**	**April, 2026**
April, 2027	5 years	33.82%	$	1,043,051	April, 2027	**$**	**4,126,944**	**April, 2027**

Estimate Assumptions:

This estimate is based on the As Built Markups and narrative information received by 4-8-22 and Revisions by 6-10-22
This estimate is based on the working in an unoccupied building with no phasing or 2nd, 3rd shift work.
This estimate is based on the mechanical, electrical, plumbing and fire protection (M/E/P/FP) systems being modified to accommodate seismic work but M/E/P/FP systems are Not upgraded to latest building codes for these systems.
This estimate does not include any Hazardous Material Abatement Costs as it is not defined.
All soft costs are the owner's responsibility to determine and verify. The Soft costs estimate has been excluded from the construction cost estimate.
Escalation is allowed in the above table for 1 to 5 years out to the mid-point of construction as the construction schedule is still to be determined.

Estimate Qualifications:

The estimate is not be relied on solely for proforma development and financial decisions.
Additional Studies of additional systems impacted by the seismic scope of work should be performed before setting construction and project budgets.
Summary sheet markups are cumulative, not additive. Percentages are added to the previous subtotal rather than the direct cost subtotal.
Estimated labor is based on an 8 hour per day shift 5 days a week. Accelerated schedule work of overtime has not been included.
Estimated construction cost is for the entire project. This estimate is not intended to be used for other projects.
Division 0/ Division 1 specifications are presumed to have normal ranges for liquidated damages, construction schedule and terms & conditions. These divisions are typically written after the final estimate. Please contact the cost estimator for a review, if desired.
Please consult the cost estimator for any modifications to this estimate. Unilaterally adding and deleting markups, scope of work, schedule, specifications, plans and bid forms could incorrectly restate the project construction cost.
The construction cost estimate does not include an estimate of owner soft costs such as taxes, A/E fees, owner contingencies and permit fees.
Construction reserve contingency for change orders is not included in the estimate.
Any modifications to the plans via addendums and code review for permits will cause cost increases and are not included in this estimate.
Sole source supply of materials and/ or installers typically results in a 40% to 100% premium on costs over open specifications.
Imposition of tariffs and market instability of resources such as fuel, insurance and labor occurring after estimate date are not included.
Contractors imposing different bidding conditions from plans and specifications on subcontractors are not bidding from the plans and specifications.
Modifications to the proposed construction schedule and modifying the phasing plans after this estimate will affect construction cost and are not included.
The estimate includes a reasonable construction escalation that can be determined based on market conditions for up to the next 6 months.
Since this project has a midpoint of construction further than 6 months, increases in escalation are not included beyond the rate shown in the estimate.

Estimate Detail

PRODIMS

520 Kirkland Way, Suite 301
Kirkland, WA 98033
Phone: 425-828-0500 Fax: 425-828-0700
www.prodims.com

Name: **6300 Building - Life Safety**
Tukwila Seismic Improvement
Second Name: **Program**
Location: **Tukwila, WA**
Design Phase: **Concept Cost Estimate**
Date of Estimate: **April 22, 2022**
Date of Revision: **June 20, 2022**
Month of Cost Basis: **April, 2022**

Area	sqft
1st Floor	16,800
2nd Floor	16,800
	33,600

WBS	Description	Quantity	U of M	Labor	Labor Total	Material	Material Total	Equipment	Equipment Total	Total $/U of M	Direct Cost	Direct $/SQFT
A- Substructure												
A10- Foundations												
A1010- Standard Foundations												
	Spread Footings Foundation System - Concrete, includes excavation, backfilling, erect and strip wood forms, re-steel	16.5	cuyd	$ 377.00	$ 6,230.97	$ 273.00	$ 4,512.08	$ 39.00	$ 644.58	$ 689.00	$ 11,387.64	
	Enlarge Column Footings with Concrete, Drilled in Rebar Dowels, Formwork, Excavation and Backfill. Remove Restore Surface Treatment	5	each	$ 2,556.00	$ 12,780.00	$ 1,044.00	$ 5,220.00	$ 216.00	$ 1,080.00	$ 3,816.00	$ 19,080.00	
	Install Angle with Epoxy Drilled in Bolts with Nut and Washer Anchors - Remove and Replace 2' of Slab on Grade	80	lnft	$ 108.12	$ 8,649.60	$ 50.88	$ 4,070.40	$ 9.54	$ 763.20	$ 168.54	$ 13,483.20	
A1030- Slab on Grade												
	Remove Existing SOG with Sawcutting and Reinstall New SOG, at new Shallow Footings	525	sqft	$ 18.46	$ 9,691.50	$ 7.54	$ 3,958.50	$ 1.56	$ 819.00	$ 27.56	$ 14,469.00	
Totals	**A10- Foundations**										**$ 58,419.84**	**$ 1.74**
B- Shell												
B10- Superstructure												
B1010- Upper Floor												
	Steel - Moment Frame - 2 Columns, 3 Beams per Frame - Replace Steel Columns - Reattach Existing Supported Structure - Temporary Shoring - At the Floor and Roof	4	each	$ 16,704.00	$ 66,816.00	$ 12,096.00	$ 48,384.00	$ 1,728.00	$ 6,912.00	$ 30,528.00	$ 122,112.00	
	Shear Walls - New and Retrofit Existing Walls - 2x Wood Framing, Sheathing Each Side	1,260	sqft	$ 7.80	$ 9,828.00	$ 4.20	$ 5,292.00	$ 0.72	$ 907.20	$ 12.72	$ 16,027.20	
	Cross Tie Across Beam Line at Floor Joists	8	each	$ 101.40	$ 811.20	$ 28.60	$ 228.80	$ 7.80	$ 62.40	$ 137.80	$ 1,102.40	

WBS	Description	Quantity	U of M	Labor	Labor Total	Material	Material Total	Equipment	Equipment Total	Total $/U of M	Direct Cost	Direct $/SQFT
B1020- Roof												
	Shear Walls - New and Retrofit Existing Walls - 2x Wood Framing, Sheathing Each Side	1,260	sqft	$ 7.80	$ 9,828.00	$ 4.20	$ 5,292.00	$ 0.72	$ 907.20	$ 12.72	$ 16,027.20	
	Cross Tie Across Beam Line at Floor Joists	4	each	$ 101.40	$ 405.60	$ 28.60	$ 114.40	$ 7.80	$ 31.20	$ 137.80	$ 551.20	
Totals	**B10- Superstructure**										**$ 155,820.00**	**$ 4.64**
B20- Exterior Closure - No Exterior Closure Work												
B30- Roofing												
B3010- Roof Coverings												
	New Roofing System - Modified Bitumen Roofing System with new R-30 Rigid Insulation, Coverboard, Vapor Retarder, Substrate Board, Walkpad accessories, Metal Flashing and Trim - Demo Existing Roofing System	16,800	sqft	$ 10.45	$ 175,560.00	$ 8.55	$ 143,640.00	$ 1.14	$ 19,152.00	$ 20.14	$ 338,352.00	
B3020- Roof Openings												
	Install New Skylight System and Curb - 3'-4" x 7'-3" and Remove Existing Skylights	4	each	$ 933.68	$ 3,734.72	$ 1,188.32	$ 4,753.28	$ 127.32	$ 509.28	$ 2,249.32	$ 8,997.28	
Totals	**B30- Roofing**										**$ 347,349.28**	**$ 10.34**
C- Interiors												
C10- Interior Construction												
C1010- Interior Partitions												
	Remove and Reinstall Walls, Doors, Specialties and Casework as required for New Structural Seismic Work	33,600	sqft	$ 2.44	$ 81,984.00	$ 1.56	$ 52,416.00	$ 0.24	$ 8,064.00	$ 4.24	$ 142,464.00	
Totals	**C10- Interior Construction**										**$ 142,464.00**	**$ 4.24**
C30- Interior Finishes												
C3010- Interior Wall Finishes												
	Restore Wall Finishes-Including Painting, Tile, Bases and Specialty Finishes as required for New Structural Seismic Work	33,600	sqft	$ 2.29	$ 76,860.00	$ 1.46	$ 49,140.00	$ 0.23	$ 7,560.00	$ 3.98	$ 133,560.00	
C3020- Interior Floor Finishes												
	Restore Floor Finishes-Including Carpet, Tile, LVT and Specialty Finishes as required for New Structural Seismic Work	33,600	sqft	$ 1.68	$ 56,364.00	$ 1.07	$ 36,036.00	$ 0.17	$ 5,544.00	$ 2.92	$ 97,944.00	
C3030- Interior Ceiling Finishes												
	Restore Ceiling Finishes - Including ACT, GWB and Specialty Finishes as required for New Structural Seismic Work	33,600	sqft	$ 2.71	$ 91,123.20	$ 1.29	$ 43,276.80	$ 0.24	$ 8,064.00	$ 4.24	$ 142,464.00	
Totals	**C30- Interior Finishes**										**$ 373,968.00**	**$ 11.13**

WBS	Description	Quantity	U of M	Labor	Labor Total	Material	Material Total	Equipment	Equipment Total	Total $/U of M	Direct Cost	Direct $/SQFT
D- Services												
D20- Plumbing Systems												
D2010- Plumbing System												
	Allowance For Modifications to Plumbing Systems as required for New Structural Seismic Work	33,600	sqft	$ 1.07	$ 35,868.00	$ 0.68	$ 22,932.00	$ 0.11	$ 3,528.00	$ 1.86	$ 62,328.00	
Totals	**D20- Plumbing Systems**										**$ 62,328.00**	**$ 1.86**
D30- HVAC Systems												
D3020- HVAC System												
	Allowance for HVAC work as required for New Structural Seismic Work	33,600	sqft	$ 9.76	$ 327,936.00	$ 6.24	$ 209,664.00	$ 0.96	$ 32,256.00	$ 16.96	$ 569,856.00	
Totals	**D30- HVAC Systems**										**$ 569,856.00**	**$ 16.96**
D40- Fire Protection Systems												
D4010- Fire Sprinkler System												
	Allowance for Fire Protection work as required for New Structural Seismic Work	33,600	sqft	$ 1.07	$ 35,868.00	$ 0.68	$ 22,932.00	$ 0.11	$ 3,528.00	$ 1.86	$ 62,328.00	
Totals	**D40- Fire Protection Systems**										**$ 62,328.00**	**$ 1.86**
D50- Electrical Systems												
D5020- Lighting and Branch Wiring												
	Allowance for Electrical Power and Lighting work as required for New Structural Seismic Work	33,600	sqft	$ 5.20	$ 174,720.00	$ 2.80	$ 94,080.00	$ 0.48	$ 16,128.00	$ 8.48	$ 284,928.00	
Totals	**D50- Electrical Systems**										**$ 284,928.00**	**$ 8.48**
										Total Direct Costs ->	**$ 2,057,461**	**$ 61.23**

Community Center
Collapse Prevention
Tukwila Seismic Improvement Program

Tukwila, WA

Conceptual Cost Estimate

June 20, 2022

Prepared for:

Reid Middleton

728 134th Street SW
Suite 200
Everett, WA 98204

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PROJECT MANAGEMENT SERVICES

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Estimate Summary

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www.prodims.com

Name:	**Community Center - Collapse Prevention**
Second Name:	**Tukwila Seismic Improvement Program**
Location:	**Tukwila, WA**
Design Phase:	**Concept Cost Estimate**
Date of Estimate:	**April 22, 2022**
Date of Revision:	**June 20, 2022**
Month of Cost Basis:	**April, 2022**

Estimate Summary

	Percentage of Previous Subtotal		Amount			
				Subtotal Direct Cost	**$**	**9,069,113**
				Subtotal	$	9,069,113
Scope Contingency	15.0%	$	1,360,367			
				Subtotal	$	10,429,480
General Conditions	16.0%	$	1,668,717			
				Subtotal	$	12,098,197
Home Office Overhead	6.0%	$	725,892			
				Subtotal	$	12,824,089
Profit	6.0%	$	769,445			
				Subtotal	$	13,593,534
Escalation - Not Included - See Escalation in Table Below	0.00%	$	-			
				Subtotal	$	13,593,534

TOTAL ESTIMATED CONSTRUCTION COST in April, 2022 Dollars → $ 13,593,534

Escalation Table

Cost Estimate in April, 2022 Dollars from Above -> $ 13,593,534

Escalation to:	Out How Many Years	Rate at 6% per year		Escalation Total:	Mid-point of Construction Allowance:		Escalated Total:	Date:
April, 2023	1 year	6.00%	$	815,612	April, 2023	**$**	**14,409,146**	**April, 2023**
April, 2024	2 years	12.36%	$	1,680,161	April, 2024	**$**	**15,273,695**	**April, 2024**
April, 2025	3 years	19.10%	$	2,596,583	April, 2025	**$**	**16,190,117**	**April, 2025**
April, 2026	4 years	26.25%	$	3,567,990	April, 2026	**$**	**17,161,524**	**April, 2026**
April, 2027	5 years	33.82%	$	4,597,681	April, 2027	**$**	**18,191,215**	**April, 2027**

Estimate Assumptions:

This estimate is based on the As Built Markups and narrative information received by 6-10-22
This estimate is based on the working in an unoccupied building with no phasing or 2nd, 3rd shift work.
This estimate is based on the mechanical, electrical, plumbing and fire protection (M/E/P/FP) systems being modified to accommodate seismic work but M/E/P/FP systems are Not upgraded to latest building codes for these systems.
This estimate does not include any Hazardous Material Abatement Costs as it is not defined.
All soft costs are the owner's responsibility to determine and verify. The Soft costs estimate has been excluded from the construction cost estimate.
Escalation is allowed in the above table for 1 to 5 years out to the mid-point of construction as the construction schedule is still to be determined.

Estimate Qualifications:

The estimate is not be relied on solely for proforma development and financial decisions.
Additional Studies of additional systems impacted by the seismic scope of work should be performed before setting construction and project budgets.
Summary sheet markups are cumulative, not additive. Percentages are added to the previous subtotal rather than the direct cost subtotal.
Estimated labor is based on an 8 hour per day shift 5 days a week. Accelerated schedule work of overtime has not been included.
Estimated construction cost is for the entire project. This estimate is not intended to be used for other projects.
Division 0/ Division 1 specifications are presumed to have normal ranges for liquidated damages, construction schedule and terms & conditions. These divisions are typically written after the final estimate. Please contact the cost estimator for a review, if desired.
Please consult the cost estimator for any modifications to this estimate. Unilaterally adding and deleting markups, scope of work, schedule, specifications, plans and bid forms could incorrectly restate the project construction cost.
The construction cost estimate does not include an estimate of owner soft costs such as taxes, A/E fees, owner contingencies and permit fees.
Construction reserve contingency for change orders is not included in the estimate.
Any modifications to the plans via addendums and code review for permits will cause cost increases and are not included in this estimate.
Sole source supply of materials and/ or installers typically results in a 40% to 100% premium on costs over open specifications.
Imposition of tariffs and market instability of resources such as fuel, insurance and labor occurring after estimate date are not included.
Contractors imposing different bidding conditions from plans and specifications on subcontractors are not bidding from the plans and specifications.
Modifications to the proposed construction schedule and modifying the phasing plans after this estimate will affect construction cost and are not included.
The estimate includes a reasonable construction escalation that can be determined based on market conditions for up to the next 6 months.
Since this project has a midpoint of construction further than 6 months, increases in escalation are not included beyond the rate shown in the estimate.

Estimate Detail

PRODIMS Project Management Services
520 Kirkland Way, Suite 301
Kirkland, WA 98033
Phone: 425-828-0500 Fax: 425-828-0700
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Name: **Community Center - Collapse Prevention**
Second Name: **Tukwila Seismic Improvement Program**
Location: **Tukwila, WA**
Design Phase: **Concept Cost Estimate**
Date of Estimate: **April 22, 2022**
Date of Revision: **June 20, 2022**
Month of Cost Basis: **April, 2022**

Area	sqft
1st Floor	55,000
	55,000

WBS	Description	Quantity	U of M	Labor	Labor Total	Material	Material Total	Equipment	Equipment Total	Total $/U of M	Direct Cost	Direct $/SQFT
A- Substructure												
A10- Foundations												
A1010- Standard Foundations												
	Spread Footings Foundation System - Concrete, includes excavation, backfilling, erect and strip wood forms, re-steel	43.3	cuyd	$ 377.00	$ 16,322.70	$ 273.00	$ 11,819.89	$ 39.00	$ 1,688.56	$ 689.00	$ 29,831.15	
	Strengthen Column Base Plate Connection at the Rotunda Columns	16	each	$ 3,740.00	$ 59,840.00	$ 1,760.00	$ 28,160.00	$ 330.00	$ 5,280.00	$ 5,830.00	$ 93,280.00	
A1020- Special Foundations												
	Compaction Grouting for Ground Improvement for Seismic Mitigation - Work is performed under and outside of building and includes restoration of all building and site elements removed to perform the work.	86,700	sqft	$ 20.48	$1,775,616.00	$ 11.52	$ 998,784.00	$ 1.92	$ 166,464.00	$ 33.92	$ 2,940,864.00	
A1030- Slab on Grade												
	Remove Existing SOG with Sawcutting and Reinstall New SOG, at new Shallow Footings	667	sqft	$ 18.46	$ 12,312.82	$ 7.54	$ 5,029.18	$ 1.56	$ 1,040.52	$ 27.56	$ 18,382.52	
Totals	**A10- Foundations**										**$ 3,082,357.67**	**$ 56.04**
B- Shell												
B10- Superstructure												
B1020- Roof												
	Shear Walls - Retrofit Existing Walls - 2x Blocking, Sheathing, Hold Downs, Anchor Bolts, Clips to Roof Framing and Anchor Bolts	15,885	sqft	$ 9.96	$ 158,286.08	$ 5.37	$ 85,230.97	$ 0.92	$ 14,611.02	$ 16.25	$ 258,128.07	
	16 GA Metal Strap at Window Headers at Modified Walls	212	each	$ 13.86	$ 2,938.32	$ 8.14	$ 1,725.68	$ 1.32	$ 279.84	$ 23.32	$ 4,943.84	
	Strengthen Beam to Column Connection of Steel Members	2	each	$ 2,400.00	$ 4,800.00	$ 600.00	$ 1,200.00	$ 180.00	$ 360.00	$ 3,180.00	$ 6,360.00	
	Grout Existing 8" CMU Walls	4,950	sqft	$ 2.93	$ 14,478.75	$ 3.58	$ 17,696.25	$ 0.39	$ 1,930.50	$ 6.89	$ 34,105.50	
	8" thick Walls added to the Existing Walls - Shotcrete, Epoxied Drilled in Bars to Face of CMU Wall and Rebar Reinforcing EW/EF. Roughen Face/Apply Concrete Adhesive, Blockout Formwork. Formwork at Wall Edges.	17.0	cuyd	$ 1,147.00	$ 19,496.88	$ 703.00	$ 11,949.70	$ 111.00	$ 1,886.79	$ 1,961.00	$ 33,333.37	
	Add Blocking at top of Concrete Shear Wall to Roof Diaphragm	64	lnft	$ 52.00	$ 3,328.00	$ 28.00	$ 1,792.00	$ 4.80	$ 307.20	$ 84.80	$ 5,427.20	
	Rigid Diaphragm Bracing at Roof Level	12,500	sqft	$ 9.75	$ 121,875.00	$ 5.25	$ 65,625.00	$ 0.90	$ 11,250.00	$ 15.90	$ 198,750.00	

WBS	Description	Quantity	U of M	Labor	Labor Total	Material	Material Total	Equipment	Equipment Total	Total $/U of M	Direct Cost	Direct $/SQFT
	Diaphragm Connection - Wall to Roof Chord Connection	597	lnft	$ 39.00	$ 23,283.00	$ 21.00	$ 12,537.00	$ 3.60	$ 2,149.20	$ 63.60	$ 37,969.20	
	Reinforce Drag Strut	270	lnft	$ 129.60	$ 34,992.00	$ 50.40	$ 13,608.00	$ 10.80	$ 2,916.00	$ 190.80	$ 51,516.00	
	New Drag Strut	117	lnft	$ 146.25	$ 17,111.25	$ 78.75	$ 9,213.75	$ 13.50	$ 1,579.50	$ 238.50	$ 27,904.50	
	Block and add Nailing to the Existing Diaphragm	1,120	sqft	$ 3.06	$ 3,427.20	$ 1.44	$ 1,612.80	$ 0.27	$ 302.40	$ 4.77	$ 5,342.40	
	New 12 Ga Strap - Nailed	225	lnft	$ 13.65	$ 3,071.25	$ 7.35	$ 1,653.75	$ 1.26	$ 283.50	$ 22.26	$ 5,008.50	
	New 12 Ga Strap with Blocking - Nailed	258	lnft	$ 16.25	$ 4,192.50	$ 8.75	$ 2,257.50	$ 1.50	$ 387.00	$ 26.50	$ 6,837.00	
Totals	**B10- Superstructure**										**$ 675,625.58**	**$ 12.28**
B20- Exterior Closure - No Exterior Closure Work												
B30- Roofing												
B3010- Roof Coverings												
	New Roofing System - Metal Roofing System with new R-30 Rigid Insulation, Coverboard, Vapor Retarder and Substrate Board and Flashing and Siding - Demo Existing Roofing System	55,000	sqft	$ 14.04	$ 772,200.00	$ 11.96	$ 657,800.00	$ 1.56	$ 85,800.00	$ 27.56	$ 1,515,800.00	
Totals	**B30- Roofing**										**$ 1,515,800.00**	**$ 27.56**
C- Interiors												
C10- Interior Construction												
C1010- Interior Partitions												
	Remove and Reinstall Walls, Doors, Specialties and Casework as required for New Structural Seismic Work	55,000	sqft	$ 3.63	$ 199,622.50	$ 2.32	$ 127,627.50	$ 0.36	$ 19,635.00	$ 6.31	$ 346,885.00	
Totals	**C10- Interior Construction**										**$ 346,885.00**	**$ 6.31**
C30- Interior Finishes												
C3010- Interior Wall Finishes												
	Restore Wall Finishes-Including Painting, Tile, Bases and Specialty Finishes as required for New Structural Seismic Work	55,000	sqft	$ 2.41	$ 132,522.50	$ 1.54	$ 84,727.50	$ 0.24	$ 13,035.00	$ 4.19	$ 230,285.00	
C3020- Interior Floor Finishes												
	Restore Floor Finishes-Including Carpet, Tile, LVT and Specialty Finishes as required for New Structural Seismic Work	55,000	sqft	$ 4.85	$ 266,722.50	$ 3.10	$ 170,527.50	$ 0.48	$ 26,235.00	$ 8.43	$ 463,485.00	
C3030- Interior Ceiling Finishes												
	Restore Ceiling Finishes - Including ACT, GWB and Specialty Finishes as required for New Structural Seismic Work	55,000	sqft	$ 4.03	$ 221,875.50	$ 1.92	$ 105,374.50	$ 0.36	$ 19,635.00	$ 6.31	$ 346,885.00	
Totals	**C30- Interior Finishes**										**$ 1,040,655.00**	**$ 18.92**

WBS	Description	Quantity	U of M	Labor	Labor Total	Material	Material Total	Equipment	Equipment Total	Total $/U of M	Direct Cost	Direct $/SQFT
D- Services												
D20- Plumbing Systems												
D2010- Plumbing System												
	Allowance For Modifications to Plumbing Systems as required for New Structural Seismic Work	55,000	sqft	$ 1.80	$ 98,972.50	$ 1.15	$ 63,277.50	$ 0.18	$ 9,735.00	$ 3.13	$ 171,985.00	
Totals	**D20- Plumbing Systems**										**$ 171,985.00**	**$ 3.13**
D30- HVAC Systems												
D3020- HVAC System												
	Allowance for HVAC work as required for New Structural Seismic Work	55,000	sqft	$ 12.14	$ 667,645.00	$ 7.76	$ 426,855.00	$ 1.19	$ 65,670.00	$ 21.09	$ 1,160,170.00	
Totals	**D30- HVAC Systems**										**$ 1,160,170.00**	**$ 21.09**
D40- Fire Protection Systems												
D4010- Fire Sprinkler System												
	Allowance for Fire Protection work as required for New Structural Seismic Work	55,000	sqft	$ 1.65	$ 90,585.00	$ 1.05	$ 57,915.00	$ 0.16	$ 8,910.00	$ 2.86	$ 157,410.00	
Totals	**D40- Fire Protection Systems**										**$ 157,410.00**	**$ 2.86**
D50- Electrical Systems												
D5020- Lighting and Branch Wiring												
	Allowance for Electrical Power and Lighting work as required for New Structural Seismic Work	55,000	sqft	$ 10.24	$ 563,062.50	$ 5.51	$ 303,187.50	$ 0.95	$ 51,975.00	$ 16.70	$ 918,225.00	
Totals	**D50- Electrical Systems**										**$ 918,225.00**	**$ 16.70**
										Total Direct Costs ->	**$ 9,069,113**	**$ 164.89**

Community Center
Immediate Occupancy
Tukwila Seismic Improvement Program

Tukwila, WA

Conceptual Cost Estimate

April 22, 2022

Prepared for:

Reid Middleton

728 134th Street SW
Suite 200
Everett, WA 98204

PRODIMS
PROJECT MANAGEMENT SERVICES

520 Kirkland Way, Suite 301
Kirkland, WA 98033
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Estimate Summary

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PROJECT MANAGEMENT SERVICES
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fax: (425) 828-0700
www.prodims.com

Name:	**Community Center - Immediate Occupancy**
Second Name:	**Tukwila Seismic Improvement Program**
Location:	**Tukwila, WA**
Design Phase:	**Concept Cost Estimate**
Date of Estimate:	**April 22, 2022**
Date of Revision:	**June 20, 2022**
Month of Cost Basis:	**April, 2022**

Estimate Summary

	Percentage of Previous Subtotal		Amount		
				Subtotal Direct Cost $	**9,150,166**
				Subtotal $	9,150,166
Scope Contingency	15.0%	$	1,372,525		
				Subtotal $	10,522,691
General Conditions	16.0%	$	1,683,631		
				Subtotal $	12,206,321
Home Office Overhead	6.0%	$	732,379		
				Subtotal $	12,938,701
Profit	6.0%	$	776,322		
				Subtotal $	13,715,023
Escalation - Not Included - See Escalation in Table Below	0.00%	$	-		
				Subtotal $	13,715,023

TOTAL ESTIMATED CONSTRUCTION COST in April, 2022 Dollars → $ 13,715,023

Escalation Table

Cost Estimate in April, 2022 Dollars from Above -> $ 13,715,023

Escalation to:	Out How Many Years	Rate at 6% per year		Escalation Total:	Mid-point of Construction Allowance:		Escalated Total:	Date:
April, 2023	1 year	6.00%	$	822,901	April, 2023	**$**	**14,537,924**	**April, 2023**
April, 2024	2 years	12.36%	$	1,695,177	April, 2024	**$**	**15,410,200**	**April, 2024**
April, 2025	3 years	19.10%	$	2,619,789	April, 2025	**$**	**16,334,812**	**April, 2025**
April, 2026	4 years	26.25%	$	3,599,877	April, 2026	**$**	**17,314,900**	**April, 2026**
April, 2027	5 years	33.82%	$	4,638,771	April, 2027	**$**	**18,353,794**	**April, 2027**

Estimate Assumptions:

This estimate is based on the As Built Markups and narrative information received by 4-8-22 and Revisions by 6-10-22
This estimate is based on the working in an unoccupied building with no phasing or 2nd, 3rd shift work.
This estimate is based on the mechanical, electrical, plumbing and fire protection (M/E/P/FP) systems being modified to accommodate seismic work but M/E/P/FP systems are Not upgraded to latest building codes for these systems.
This estimate does not include any Hazardous Material Abatement Costs as it is not defined.
All soft costs are the owner's responsibility to determine and verify. The Soft costs estimate has been excluded from the construction cost estimate.
Escalation is allowed in the above table for 1 to 5 years out to the mid-point of construction as the construction schedule is still to be determined.

Estimate Qualifications:

The estimate is not be relied on solely for proforma development and financial decisions.
Additional Studies of additional systems impacted by the seismic scope of work should be performed before setting construction and project budgets.
Summary sheet markups are cumulative, not additive. Percentages are added to the previous subtotal rather than the direct cost subtotal.
Estimated labor is based on an 8 hour per day shift 5 days a week. Accelerated schedule work of overtime has not been included.
Estimated construction cost is for the entire project. This estimate is not intended to be used for other projects.
Division 0/ Division 1 specifications are presumed to have normal ranges for liquidated damages, construction schedule and terms & conditions. These divisions are typically written after the final estimate. Please contact the cost estimator for a review, if desired.
Please consult the cost estimator for any modifications to this estimate. Unilaterally adding and deleting markups, scope of work, schedule, specifications, plans and bid forms could incorrectly restate the project construction cost.
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Sole source supply of materials and/ or installers typically results in a 40% to 100% premium on costs over open specifications.
Imposition of tariffs and market instability of resources such as fuel, insurance and labor occurring after estimate date are not included.
Contractors imposing different bidding conditions from plans and specifications on subcontractors are not bidding from the plans and specifications.
Modifications to the proposed construction schedule and modifying the phasing plans after this estimate will affect construction cost and are not included.
The estimate includes a reasonable construction escalation that can be determined based on market conditions for up to the next 6 months.
Since this project has a midpoint of construction further than 6 months, increases in escalation are not included beyond the rate shown in the estimate.

Estimate Detail

PRODIMS Project Management Services

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Name: **Community Center - Immediate Occupancy**
Second Name: **Tukwila Seismic Improvement Program**
Location: **Tukwila, WA**
Design Phase: **Concept Cost Estimate**
Date of Estimate: **April 22, 2022**
Date of Revision: **June 20, 2022**
Month of Cost Basis: **April, 2022**

Area	sqft
1st Floor	55,000
	55,000

WBS	Description	Quantity	U of M	Labor	Labor Total	Material	Material Total	Equipment	Equipment Total	Total $/U of M	Direct Cost	Direct $/SQFT
A- Substructure												
A10- Foundations												
A1010- Standard Foundations												
	Spread Footings Foundation System - Concrete, includes excavation, backfilling, erect and strip wood forms, re-steel	43.3	cuyd	$ 377.00	$ 16,322.70	$ 273.00	$ 11,819.89	$ 39.00	$ 1,688.56	$ 689.00	$ 29,831.15	
	Strengthen Column Base Plate Connection at the Rotunda Columns	16	each	$ 3,740.00	$ 59,840.00	$ 1,760.00	$ 28,160.00	$ 330.00	$ 5,280.00	$ 5,830.00	$ 93,280.00	
A1020- Special Foundations												
	Compaction Grouting for Ground Improvement for Seismic Mitigation - Work is performed under and outside of building and includes restoration of all building and site elements removed to perform the work.	86,700	sqft	$ 20.48	$1,775,616.00	$ 11.52	$ 998,784.00	$ 1.92	$ 166,464.00	$ 33.92	$ 2,940,864.00	
A1030- Slab on Grade												
	Remove Existing SOG with Sawcutting and Reinstall New SOG, at new Shallow Footings	667	sqft	$ 18.46	$ 12,312.82	$ 7.54	$ 5,029.18	$ 1.56	$ 1,040.52	$ 27.56	$ 18,382.52	
Totals	**A10- Foundations**										**$ 3,082,357.67**	**$ 56.04**
B- Shell												
B10- Superstructure												
B1020- Roof												
	Shear Walls - Retrofit Existing Walls - 2x Blocking, Sheathing, Hold Downs, Anchor Bolts, Clips to Roof Framing and Anchor Bolts	18,430	sqft	$ 9.96	$ 183,645.74	$ 5.37	$ 98,886.17	$ 0.92	$ 16,951.91	$ 16.25	$ 299,483.81	
	16 GA Metal Strap at Window Headers at Modified Walls	212	each	$ 13.86	$ 2,938.32	$ 8.14	$ 1,725.68	$ 1.32	$ 279.84	$ 23.32	$ 4,943.84	
	Strengthen Beam to Column Connection of Steel Members	2	each	$ 2,400.00	$ 4,800.00	$ 600.00	$ 1,200.00	$ 180.00	$ 360.00	$ 3,180.00	$ 6,360.00	
	Grout Existing 8" CMU Walls	4,950	sqft	$ 2.93	$ 14,478.75	$ 3.58	$ 17,696.25	$ 0.39	$ 1,930.50	$ 6.89	$ 34,105.50	
	8" thick Walls added to the Existing Walls - Shotcrete, Epoxied Drilled in Bars to Face of CMU Wall and Rebar Reinforcing EW/EF. Roughen Face/Apply Concrete Adhesive, Blockout Formwork. Formwork at Wall Edges.	17.0	cuyd	$ 1,147.00	$ 19,496.88	$ 703.00	$ 11,949.70	$ 111.00	$ 1,886.79	$ 1,961.00	$ 33,333.37	
	Add Blocking at top of Concrete Shear Wall to Roof Diaphragm	64	lnft	$ 52.00	$ 3,328.00	$ 28.00	$ 1,792.00	$ 4.80	$ 307.20	$ 84.80	$ 5,427.20	
	Rigid Diaphragm Bracing at Roof Level	12,500	sqft	$ 9.75	$ 121,875.00	$ 5.25	$ 65,625.00	$ 0.90	$ 11,250.00	$ 15.90	$ 198,750.00	

WBS	Description	Quantity	U of M	Labor	Labor Total	Material	Material Total	Equipment	Equipment Total	Total $/U of M	Direct Cost	Direct $/SQFT
	Diaphragm Connection - Wall to Roof Chord Connection	597	lnft	$ 39.00	$ 23,283.00	$ 21.00	$ 12,537.00	$ 3.60	$ 2,149.20	$ 63.60	$ 37,969.20	
	Reinforce Drag Strut	270	lnft	$ 129.60	$ 34,992.00	$ 50.40	$ 13,608.00	$ 10.80	$ 2,916.00	$ 190.80	$ 51,516.00	
	New Drag Strut	117	lnft	$ 146.25	$ 17,111.25	$ 78.75	$ 9,213.75	$ 13.50	$ 1,579.50	$ 238.50	$ 27,904.50	
	Block and add Nailing to the Existing Diaphragm	1,120	sqft	$ 3.06	$ 3,427.20	$ 1.44	$ 1,612.80	$ 0.27	$ 302.40	$ 4.77	$ 5,342.40	
	New 12 Ga Strap - Nailed	225	lnft	$ 13.65	$ 3,071.25	$ 7.35	$ 1,653.75	$ 1.26	$ 283.50	$ 22.26	$ 5,008.50	
	New 12 Ga Strap with Blocking - Nailed	326	lnft	$ 16.25	$ 5,297.50	$ 8.75	$ 2,852.50	$ 1.50	$ 489.00	$ 26.50	$ 8,639.00	
Totals	**B10- Superstructure**										**$ 718,783.32**	**$ 13.07**
B20- Exterior Closure - No Exterior Closure Work												
B30- Roofing												
B3010- Roof Coverings												
	New Roofing System - Metal Roofing System with new R-30 Rigid Insulation, Coverboard, Vapor Retarder and Substrate Board and Flashing and Siding - Demo Existing Roofing System	55,000	sqft	$ 14.04	$ 772,200.00	$ 11.96	$ 657,800.00	$ 1.56	$ 85,800.00	$ 27.56	$ 1,515,800.00	
Totals	**B30- Roofing**										**$ 1,515,800.00**	**$ 27.56**
C- Interiors												
C10- Interior Construction												
C1010- Interior Partitions												
	Remove and Reinstall Walls, Doors, Specialties and Casework as required for New Structural Seismic Work	55,000	sqft	$ 3.66	$ 201,300.00	$ 2.34	$ 128,700.00	$ 0.36	$ 19,800.00	$ 6.36	$ 349,800.00	
Totals	**C10- Interior Construction**										**$ 349,800.00**	**$ 6.36**
C30- Interior Finishes												
C3010- Interior Wall Finishes												
	Restore Wall Finishes-Including Painting, Tile, Bases and Specialty Finishes as required for New Structural Seismic Work	55,000	sqft	$ 2.44	$ 134,200.00	$ 1.56	$ 85,800.00	$ 0.24	$ 13,200.00	$ 4.24	$ 233,200.00	
C3020- Interior Floor Finishes												
	Restore Floor Finishes-Including Carpet, Tile, LVT and Specialty Finishes as required for New Structural Seismic Work	55,000	sqft	$ 4.88	$ 268,400.00	$ 3.12	$ 171,600.00	$ 0.48	$ 26,400.00	$ 8.48	$ 466,400.00	
C3030- Interior Ceiling Finishes												
	Restore Ceiling Finishes - Including ACT, GWB and Specialty Finishes as required for New Structural Seismic Work	55,000	sqft	$ 4.07	$ 223,740.00	$ 1.93	$ 106,260.00	$ 0.36	$ 19,800.00	$ 6.36	$ 349,800.00	
Totals	**C30- Interior Finishes**										**$ 1,049,400.00**	**$ 19.08**

WBS	Description	Quantity	U of M	Labor	Labor Total	Material	Material Total	Equipment	Equipment Total	Total $/U of M	Direct Cost	Direct $/SQFT
D- Services												
D20- Plumbing Systems												
D2010- Plumbing System												
	Allowance For Modifications to Plumbing Systems as required for New Structural Seismic Work	55,000	sqft	$ 1.83	$ 100,650.00	$ 1.17	$ 64,350.00	$ 0.18	$ 9,900.00	$ 3.18	$ 174,900.00	
Totals	**D20- Plumbing Systems**										**$ 174,900.00**	**$ 3.18**
D30- HVAC Systems												
D3020- HVAC System												
	Allowance for HVAC work as required for New Structural Seismic Work	55,000	sqft	$ 12.20	$ 671,000.00	$ 7.80	$ 429,000.00	$ 1.20	$ 66,000.00	$ 21.20	$ 1,166,000.00	
Totals	**D30- HVAC Systems**										**$ 1,166,000.00**	**$ 21.20**
D40- Fire Protection Systems												
D4010- Fire Sprinkler System												
	Allowance for Fire Protection work as required for New Structural Seismic Work	55,000	sqft	$ 1.68	$ 92,262.50	$ 1.07	$ 58,987.50	$ 0.17	$ 9,075.00	$ 2.92	$ 160,325.00	
Totals	**D40- Fire Protection Systems**										**$ 160,325.00**	**$ 2.92**
D50- Electrical Systems												
D5020- Lighting and Branch Wiring												
	Allowance for Electrical Power and Lighting work as required for New Structural Seismic Work	55,000	sqft	$ 10.40	$ 572,000.00	$ 5.60	$ 308,000.00	$ 0.96	$ 52,800.00	$ 16.96	$ 932,800.00	
Totals	**D50- Electrical Systems**										**$ 932,800.00**	**$ 16.96**
										Total Direct Costs ->	**$ 9,150,166**	**$ 166.37**

Reid Middleton

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File No. 262022.017