707 WILSHIRE BOULEVARD,
SUITE 3650
LOS ANGELES, CA 90017
213.488.4911

October 26, 2023

Anastasia Seims, P.E.
Public Works Director
City of Palos Verdes Estates
340 Palos Verdes Drive West
Verdes Estates, CA 90274

Re: *Seismic Evaluation of Palos Verdes Estates City Hall and Home Owner Association Buildings*
Palos Verdes Estates, CA
Walker Consultants Project 37-009696.02

Dear Ms. Seims:

Walker Consultants is pleased to submit for your review this report for seismic evaluation of Palos Verdes Estates City Hall and Home Owner Association Buildings.

We appreciate the opportunity to be of service to you on this project. If you have any questions or comments, please do not hesitate to call.

Sincerely,

WALKER CONSULTANTS

October 26, 2023

Sohban Khan, PE — Date
Senior Engineer

October 26, 2023

Behnam Arya, PhD, PE — Date
Principal - Forensics & Restoration

BUILDING ENVELOPE

CONSULTING

FORENSIC RESTORATION

PARKING DESIGN

PLANNING

CITY OF PALOS VERDES ESTATES CITY HALL & HOA BUILDINGS SEISMIC EVALUATION

340 Palos Verdes Drive West,
Palos Verdes Estates, CA 90274
October 26, 2023

Prepared for:
City of Palos Verdes Estates

WALKER CONSULTANTS
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Los Angeles, CA 90017
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WC PROJECT No. 37-009696.02 October 26, 2023

EXECUTIVE SUMMARY

This report contains the results of a seismic evaluation of the Palos Verdes Estates City Hall and Homeowner Association (HOA) Buildings located at 340 Palos Verdes Drive in the City of Palos Verdes Estates, California. Walker used a Tier 1 seismic screening and Tier 2 deficiency-based approaches outlined in the American Society of Civil -17 Seismic Evaluation and Retrofit of Existing Buildings, for this evaluation. The City Hall and HOA buildings are occupied by the City Hall and Fire Station personnel and shall remain operational after a major, design-level earthquake. Therefore, the facility is classified as Risk Category IV as defined in the American Society of Civil Engineers Standard (ASCE 7-16), referenced by the current California Building Code.

The Tier 1 screening checks identified several potential deficiencies in the lateral system of the building. Therefore, a Tier 2 evaluation is conducted during which a more detailed engineering analysis was performed to investigate the deficiencies identified in Tier 1 and to propose conceptual repairs to address those deficiencies, if necessary.

The ASCE 41-17 Tier 1 and Tier 2 seismic evaluation approach identified several structural deficiencies in the City Hall and HOA buildings as summarized below:

<u>City Hall Building</u>

- Insufficient shear capacity of plywood shear walls in resisting ASCE 41-17 partially distributed lateral loads in the East-West direction at the second-floor level.
- Insufficient shear capacity of masonry walls in resisting ASCE 41-17 specified lateral loads in the North-South direction at the first and second-floor levels.
- Insufficient shear capacity of plywood shear walls in resisting ASCE 41-17 partially distributed lateral loads in the North-South direction at the second-floor level.
- Inadequate strength of connection between masonry shear walls and diaphragms and between masonry shear walls and foundations to transfer of lateral forces.
- Vertical elements in seismic-force-resisting system are not continues to the foundation. Vault wall at west end of second floor has an offset with the masonry wall below.
- Diagonal roof sheathing does not have sufficient capacity to transfer ASCE 41-17 specified seismic forces to the second-floor seismic force resisting plywood shear walls, tension only braces and vault CMU walls.

<u>HOA Building</u>

- Insufficient shear capacity of plywood shear walls in resisting ASCE 41-17 specified seismic forces at the second floor east-west and north-south directions.
- Original drawings do not specify wood blocking for the roof diaphragm, nor it was accessible during our site visit to investigate the presence of wood blocking. If unblocked, it is required to add blocking or replace the existing sheathing.

We have proposed conceptual seismic upgrades to remediate lateral load resisting system deficiencies as listed in this report. We recommend that the City of Palos Verdes Estates budget a minimum of $2,350,000 for seismic upgrades of City Hall and HOA Buildings for compliance with the requirements of ASCE 41-17 for existing structures.

INTRODUCTION

Walker received a request from Ms. Anastasia Seims, the City of Palos Verdes Estates Public Works Director, to conduct a seismic evaluation of the Palos Verdes Estates City Hall and HOA buildings located at 340 Palos Verdes Drive in the City of Palos Verdes Estates, California. For this evaluation, Walker used a Tier 1 seismic screening and Tier 2 deficiency-based approaches outlined in the American Society of Civil Engineers publication, ASCE 41-17 Standard, as referenced by the current California Existing Building Code. Only structural assessments were
-17
prescribed earthquake loads as requested by the City of Palos Verdes Estates.

SCOPE OF WORK

For this investigation and to meet the project objectives, we performed the following tasks:

1. Reviewed available existing structural drawings and reports provided by the City.
2. Visited the site to observe/compare the building with existing drawings (where possible).
3. Performed ASCE 41-17 Tier 1 screening based on appropriate checklist to identify deficiencies.
4. Compiled a list of the deficiencies upon completion of Tier 1 screening.
5. Performed a Tier 2 analysis to determine and mitigate as many deficiencies as possible by calculations.
6. Made recommendations for strengthening (retrofit scheme) of the structure (as necessary) to meet the current seismic code requirements.
7. Provided approximate cost of the recommended retrofit scheme.
8. Prepared this written narrative of the results of evaluation and recommendations for review and further directions.

REFERENCES

In addition to our site investigation and engineering analysis, we reviewed the following documents that were provided to us:

1. Structural Design Calculations of City Hall building, prepared by C.H. Lewis, dated August 15, 1957.
2. Architectural, Structural, Mechanical and Electrical drawings of the City Hall Building by Carrington H. Lewis Architect, undated (likely around 1957 based on calculation sheets provided).
3. Structural Design Calculations of Public Works Facility (HOA) and Parking Structure, prepared by South Bay Engineering Corporation, dated October 1974.
4. Architectural, Structural and Mechanical drawings of Public Works Facilities (HOA building) and parking structure (addition to original City Hall prepared by Friel and Linde dated February 1975).
5. Parking Structure and Public Works Facility (HOA) Precast Concrete drawings, prepared by Western Precast Inc., dated September 1975.
6. City Hall remodeling plans for interior finishes prepared by Greenlaw Design Associates, Inc. dated February 1988.
7. Council Chamber Plans, prepared by Greenlaw Design Associates, Inc. dated 1990.
8. Remodeling plans for Administrative Offices and Council Chambers, prepared by Hosa Design Associates, dated December 15, 2006.

9. Palos Verdes City Hall Seismic Safety Study Report and Findings, prepared by Melvyn Green and Associates Structural Engineers, dated May 2011.
10. Seismic retrofit drawings prepared by IDS Group, dated April 4, 2012.
11. Proposal for Consulting Architectural and Engineering Services for Studying City Hall Facilities, prepared by IDS Group, dated June 2, 2015.
12. ASCE 7-16 Minimum Design Loads and Associated Criteria for Buildings and Other Structures, publication of American Society of Civil Engineers, Resto Virginia, 2016.
13. ASCE 41-17 Seismic Evaluation and Retrofit of Existing Buildings, publication of American Society of Civil Engineers, Resto Virginia, 2017.

BUILDING DESCRIPTION

The Palos Verdes Estates City Hall complex is located at 340 Palos Verdes Drive in the City of Palos Verdes Estates, California. The complex consists of the main City Hall Building and Public Works Facility along the north and a Parking Structure at the south section. The Public Works Facility is now referred to as Homeowner Association (HOA) building. The main City Hall is a two-story structure with a partial basement level built circa 1957. The plan dimensions are approximately 142 feet in the east-west direction by 43 feet in the north-south direction. It houses the City staff, the City Council Chamber as well as Police and Fire Department at the lower floor. The basement level is used as a firing range for the Police Department. There is also a clock tower structure at the south elevation which is also used for hanging Fire Department hoses. The building has a hipped roof with a ridge line running in the eat-west direction covered with S-shaped clay tiles.

The HOA building and the parking structure were added circa 1975 according to the documents provided to us. The HOA building is a two-story structure with plan dimensions of approximately 77 feet in the east-west direction by 40 feet in the north-south direction. It is used for keeping city records and also houses City Parks and Streets staff. The building has a hipped roof with a ridge line running in the eat-west direction covered with S-shaped clay tiles. A 2-inch-wide joint separates the HOA building from the parking structure along the south elevation of the building. The HOA building is connected to the City Hall with a bridge at the second floor. The bridge in enclosed by metal railing and has a hipped roof with clay tiles.

The parking garage is a two-level concrete structure. Walker has already performed a condition assessment and seismic evaluation of the parking structure and issued a report of findings on May 1, 2023. The focus of this report is on seismic evaluation of the City Hall and HOA buildings.

According to the documents provided to us, a seismic retrofit of the City buildings was performed in 1992. However, no plans of the retrofit were available for review. A seismic evaluation of the buildings was performed in 2011 by Melvyn Green and Associates Structural Engineers. A set of seismic retrofit drawings was prepared by IDS Group in 2012 based on the 2011 evaluation. However, according to the City of Palos Verdes Estates Public Works Director, the proposed IDS retrofit was not constructed.

Figure1 shows an aerial view of the parking structure, and Figures 2 through 6 display the floor plans of the City Hall and HOA buildings.

Below is a detailed description of the gravity (i.e., vertical) and lateral load resisting systems:

CITY HALL STRUCTURAL SYSTEM

Gravity System

The City Hall structural system consists of a combination of structural materials on different floors including concrete, concrete masonry, brick masonry, steel, and wood. The hipped roof is supported on nine steel trusses spanning in the north-south direction which are connected to each other at the ridge and at quarter points of the top chords by steel beams. At the east and west ends of the structure, additional steel beams are provided to support the ends slopes. Roof rafters (2 x 8 at 16 inches on center) support the weight of the 1 x 6-inch roof sheathing boards and clay tiles and transfer the load to the roof steel structure. The weight of roof trusses is supported by steel beams at the perimeters which in turn are supported on 3.0-inch diameter pipe columns embedded in perimeter stud walls according to the original structural drawings.

The exterior walls on the second floor consists of light-weight wood framing (stud walls). There are two vaults with plan dimensions of approximately 10 feet by 12 feet on the second level, one at the west end and one in the middle of the floor. The vault walls are made of reinforced concrete masonry.

The second floor which is the floor below the roof consists of a 4.5-inch-thick concrete slab supported on concrete and steel beams. The concrete floor is supported on an interior 10-inch reinforced concrete wall at first floor in the east-west direction and 9-inch-thick exterior brick walls around the perimeters. The second-floor structure in the fire department is partially supported on steel pipe columns.

The first floor consists of a 6-inch-thick concrete slab supported on concrete beams. The concrete floor is supported on an interior 10-inch reinforced concrete wall in the east-west direction and 10-inch-thick exterior reinforced concrete walls around the perimeters of basement. The basement does not extend to the west end of the City Hall. The west end where the fire department is housed has a 5-inch-thick reinforced concrete slab-on-grade. The foundation system consist of shallow continues reinforced concrete footings below the interior and exterior walls and concrete spread footings below pipe columns.

Lateral System

The roof 1 x 6 diagonal roof sheathing acts as a diaphragm and transfers lateral loads to the roof framing system and to the second-floor vertical elements (walls) of lateral resisting system. According to the original structural plans of the building, cross bracing consisting of flat steel bars were provided between pipe columns at 10 locations at the second floor. The 2011 Melvyn Green report noted that the exterior wall panels at the second floor were retrofitted in 1992 by adding new plywood sheathing to each steel braced frame. The east CMU wall of the middle vault at the second floor is connected to the roof steel trusses (Truss #4 in the original City Hall drawings, detail G-S-8) in the north-south direction according to the original structural plans. At the west end, the edge CMU wall of the west vault is also connected to the roof framing in the north-south direction. Therefore, the lateral resisting system of the second floor consists of two CMU vault walls, perimeter plywood shear walls and two ordinary steel braced frames in the north south direction of the building and perimeter plywood shear walls and eight ordinary steel braced frames in the east-west direction of the building.

The second floor 4.5-inch-thick concrete slab acts as a diaphragm and transfers the earthquake loads from the second floor to the first-floor walls. The first-floor lateral resisting elements in the east-west direction consist of an interior 10-inch-thick reinforced concrete wall at the middle of the floor as well as exterior reinforced brick walls. In the north-south direction, the first-floor lateral system consists of an interior 9-inch reinforced brick party wall between offices and the fire department and 9-inch reinforced walls at the east and west ends of the building.

The first floor 6-inch-thick concrete slab acts as a diaphragm and transfers the earthquake loads from the first floor to the basement reinforced concrete walls. In the east-west direction, the basement vertical elements of lateral resisting system consist of an interior 12-inch-thick reinforced concrete wall at the middle of the floor as well as 10-inch-thick reinforced concrete walls at the north and south perimeters. In the north-south direction, the basement lateral system consists of 10-inch reinforced concrete walls at the east and west perimeters of basement. Figures 2 to 4 show the approximate locations of the lateral load resisting walls of the City Hall building.

The foundation system consist of shallow continues reinforced concrete footings below the interior and exterior walls.

HOA STRUCTURAL SYSTEM

Gravity System

concrete, concrete masonry, brick masonry, steel, and wood. The hipped roof structural system consists of ½ inches thick plywood sheathing supported on 2 x 10-inch rafters at 16 inches on center and 2 x 6 ceiling joists at 16 inches on center. The ceiling joists are connected to the roof rafters by 2 x 4 wood posts (i.e., wood hangers) at the one-third points. The roof structure is supported on perimeter stud walls with 2 x 8-inch studs at south exterior face of the building and 2 x 12-inch studs at the north exterior face of the building. For both exterior walls, studs are located at 16 inches on center. Roof rafters are supported on a steel I beam along the roof ridge.

The second floor which is the floor below the roof consists of a 10 inch thick precast hollow-core concrete planks. Hollow core planks are topped with 2.5 inches of reinforced concrete. The concrete slab is supported on steel beams, and first floor interior and exterior brick walls, and exterior concrete walls. Walls are typically 8 inches thick except for two corner sections of brick walls along the north elevation which are 20 inches thick.

The foundation system consist of shallow continues reinforced concrete footings below the interior and exterior walls.

Lateral System

The roof sheathing acts as a diaphragm and transfers the lateral loads (such as earthquakes) to the roof framing elements and the second-floor lateral resisting walls. According to the 2011 Melvyn Green report, exterior wall panels at the second floor were retrofitted in 1992 by adding new ½ inch thick plywood sheathing to one side of the exterior walls with 8d at 2.5 inches on center boundary and edge nailing. There is a rectangular vault at the middle south edge of the second floor. The vault walls are 8-inch-thick reinforced masonry, and the vault ceiling consists of 5-ich thick concrete slab. The vault walls do not seem to be connected to the roof diaphragm according to the original building plans and as such, they do not contribute to the lateral resisting system. The 2011 Melvyn Green report notes that new stud walls were added in 1992 that extend from top of the CMU walls to the roof framing, however, we could not verify this when we were at the site. Therefore, the lateral resisting system at the second floor primarily consists of plywood shear walls in both directions of the building.

The second-floor concrete hollow core planks that acts as a diaphragm transferring loads to concrete and masonry walls at the first floor and subsequently from walls to the foundations. The first-floor walls in the east-west direction consists of 8-inch-thick concrete wall at the precast stair and 8-inch-thick perimeter CMU walls along the north and the south edge. In the north-south direction, the lateral resisting system consist of 8-inch concrete wall at the east exterior and 9-inch brick wall along the west exterior. Figures 5 and 6 show the approximate locations of the lateral load resisting walls of the HOA building.

Figure 1 - Aerial view of the City Hall Buildings (Google Maps)

Figure 2 City Hall, Second Floor Plan

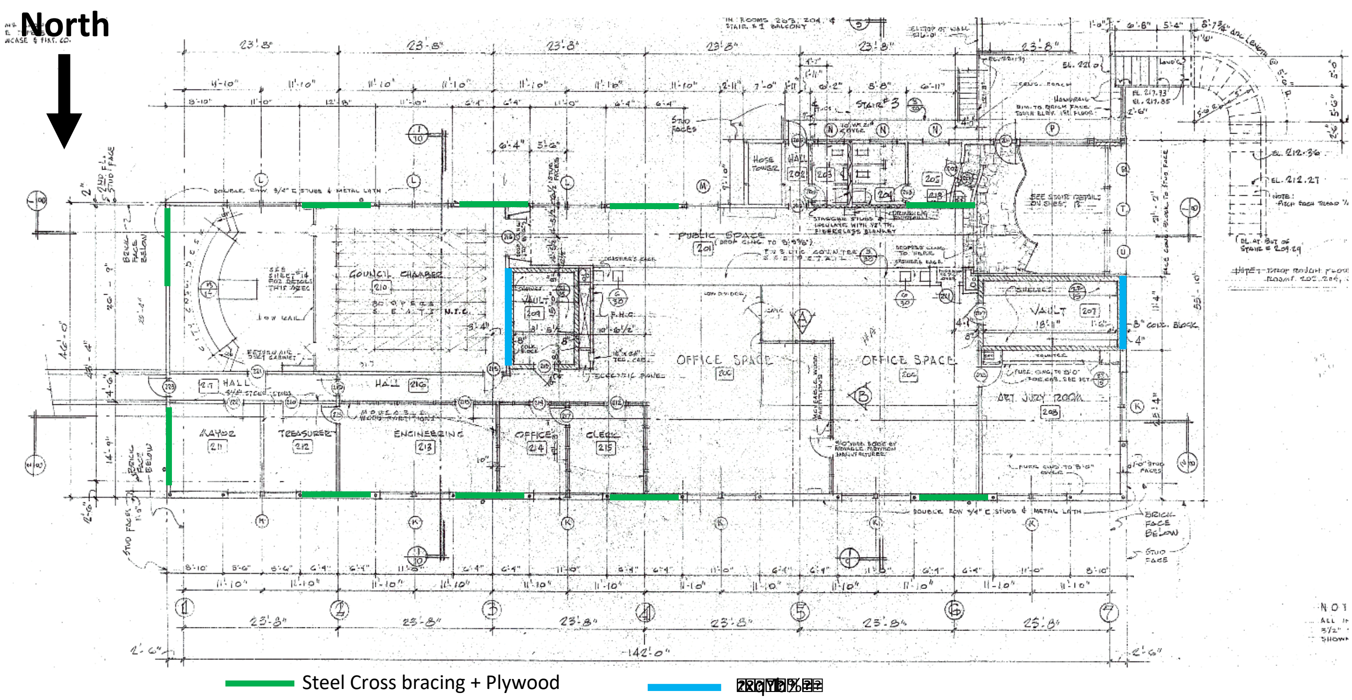


Figure 3 City Hall, First Floor Plan

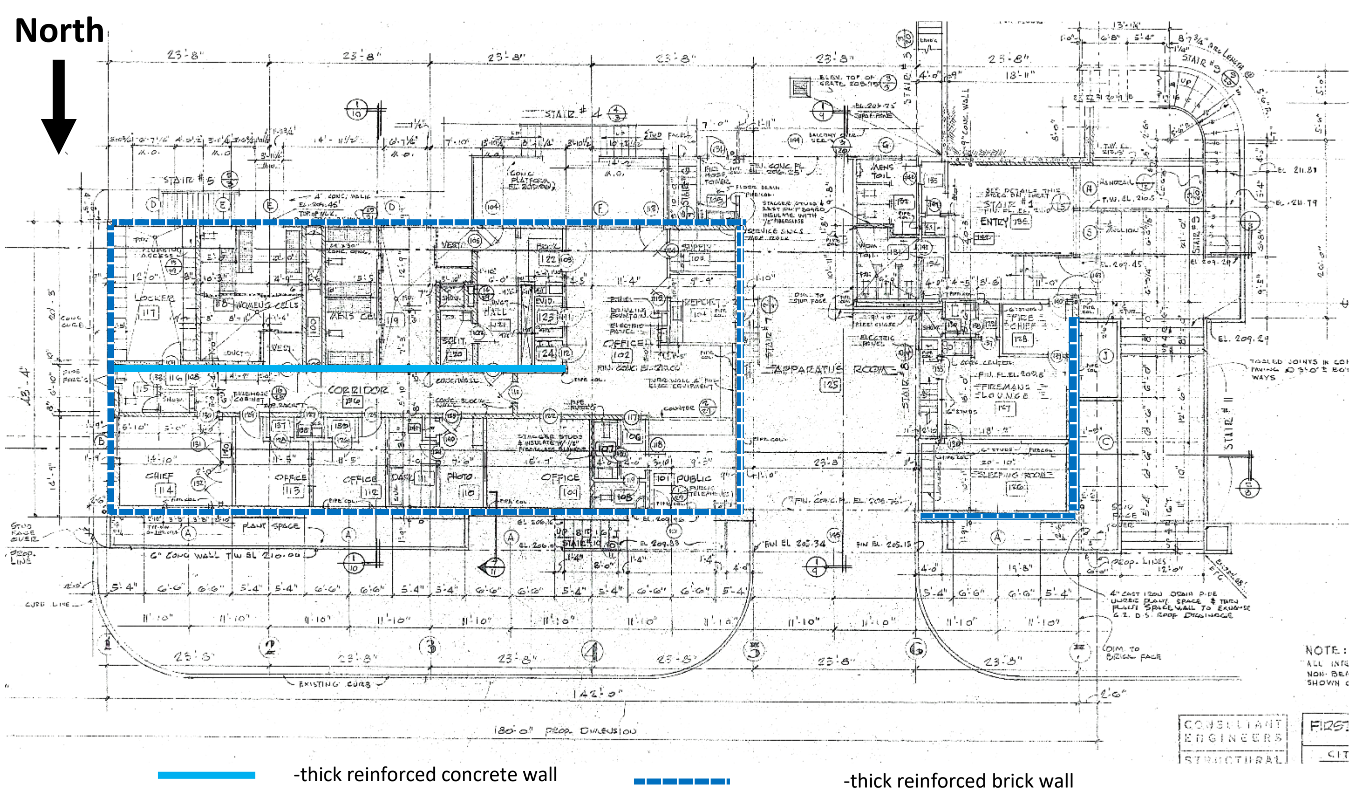
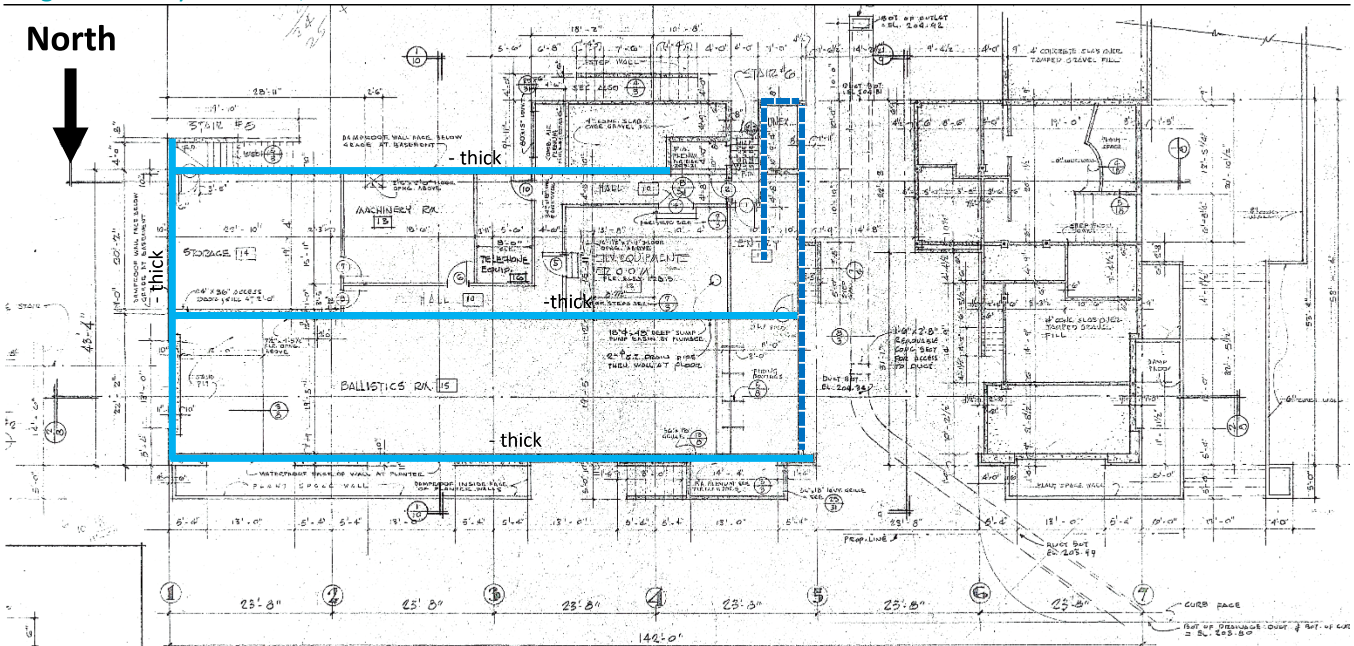


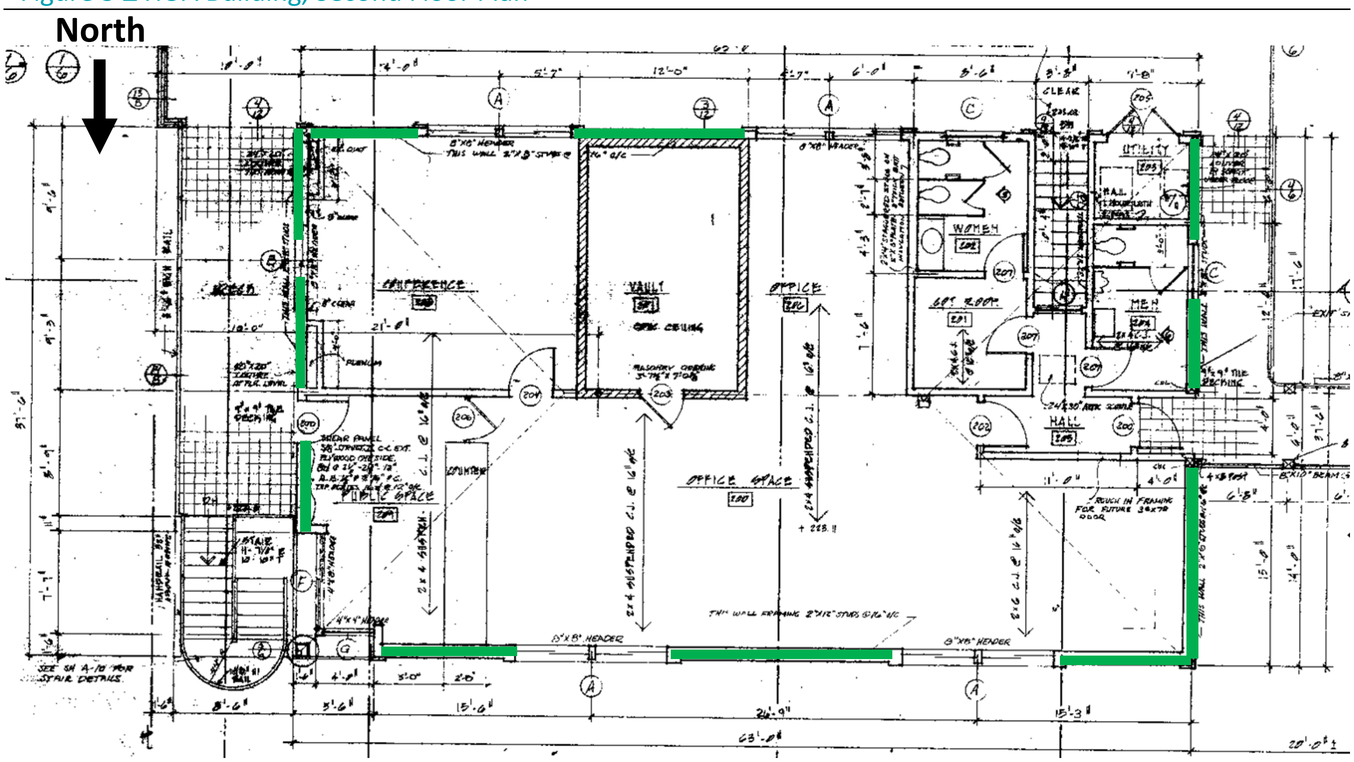
Figure 4 – City Hall Plan, Basement Plan



10 and 12 inch thick Reinforced Concrete Walls

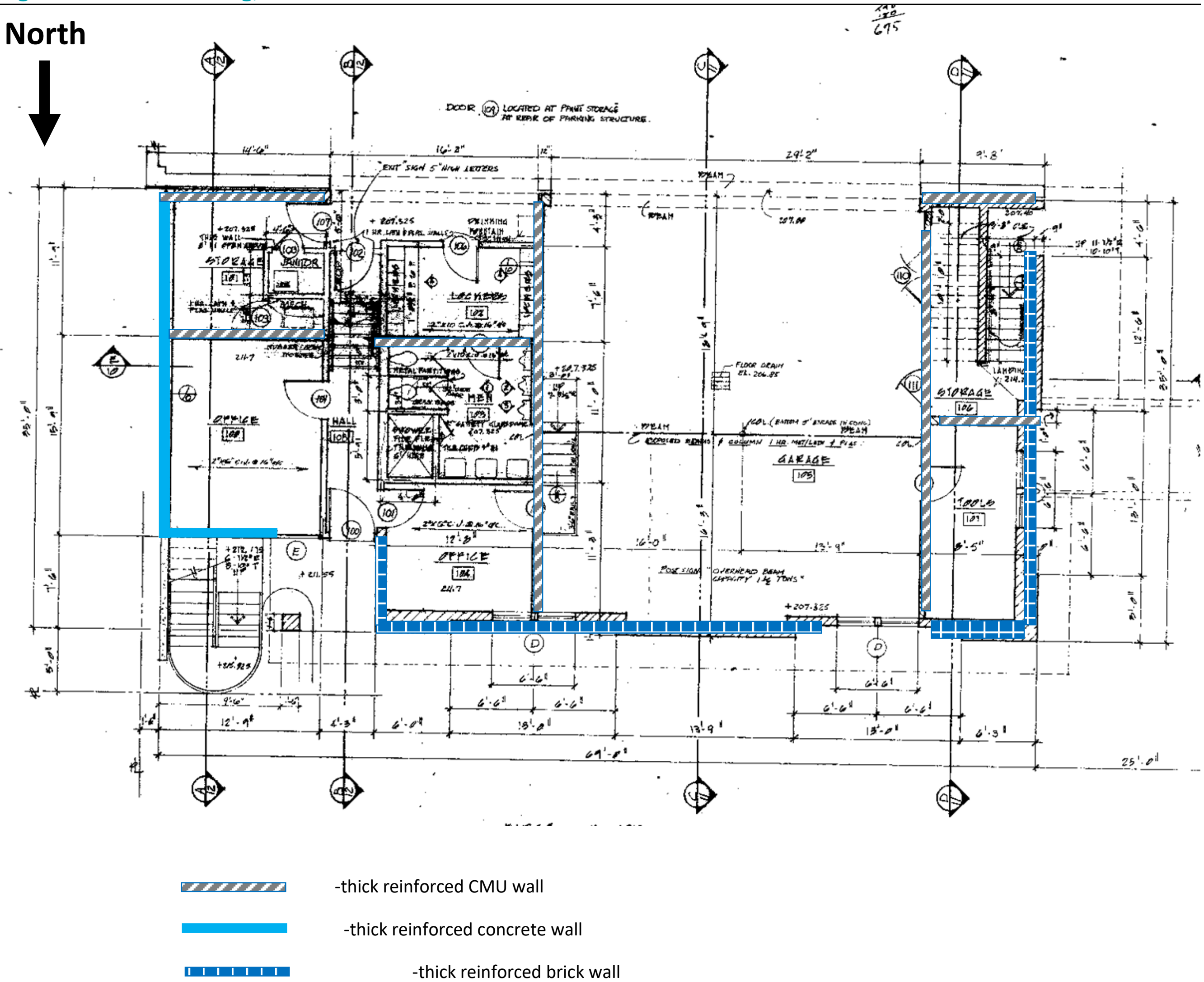
-thick reinforced brick wall

Figure 5 – HOA Building, Second Floor Plan



½ inch plywood - 8d@ 2.5" o.c. edge nailing

Figure 6 ? HOA Building, First Floor Plan



TIER-1 SEISMIC EVALUATION PER ASCE 41-17

As stated earlier, the ASCE 41-17 Tier 1 screening approach was used as the first step of evaluation for this project. Tier 1 evaluation aims at identifying potential deficiencies based on the performance of similar buildings in past earthquakes. As part of completing the checklists, onsite observation was performed on August 22, 2023, to verify the as-built properties of the lateral resisting system, and to observe its condition.

The evaluation is performed through specified checklists given in the ASCE 41-17 standard for various types of buildings. To determine the appropriate checklist for Tier 1 evaluation, the first step is to specify the performance objective for the structure. This is based on the performance level required for the structure after an earthquake and the level of seismicity at the location of the structure. The buildings are occupied by the City and Fire Department personnel and need to remain operational after a major, design-level earthquake. Therefore, the Palos Verdes Estates parking City Hall and HOA buildings are considered an essential facility with occupancy Risk Category IV as defined in the American Society of Civil Engineers Standard (ASCE 7-16).

Per our discussion with the City of Palos Verdes Estates Public Works, Basic Performance Objective for Existing Buildings (BPOE) was selected for this evaluation which lists default (pre-defined) performance objectives in ASCE 41-17 for buildings with various risk categories. The purpose of the BPOE is to introduce a reduced performance level for an existing building compared to a new building. In Table 2-1 of ASCE 41-17 (reproduced herein as Figure 6) two basic performance objectives are considered for Risk Category IV:

1. Immediate Occupancy (IO) Structural Performance for seismicity level BSE-1E
2. Life Safety Structural (LS) Structural Performance for seismicity level BSE-2E

Seismicity Level BSE-1E in ASCE 41-17 is defined as a seismic hazard with a 20% probability of exceedance in 50 years for use with the Basic Performance Objective for Existing Buildings. Seismicity Level BSE-2E is defined as a seismic hazard with a 5% probability of exceedance in 50 years for use with the Basic Performance Objective for Existing Buildings.

The scope of Tier 1 assessment required for the two BPOEs identified above is listed in Table 2-2 of ASCE 41-17 (the table on the right in Figure 7). Based on this table, for occupancy Risk Category IV, the Tier 1 screening shall be conducted for both Immediate Occupancy (with seismic hazard BSE-1E) and Life Safety (with seismic hazard BSE-2E). According to footnote *d* of Table 2-2, the Tier 1 screening checklists for Life Safety Structural Performance shall be based on the Collapse Prevention Structural Performance Level, except that analysis of stresses in lateral resisting system should be modified to consider Life Safety Structural Performance Level. Therefore, wherever required, two sets of analysis and engineering calculations as described above are conducted for Tier 1 evaluation of the structures.

Figure E7 ASCE 41-17 Tables for Basic Performance Objective for Existing Buildings (BPOE) and Scope of Assessment for the Parking Structure

Table 2-1. Basic Performance Objective for Existing Buildings (BPOE)

Risk Category	BSE-1E	BSE-2E
I and II	Life Safety Structural Performance Life Safety Nonstructural Performance (3-C)	Collapse Prevention Structural Performance Hazards Reduced Nonstructural Performance[a] (5-D)
III	Damage Control Structural Performance Position Retention Nonstructural Performance (2-B)	Limited Safety Structural Performance Hazards Reduced Nonstructural Performance[a] (4-D)
IV	Immediate Occupancy Structural Performance Position Retention Nonstructural Performance (1-B)	Life Safety Structural Performance Hazards Reduced Nonstructural Performance[a] (3-D)

[a] Compliance with ASCE 7 provisions for new construction is deemed to comply.

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 Life Safety Nonstructural Performance (3-C)	Collapse Prevention Structural Performance Hazards Reduced Nonstructural Performance[b] (5-D)
III	Not evaluated Position Retention Nonstructural Performance (2-B)	Limited Safety Structural Performance[c] Hazards Reduced Nonstructural Performance[b] (4-D)
IV	Immediate Occupancy Structural Performance Position Retention Nonstructural Performance (1-B)	Life Safety Structural Performance[d] 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.

As part of the evaluation, the following information for the site were also evaluated:

SEISMIC HAZARD AT THE SITE

The City Hall buildings are located at 340 Palos Verdes Drive West at a latitude and longitude of 33.7999 and -118.3915, respectively. No soil reports were available for the site to review; therefore, a default soil type Class D (stiff soil) was assumed due to lack of site-specific field data (per ASCE 7-16 Section 20.1, 11.4.3 and 11.4.4). According to ASCE 41-17 Section 2.5, level of seismicity should be determined based on BSE-2N seismicity level. BSE-2N is Basic Safety Earthquake (with 2% probability of exceedance in 50 years) for use with the Basic Performance Objective Equivalent to New Building Standards, taken as the ground shaking based on the Risk-Targeted Maximum Considered Earthquake (MCE_R) per ASCE 7 at a site. According to U.S. Seismic Design Maps, the location of Palos Verdes Estates City Hall parking structure falls into high seismicity zone (see below).

Seismicity Level

$$S_{DS} = \frac{2}{3} F_a S_S \qquad (2\text{-}4)$$

$$S_{D1} = \frac{2}{3} F_v S_1 \qquad (2\text{-}5)$$

Table 2-4. Level of Seismicity Definitions

Level of Seismicity[a]	S_{DS}	S_{D1}
Very low	<0.167 g	<0.067 g
Low	≥0.167 g <0.33 g	≥0.067 g <0.133 g
Moderate	≥0.33 g <0.50 g	≥0.133 g <0.20 g
High	≥0.50 g	≥0.20 g

[a] The higher level of seismicity defined by S_{DS} or S_{D1} shall govern.

Seismic Parameters are obtained from Seismic Design Map Tool (https://www.seismicmaps.org/)
For BSE-2N, An earthquake with 2% probability of occurring in 50 years
Basic Safety Earthquake-2 for use with the Basic Performance Objective Equivalent to New Building Standards, taken as the ground shaking based on the Risk-Targeted Maximum Considered Earthquake (MCER) per ASCE 7 at a site.

Ss=	2.088	sec
S1=	0.757	sec
Fa=	1.2	
Fv=	1.7	

SDS=	1.670	>	0.5 g	High Seismicity Level
SD1=	0.858	>	0.2 g	High Seismicity Level

Figures 8 to 10 show Seismic parameters for seismicity Level BSE-1E (20% in 50 years), BSE-2E (5% in 50 years) and BSE-2N (2% in 50 years), respectively, for use with the Basic Performance Objective for Existing Buildings.

Figure 8 ⍰ Seismic Parameters Seismic Hazard **BSE-1E** with 20% probability of exceedance in 50 years for Basic Performance Objective for Existing Buildings (BPOE) and Scope of Assessment for the Parking Structure, Based on U.S. Seismic Design Maps (seismicmaps.org)

OSHPD

Palos Verdes Estates City Hall

340 Palos Verdes Dr W, Palos Verdes Estates, CA 90274, USA

Latitude, Longitude: 33.7999009, -118.3916122

Date	10/2/2023, 4:44:29 PM
Design Code Reference Document	ASCE41-17
Custom Probability	0.2
Site Class	D - Default (See Section 11.4.3)

Type	Description	Value
Hazard Level		TL Data
T-Sub-L	Long-period transition period in seconds	8

Type	Description	Value
Hazard Level		Custom
Custom Probability	Decimal probability of exceedance in 50 years for target ground motion.	0.2
S_S	spectral response (0.2 s)	0.586
F_a	site amplification factor (0.2 s)	1.331
S_{XS}	site-modified spectral response (0.2 s)	0.78
S_1	spectral response (1.0 s)	0.195
F_v	site amplification factor (1.0 s)	2.209
S_{X1}	site-modified spectral response (1.0 s)	0.432

Figure 9 ▯ Seismic Parameters Seismic Hazard **BSE-2E** with 5% probability of exceedance in 50 years for Basic Performance Objective for Existing Buildings (BPOE) and Scope of Assessment for the Parking Structure, Based on U.S. Seismic Design Maps (seismicmaps.org)

OSHPD

Palos Verdes Estates City Hall

340 Palos Verdes Dr W, Palos Verdes Estates, CA 90274, USA

Latitude, Longitude: 33.7999009, -118.3916122

Date	10/2/2023, 4:36:02 PM
Design Code Reference Document	ASCE41-17
Custom Probability	0.05
Site Class	D - Default (See Section 11.4.3)

Type	Description	Value
Hazard Level		TL Data
T-Sub-L	Long-period transition period in seconds	8

Type	Description	Value
Hazard Level		Custom
Custom Probability	Decimal probability of exceedance in 50 years for target ground motion.	0.05
S_S	spectral response (0.2 s)	1.356
F_a	site amplification factor (0.2 s)	1.2
S_{XS}	site-modified spectral response (0.2 s)	1.627
S_1	spectral response (1.0 s)	0.471
F_v	site amplification factor (1.0 s)	1.829
S_{X1}	site-modified spectral response (1.0 s)	0.862

Figure 10 ⍰ Seismic Parameters Seismic Hazard **BSE-2N** with 2% probability of exceedance in 50 years to determine seismicity level at the site, Based on U.S. Seismic Design Maps (seismicmaps.org)

OSHPD

Palos Verdes Parking STructure

340 Palos Verdes Dr W, Palos Verdes Estates, CA 90274, USA

Latitude, Longitude: 33.7999009, -118.3916122

Date	1/23/2023, 6:43:46 PM
Design Code Reference Document	ASCE41-17
Custom Probability	0,02
Site Class	D - Default (See Section 11.4.3)

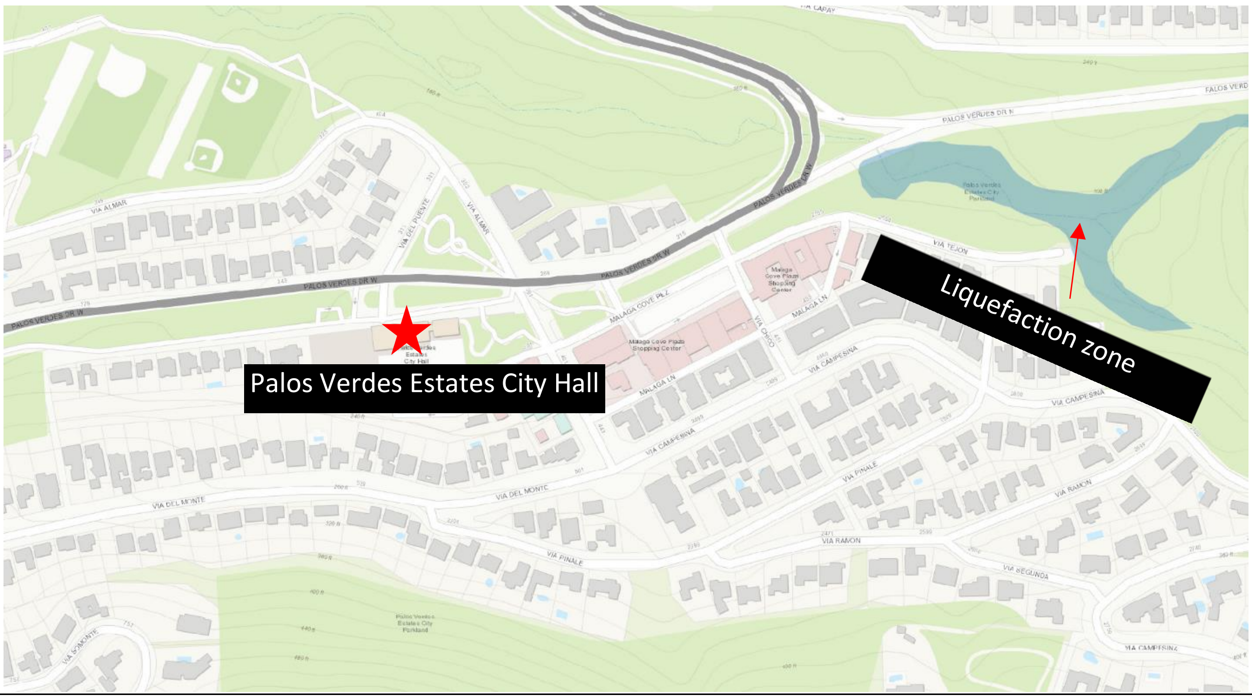
Type	Description	Value
Hazard Level		TL Data
T-Sub-L	Long-period transition period in seconds	8

Type	Description	Value
Hazard Level		Custom
Custom Probability	Decimal probability of exceedance in 50 years for target ground motion.	0,02
S_S	spectral response (0.2 s)	2.088
F_a	site amplification factor (0.2 s)	1.2
S_{XS}	site-modified spectral response (0.2 s)	2.506
S_1	spectral response (1.0 s)	0.757
F_v	site amplification factor (1.0 s)	1.7
S_{X1}	site-modified spectral response (1.0 s)	1.287

LIQUEFACTION POTENTIAL AT THE SITE

According to the Liquefaction zones produced for the Los Angeles County, the site is not located in a liquefaction-prone region (as shown in Figure 11) and hence liquefaction is not expected at the site during an earthquake event.

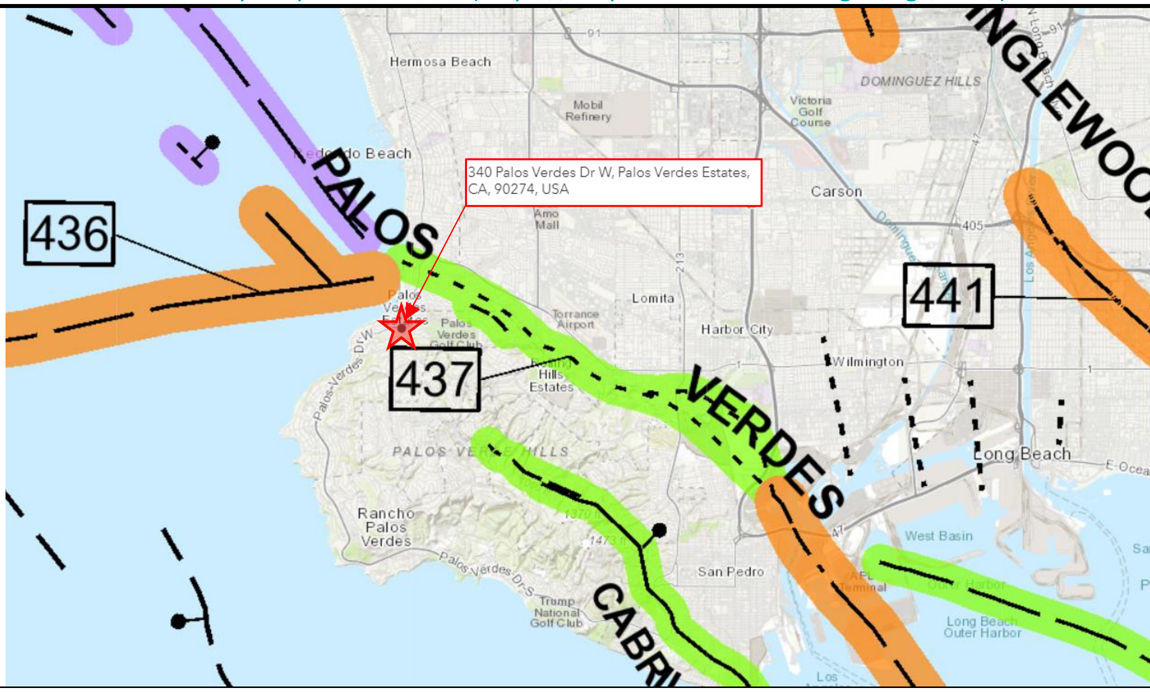
Figure 11 – Liquefaction Zone Map for the vicinity of the site of interest
Source: Liquefaction zones | City of Los Angeles Hub (lacity.org)



FAULT ACTIVITY AT THE SITE

The City Hall and HOA buildings are more than a mile away to the earthquake producing faults based on California Department of Conservation Fault Activity Map of Los Angeles, as shown in Figure 12. Therefore, no surface rupture is expected at the site.

Figure 12 ⍰ Fault Activity Map for the vicinity of the site of interest
Source: Fault Activity Map of California (https://maps.conservation.ca.gov/cgs/fam/)



TIER 1 SCREENING CHECKLISTS

The Tier 1 seismic evaluation study in this report is based on Basic Configuration and Structural Checklists. Nonstructural Screening Checklists were not part of the scope of work for this seismic evaluation phase. Potential seismic deficiencies in the structure are identified using Tier 1 Screening Checklists.

As described earlier, the lateral resisting system components of the City Hall building consists of ordinary steel braced frames, and wood and CMU shear walls at the second floor and concrete and CMU walls at the first and basement floor levels. The City Hall building can be classified as Building Type S2a (Steel Braced Frames), Type W2 (Wood Light Frames), and RM1 (Reinforced Masonry Bearing Walls) with Flexible Wood Diaphragm at the Roof Level. Type C2 (concrete shear walls with Stiff concrete diaphragm), and RM2 (Reinforced Masonry Bearing Walls with Stiff concrete diaphragms) at the first and basement floor levels. For Tier 1 evaluation, we ignored the contribution of steel bracing at the second floor of City Hall compared to plywood shear walls. Therefore, ASCE 41-17 Checklists for structure types W2, RM1, RM2 and C2 were used for the City Hall building evaluation.

The lateral resisting system components of the HOA building consists of structural wood panels at the second floor and concrete and masonry shear walls at the first floor. The HOA building can be classified as Type W2 (Wood Light Frames) with flexible roof diaphragm at the Roof level and Type C2 (concrete shear walls with Stiff concrete diaphragm) and RM2 (Reinforced Masonry Bearing Walls with Stiff concrete diaphragm) at the first floor. Therefore, ASCE 41-17 Checklists for structure types W2, RM2 and C2 were used for the HOA building evaluation.

According to ASCE 41-17, both Immediate Occupancy and Collapse Prevention checklists should be used for City Hall Buildings (assuming occupancy Risk Category IV). While Immediate Occupancy has a more comprehensive Checklist, a smaller seismic hazard (BSE-1E) is considered for performing structural checks compared to Collapse Prevention checklist. To consider Life Safety structural performance for BSE-2E seismic hazard, the quick check procedures of Section 4.4.3 shall be based on M_s (System modification factor) for Life Safety as described in *d* -2 of ASCE 41-17. The results of our investigation are identified with red markings in Figures 13 through 21. Each item in the checklists was investigated by reviewing available drawings, preliminary calculations, or on-site observations. Red circles identify the results of investigation for each item. In the following checklists, iItems identified as a seismic deficiency are marked with rectangles with red fill color.

Figure 13 - Tier 1 Screening - Collapse Prevention Basic Configuration Checklist, City Hall Building

Table 17-2. Collapse Prevention Basic Configuration Checklist

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Low Seismicity			
Building System—General			
C NC N/A U	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.	5.4.1.1	A.2.1.1
C NC N/A U	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.	5.4.1.2	A.2.1.2
C NC N/A U	MEZZANINES: Interior mezzanine levels are braced independently from the main structure or are anchored to the seismic-force-resisting elements of the main structure.	5.4.1.3	A.2.1.3
Building System—Building Configuration			
C NC N/A U	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.	5.4.2.1	A.2.2.2
C NC N/A U	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.	5.4.2.2	A.2.2.3
C NC N/A U	VERTICAL IRREGULARITIES: All vertical elements in the seismic-force-resisting system are continuous to the foundation.	5.4.2.3	A.2.2.4 Vault wall @ West has an offset with the wall below
C NC N/A U	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.	5.4.2.4	A.2.2.5
C NC N/A U	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.	5.4.2.5	A.2.2.6
C NC N/A U	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.	5.4.2.6	A.2.2.7
Moderate Seismicity (Complete the Following Items in Addition to the Items for Low Seismicity)			
Geologic Site Hazards			
C NC N/A U	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.	5.4.3.1	A.6.1.1
C NC N/A U	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.	5.4.3.1	A.6.1.2
C NC N/A U	SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site are not anticipated.	5.4.3.1	A.6.1.3
High Seismicity (Complete the Following Items in Addition to the Items for Moderate Seismicity)			
Foundation Configuration			
C NC N/A U	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 $0.6S_a$.	5.4.3.3	A.6.2.1
C NC N/A U	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.	5.4.3.4	A.6.2.2

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

Figure 14 - Tier 1 Screening - Collapse Prevention Structural Checklist for Building Types W2 - Wood Light Frames, City Hall Second Floor

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

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Low and Moderate Seismicity			
Seismic-Force-Resisting System			
C NC N/A U	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2.	5.5.1.1	A.3.2.1.1
C NC N/A U	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 Diagonal sheathing 700 lb/ft Straight sheathing 100 lb/ft All other conditions 100 lb/ft	5.5.3.1.1	A.3.2.7.1
C NC N/A U	STUCCO (EXTERIOR PLASTER) SHEAR WALLS: Multi-story buildings do not rely on exterior stucco walls as the primary seismic-force-resisting system.	5.5.3.6.1	A.3.2.7.2
C NC N/A U	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.	5.5.3.6.1	A.3.2.7.3
C NC N/A U	NARROW WOOD SHEAR WALLS: Narrow wood shear walls with an aspect ratio greater than 2-to-1 are not used to resist seismic forces.	5.5.3.6.1	A.3.2.7.4
C NC N/A U	WALLS CONNECTED THROUGH FLOORS: Shear walls have an interconnection between stories to transfer overturning and shear forces through the floor.	5.5.3.6.2	A.3.2.7.5
C NC N/A U	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-1.	5.5.3.6.3	A.3.2.7.6
C NC N/A U	CRIPPLE WALLS: Cripple walls below first-floor-level shear walls are braced to the foundation with wood structural panels.	5.5.3.6.4	A.3.2.7.7
C NC N/A U	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.	5.5.3.6.5	A.3.2.7.8
Connections			
C NC N/A U	WOOD POSTS: There is a positive connection of wood posts to the foundation.	5.7.3.3	A.5.3.3
C NC N/A U	WOOD SILLS: All wood sills are bolted to the foundation.	5.7.3.3	A.5.3.4
C NC N/A U	GIRDER–COLUMN CONNECTION: There is a positive connection using plates, connection hardware, or straps between the girder and the column support.	5.7.4.1	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	WOOD SILL BOLTS: Sill bolts are spaced at 6 ft (1.8 m) or less with acceptable edge and end distance provided for wood and concrete.	5.7.3.3	A.5.3.7
Diaphragms			
C NC N/A U	DIAPHRAGM CONTINUITY: The diaphragms are not composed of split-level floors and do not have expansion joints.	5.6.1.1	A.4.1.1
C NC N/A U	ROOF CHORD CONTINUITY: All chord elements are continuous, regardless of changes in roof elevation.	5.6.1.1	A.4.1.3
C NC N/A U	DIAPHRAGM REINFORCEMENT AT OPENINGS: There is reinforcing around all diaphragm openings larger than 50% of the building width in either major plan dimension.	5.6.1.5	A.4.1.8
C NC N/A U	STRAIGHT SHEATHING: All straight-sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered.	5.6.2	A.4.2.1
C NC N/A U	SPANS: All wood diaphragms with spans greater than 24 ft (7.3 m) consist of wood structural panels or diagonal sheathing.	5.6.2	A.4.2.2
C NC N/A U	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 have aspect ratios less than or equal to 4-to-1.	5.6.2	A.4.2.3
C NC N/A U	OTHER DIAPHRAGMS: The diaphragms do not consist of a system other than wood, metal deck, concrete, or horizontal bracing.	5.6.5	A.4.7.1

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

Figure 15 - Tier 1 Screening - Collapse Prevention Structural Checklist for Building Type RM1 - Reinforced Masonry Bearing Walls with flexible diaphragms, City Hall Second Floor

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

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Low and Moderate Seismicity			
Seismic-Force-Resisting System			
C NC N/A U	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2.	5.5.1.1	A.3.2.1.1
C NC N/A U	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).	5.5.3.1.1	A.3.2.4.1
C NC N/A U	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.	5.5.3.1.3	A.3.2.4.2
Stiff Diaphragms			
C NC N/A U	TOPPING SLAB: Precast concrete diaphragm elements are interconnected by a continuous reinforced concrete topping slab.	5.6.4	A.4.5.1
Connections			
C NC N/A U	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.	5.7.1.1	A.5.1.1
C NC N/A U	WOOD LEDGERS: The connection between the wall panels and the diaphragm does not induce cross-grain bending or tension in the wood ledgers.	5.7.1.3	A.5.1.2
C NC N/A U	TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls.	5.7.2	A.5.2.1
C NC N/A U	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.	5.7.2	A.5.2.3
C NC N/A U	FOUNDATION DOWELS: Wall reinforcement is doweled into the foundation.	5.7.3.4	A.5.3.5
C NC N/A U	GIRDER–COLUMN CONNECTION: There is a positive connection using plates, connection hardware, or straps between the girder and the column support.	5.7.4.1	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	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 25% of the wall length.	5.6.1.3	A.4.1.4
C NC N/A U	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.	5.6.1.3	A.4.1.6
Flexible Diaphragms			
C NC N/A U	CROSS TIES: There are continuous cross ties between diaphragm chords.	5.6.1.2	A.4.1.2
C NC N/A U	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 25% of the wall length.	5.6.1.3	A.4.1.4
C NC N/A U	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.	5.6.1.3	A.4.1.6
C NC N/A U	STRAIGHT SHEATHING: All straight-sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered.	5.6.2	A.4.2.1
C NC N/A U	SPANS: All wood diaphragms with spans greater than 24 ft (7.3 m) consist of wood structural panels or diagonal sheathing.	5.6.2	A.4.2.2
C NC N/A U	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.	5.6.2	A.4.2.3
C NC N/A U	OTHER DIAPHRAGMS: Diaphragms do not consist of a system other than wood, metal deck, concrete, or horizontal bracing.	5.6.5	A.4.7.1
Connections			
C NC N/A U	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.	5.7.1.2	A.5.1.4

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

While stress check is Compliant for Life Safety, it is non-compliant for Immediate Occupancy

Figure 16 - Tier 1 Screening - Collapse Prevention Structural Checklist for Building Type RM2 - Reinforced Masonry Bearing Walls with Stiff Diaphragms City Hall First Floor and Basement

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

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Low and Moderate Seismicity			
Seismic-Force-Resisting System			
C NC N/A U	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2.	5.5.1.1	A.3.2.1.1
C NC N/A U	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).	5.5.3.1.1	A.3.2.4.1
C NC N/A U	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.	5.5.3.1.3	A.3.2.4.2
Stiff Diaphragms			
C NC N/A U	TOPPING SLAB: Precast concrete diaphragm elements are interconnected by a continuous reinforced concrete topping slab.	5.6.4	A.4.5.1
Connections			
C NC N/A U	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.	5.7.1.1	A.5.1.1
C NC N/A U	WOOD LEDGERS: The connection between the wall panels and the diaphragm does not induce cross-grain bending or tension in the wood ledgers.	5.7.1.3	A.5.1.2
C NC N/A U	TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls.	5.7.2	A.5.2.1
C NC N/A U	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.	5.7.2	A.5.2.3
C NC N/A U	FOUNDATION DOWELS: Wall reinforcement is doweled into the foundation.	5.7.3.4	A.5.3.5
C NC N/A U	GIRDER–COLUMN CONNECTION: There is a positive connection using plates, connection hardware, or straps between the girder and the column support.	5.7.4.1	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	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 25% of the wall length.	5.6.1.3	A.4.1.4
C NC N/A U	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.	5.6.1.3	A.4.1.6
Flexible Diaphragms			
C NC N/A U	CROSS TIES: There are continuous cross ties between diaphragm chords.	5.6.1.2	A.4.1.2
C NC N/A U	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 25% of the wall length.	5.6.1.3	A.4.1.4
C NC N/A U	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.	5.6.1.3	A.4.1.6
C NC N/A U	STRAIGHT SHEATHING: All straight-sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered.	5.6.2	A.4.2.1
C NC N/A U	SPANS: All wood diaphragms with spans greater than 24 ft (7.3 m) consist of wood structural panels or diagonal sheathing.	5.6.2	A.4.2.2
C NC N/A U	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.	5.6.2	A.4.2.3
C NC N/A U	OTHER DIAPHRAGMS: Diaphragms do not consist of a system other than wood, metal deck, concrete, or horizontal bracing.	5.6.5	A.4.7.1
Connections			
C NC N/A U	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.	5.7.1.2	A.5.1.4

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

While stress check is Compliant for Life Safety, it is non-compliant for Immediate Occupancy

While shear walls are connected to diaphragms, strength of connection in transferring lateral loads is potentially insufficient

Figure 17 - Tier 1 Screening - Collapse Prevention Structural Checklist for Building Types C2 concrete shear walls with stiff concrete diaphragms City Hall First Floor and Basement

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

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Low and Moderate Seismicity			
Seismic-Force-Resisting System			
C NC N/A U	COMPLETE FRAMES: Steel or concrete frames classified as secondary components form a complete vertical-load-carrying system.	5.5.2.5.1	A.3.1.6.1
C NC N/A U	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2.	5.5.1.1	A.3.2.1.1
C NC N/A U	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 $2\sqrt{f'_c}$.	5.5.3.1.1	A.3.2.2.1
C NC N/A U	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.	5.5.3.1.3	A.3.2.2.2
Connections			
C NC N/A U	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.	5.7.1.1	A.5.1.1
C NC N/A U	TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls.	5.7.2	A.5.2.1
C NC N/A U	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.	5.7.3.4	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	DEFLECTION COMPATIBILITY: Secondary components have the shear capacity to develop the flexural strength of the components.	5.5.2.5.2	A.3.1.6.2
C NC N/A U	FLAT SLABS: Flat slabs or plates not part of the seismic-force-resisting system have continuous bottom steel through the column joints.	5.5.2.5.3	A.3.1.6.3
C NC N/A U	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.	5.5.3.2.1	A.3.2.2.3
Diaphragms (Stiff or Flexible)			
C NC N/A U	DIAPHRAGM CONTINUITY: The diaphragms are not composed of split-level floors and do not have expansion joints.	5.6.1.1	A.4.1.1
C NC N/A U	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 25% of the wall length.	5.6.1.3	A.4.1.4
Flexible Diaphragms			
C NC N/A U	CROSS TIES: There are continuous cross ties between diaphragm chords.	5.6.1.2	A.4.1.2
C NC N/A U	STRAIGHT SHEATHING: All straight-sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered.	5.6.2	A.4.2.1
C NC N/A U	SPANS: All wood diaphragms with spans greater than 24 ft (7.3 m) consist of wood structural panels or diagonal sheathing.	5.6.2	A.4.2.2
C NC N/A U	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.	5.6.2	A.4.2.3
C NC N/A U	OTHER DIAPHRAGMS: Diaphragms do not consist of a system other than wood, metal deck, concrete, or horizontal bracing.	5.6.5	A.4.7.1
Connections			
C NC N/A U	UPLIFT AT PILE CAPS: Pile caps have top reinforcement, and piles are anchored to the pile caps.	5.7.3.5	A.5.3.8

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

While shear walls are connected to diaphragms, strength of connection in transferring lateral loads seems is potentially insufficient

Figure 18 - Tier 1 Screening - Collapse Prevention Basic Configuration Checklist (Reproduced herein from ASCE 41-17, Table 17-2), HOA Building

Table 17-2. Collapse Prevention Basic Configuration Checklist

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Low Seismicity			
Building System—General			
C NC N/A U	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.	5.4.1.1	A.2.1.1
C NC N/A U	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.	5.4.1.2	A.2.1.2
C NC N/A U	MEZZANINES: Interior mezzanine levels are braced independently from the main structure or are anchored to the seismic-force-resisting elements of the main structure.	5.4.1.3	A.2.1.3
Building System—Building Configuration			
C NC N/A U	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.	5.4.2.1	A.2.2.2
C NC N/A U	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.	5.4.2.2	A.2.2.3
C NC N/A U	VERTICAL IRREGULARITIES: All vertical elements in the seismic-force-resisting system are continuous to the foundation.	5.4.2.3	A.2.2.4
C NC N/A U	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.	5.4.2.4	A.2.2.5
C NC N/A U	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.	5.4.2.5	A.2.2.6
C NC N/A U	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.	5.4.2.6	A.2.2.7
Moderate Seismicity (Complete the Following Items in Addition to the Items for Low Seismicity)			
Geologic Site Hazards			
C NC N/A U	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.	5.4.3.1	A.6.1.1
C NC N/A U	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.	5.4.3.1	A.6.1.2
C NC N/A U	SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site are not anticipated.	5.4.3.1	A.6.1.3
High Seismicity (Complete the Following Items in Addition to the Items for Moderate Seismicity)			
Foundation Configuration			
C NC N/A U	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 $0.6S_a$.	5.4.3.3	A.6.2.1
C NC N/A U	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.	5.4.3.4	A.6.2.2

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

Figure 19 - Tier 1 Screening - Collapse Prevention Structural Checklist for Building Types W2 - Wood Light Frames, HOA Second Floor

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

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Low and Moderate Seismicity			
Seismic-Force-Resisting System			
C NC N/A U	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2.	5.5.1.1	A.3.2.1.1
C NC N/A U	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 Diagonal sheathing 700 lb/ft Straight sheathing 100 lb/ft All other conditions 100 lb/ft	5.5.3.1.1	A.3.2.7.1
C NC N/A U	STUCCO (EXTERIOR PLASTER) SHEAR WALLS: Multi-story buildings do not rely on exterior stucco walls as the primary seismic-force-resisting system.	5.5.3.6.1	A.3.2.7.2
C NC N/A U	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.	5.5.3.6.1	A.3.2.7.3
C NC N/A U	NARROW WOOD SHEAR WALLS: Narrow wood shear walls with an aspect ratio greater than 2-to-1 are not used to resist seismic forces.	5.5.3.6.1	A.3.2.7.4
C NC N/A U	WALLS CONNECTED THROUGH FLOORS: Shear walls have an interconnection between stories to transfer overturning and shear forces through the floor.	5.5.3.6.2	A.3.2.7.5
C NC N/A U	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-1.	5.5.3.6.3	A.3.2.7.6
C NC N/A U	CRIPPLE WALLS: Cripple walls below first-floor-level shear walls are braced to the foundation with wood structural panels.	5.5.3.6.4	A.3.2.7.7
C NC N/A U	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.	5.5.3.6.5	A.3.2.7.8
Connections			
C NC N/A U	WOOD POSTS: There is a positive connection of wood posts to the foundation.	5.7.3.3	A.5.3.3
C NC N/A U	WOOD SILLS: All wood sills are bolted to the foundation.	5.7.3.3	A.5.3.4
C NC N/A U	GIRDER–COLUMN CONNECTION: There is a positive connection using plates, connection hardware, or straps between the girder and the column support.	5.7.4.1	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	WOOD SILL BOLTS: Sill bolts are spaced at 6 ft (1.8 m) or less with acceptable edge and end distance provided for wood and concrete.	5.7.3.3	A.5.3.7
Diaphragms			
C NC N/A U	DIAPHRAGM CONTINUITY: The diaphragms are not composed of split-level floors and do not have expansion joints.	5.6.1.1	A.4.1.1
C NC N/A U	ROOF CHORD CONTINUITY: All chord elements are continuous, regardless of changes in roof elevation.	5.6.1.1	A.4.1.3
C NC N/A U	DIAPHRAGM REINFORCEMENT AT OPENINGS: There is reinforcing around all diaphragm openings larger than 50% of the building width in either major plan dimension.	5.6.1.5	A.4.1.8
C NC N/A U	STRAIGHT SHEATHING: All straight-sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered.	5.6.2	A.4.2.1
C NC N/A U	SPANS: All wood diaphragms with spans greater than 24 ft (7.3 m) consist of wood structural panels or diagonal sheathing.	5.6.2	A.4.2.2
C NC N/A U	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 have aspect ratios less than or equal to 4-to-1.	5.6.2	A.4.2.3
C NC N/A U	OTHER DIAPHRAGMS: The diaphragms do not consist of a system other than wood, metal deck, concrete, or horizontal bracing.	5.6.5	A.4.7.1

It is not clear whether roof sheathing is blocked.

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

Figure 20 - Tier 1 Screening - Collapse Prevention Structural Checklist for Building Types RM2 - Reinforced Masonry Bearing Walls with Stiff Diaphragms, HOA First Floor

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

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Low and Moderate Seismicity			
Seismic-Force-Resisting System			
C NC N/A U	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2.	5.5.1.1	A.3.2.1.1
C NC N/A U	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).	5.5.3.1.1	A.3.2.4.1
C NC N/A U	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.	5.5.3.1.3	A.3.2.4.2
Stiff Diaphragms			
C NC N/A U	TOPPING SLAB: Precast concrete diaphragm elements are interconnected by a continuous reinforced concrete topping slab.	5.6.4	A.4.5.1
Connections			
C NC N/A U	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.	5.7.1.1	A.5.1.1
C NC N/A U	WOOD LEDGERS: The connection between the wall panels and the diaphragm does not induce cross-grain bending or tension in the wood ledgers.	5.7.1.3	A.5.1.2
C NC N/A U	TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls.	5.7.2	A.5.2.1
C NC N/A U	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.	5.7.2	A.5.2.3
C NC N/A U	FOUNDATION DOWELS: Wall reinforcement is doweled into the foundation.	5.7.3.4	A.5.3.5
C NC N/A U	GIRDER–COLUMN CONNECTION: There is a positive connection using plates, connection hardware, or straps between the girder and the column support.	5.7.4.1	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	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 25% of the wall length.	5.6.1.3	A.4.1.4
C NC N/A U	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.	5.6.1.3	A.4.1.6
Flexible Diaphragms			
C NC N/A U	CROSS TIES: There are continuous cross ties between diaphragm chords.	5.6.1.2	A.4.1.2
C NC N/A U	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 25% of the wall length.	5.6.1.3	A.4.1.4
C NC N/A U	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.	5.6.1.3	A.4.1.6
C NC N/A U	STRAIGHT SHEATHING: All straight-sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered.	5.6.2	A.4.2.1
C NC N/A U	SPANS: All wood diaphragms with spans greater than 24 ft (7.3 m) consist of wood structural panels or diagonal sheathing.	5.6.2	A.4.2.2
C NC N/A U	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.	5.6.2	A.4.2.3
C NC N/A U	OTHER DIAPHRAGMS: Diaphragms do not consist of a system other than wood, metal deck, concrete, or horizontal bracing.	5.6.5	A.4.7.1
Connections			
C NC N/A U	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.	5.7.1.2	A.5.1.4

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

Figure 21 - Tier 1 Screening - Collapse Prevention Structural Checklist for Building Types C2 concrete shear walls and concrete diaphragms HOA First Floor

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

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Low and Moderate Seismicity			
Seismic-Force-Resisting System			
C NC N/A U	COMPLETE FRAMES: Steel or concrete frames classified as secondary components form a complete vertical-load-carrying system.	5.5.2.5.1	A.3.1.6.1
C NC N/A U	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2.	5.5.1.1	A.3.2.1.1
C NC N/A U	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 $2\sqrt{f'_c}$.	5.5.3.1.1	A.3.2.2.1
C NC N/A U	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.	5.5.3.1.3	A.3.2.2.2
Connections			
C NC N/A U	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.	5.7.1.1	A.5.1.1
C NC N/A U	TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls.	5.7.2	A.5.2.1
C NC N/A U	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.	5.7.3.4	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	DEFLECTION COMPATIBILITY: Secondary components have the shear capacity to develop the flexural strength of the components.	5.5.2.5.2	A.3.1.6.2
C NC N/A U	FLAT SLABS: Flat slabs or plates not part of the seismic-force-resisting system have continuous bottom steel through the column joints.	5.5.2.5.3	A.3.1.6.3
C NC N/A U	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.	5.5.3.2.1	A.3.2.2.3
Diaphragms (Stiff or Flexible)			
C NC N/A U	DIAPHRAGM CONTINUITY: The diaphragms are not composed of split-level floors and do not have expansion joints.	5.6.1.1	A.4.1.1
C NC N/A U	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 25% of the wall length.	5.6.1.3	A.4.1.4
Flexible Diaphragms			
C NC N/A U	CROSS TIES: There are continuous cross ties between diaphragm chords.	5.6.1.2	A.4.1.2
C NC N/A U	STRAIGHT SHEATHING: All straight-sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered.	5.6.2	A.4.2.1
C NC N/A U	SPANS: All wood diaphragms with spans greater than 24 ft (7.3 m) consist of wood structural panels or diagonal sheathing.	5.6.2	A.4.2.2
C NC N/A U	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.	5.6.2	A.4.2.3
C NC N/A U	OTHER DIAPHRAGMS: Diaphragms do not consist of a system other than wood, metal deck, concrete, or horizontal bracing.	5.6.5	A.4.7.1
Connections			
C NC N/A U	UPLIFT AT PILE CAPS: Pile caps have top reinforcement, and piles are anchored to the pile caps.	5.7.3.5	A.5.3.8

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

The results of stress check of seismic force resisting walls using ASCE 41-17 Tier 1 screening quick procedure are shown in Tables 1 and 2 for the City Hall Building and the HOA building, respectively. Tier 1 screening calculations are shown in Appendix A.

Table 1. City Hall Building - Tier 1 Wall Stress Check Results

Floor	Direction	Type of Seismic Force Resisting Walls	Tier 1 Evaluation Results for Immediate Occupancy Performance Objectives	Tier 1 Evaluation Results for Life Safety Performance Objectives
2nd	EW	Plywood	Not Good	Not Good
2nd	NS	Plywood and Reinforced CMU	Not Good (for both)	Plywood OK, CMU Not good
1st	EW	Reinforced Brick and Reinforced Concrete	OK (for both)	OK (for both)
1st	NS	Reinforced Brick	Not Good	Not Good
Basement	EW	Reinforced Concrete	OK	OK
Basement	NS	Reinforced Brick and Reinforced Concrete	OK (for both)	OK (for both)

Table 2. HOA Building - Tier 1 Wall Stress Check Results

Floor	Direction	Type of Seismic Force Resisting Walls	Tier 1 Evaluation Results for Immediate Occupancy Performance Objectives	Tier 1 Evaluation Results for Life Safety Performance Objectives
2nd	EW	Plywood	Not Good	Not Good
2nd	NS	Plywood	Not Good	Not Good
1st	EW	Reinforced Brick and Reinforced Concrete	OK	OK
1st	NS	Reinforced Brick and Reinforced CMU	OK	OK

Our Tier 1 analysis indicated the following potential deficiencies:

City Hall Building

Insufficient shear capacity of plywood shear and masonry walls in resisting ASCE 41-17 specified lateral loads at the second floor east-west and north-south directions.

Insufficient shear capacity of masonry walls in resisting ASCE 41-17 specified lateral loads at the first floor North-South Direction.

Inadequate strength of connection between masonry shear walls and diaphragms and between masonry shear walls and foundations to transfer of lateral forces.

Vertical elements in seismic-force-resisting system are not continues to the foundation. Vault wall at west end of second floor has an offset with the masonry wall below.

Diagonal roof sheathing does not have sufficient capacity to transfer ASCE 41-17 specified seismic forces to the second-floor seismic force resisting walls.

HOA Building

Insufficient shear capacity of plywood shear walls in resisting ASCE 41-17 specified seismic forces at the second floor east-west and north-south directions.

Original drawings do not specify wood blocking for the roof diaphragm, nor it was accessible during our site visit to investigate the presence of wood blocking. If unblocked, it is required to add blocking or replace the existing sheathing.

TIER 2 ANALYSIS

The Tier 1 screening checks identified several potential deficiencies in the lateral system of the building. Therefore, a Tier 2 evaluation is conducted during which a more detailed engineering analysis was performed to investigate the deficiencies identified in Tier 1 and to propose conceptual repairs to address those deficiencies, if necessary. The results of deficiency based seismic evaluation of seismic force resisting system using ASCE 41-17 Tier 2 analysis procedure are shown in Tables 3 and 4 for the City Hall Building and the HOA building, respectively. Table 5, shows the result of our analysis for the Fire Hose Tower and Pedestrian Bridge between HOA Building and City Hall Building. Tier 2 Analysis seismic calculations are shown in Appendix B.

Table 3. City Hall Building - Tier 2 Analysis - Summary Results

Floor	**Direction**	**Type of Seismic Force Resisting System**	**Tier 2 Evaluation Results for Immediate Occupancy Performance Objective**	**Tier 2 Evaluation Results for Life Safety Performance Objective**
2nd	EW	Plywood (resisting 60% of seismic force)	Not Good	OK
2nd	EW	Steel Braced Frames (resisting 25% of seismic force)	OK	OK
2nd	EW	Reinforced CMU (resisting 15% of seismic force)	OK	OK
2nd	NS	Plywood (resisting 14% of seismic force)	Not Good	OK
2nd	NS	Steel Braced Frames (resisting 4% of seismic force)	OK	OK
2nd	NS	Reinforced CMU (resisting 82% of seismic force)	Not Good	Not Good
1st	EW	Reinforced Brick	OK	OK
1st	EW	Reinforced Concrete	Not Good	Not Good
1st	NS	Reinforced Brick	Not Good	Not Good
Basement	EW	Reinforced Concrete	OK	OK
Basement	NS	Reinforced Concrete	OK	OK
Basement	NS	Reinforced Brick	Not Good	Not Good

Table 4. HOA Building - Tier 2 Analysis - Summary Results

Floor	Direction	Type of Seismic Force Resisting Walls	Tier 2 Evaluation Results for Immediate Occupancy Performance Objectives	Tier 2 Evaluation Results for Life Safety Performance Objectives
2nd	EW	Plywood	Not Good	Not Good
2nd	NS	Plywood	Not Good	Not Good
1st	EW	Reinforced Brick/CMU	OK	OK
1st	EW	Reinforced Concrete	OK	OK
1st	NS	Reinforced Brick/CMU	OK	OK
1st	NS	Reinforced Concrete	OK	OK

Table 5. City Hall Building Fire Hose Tower and Pedestrian Bridge between HOA Building and City Hall Building - Tier 2 Analysis - Summary Results

Floor	Direction	Type of Seismic Force Resisting Walls	Tier 2 Evaluation Results for Immediate Occupancy Performance Objective	Tier 2 Evaluation Results for Life Safety Performance Objective
1st	EW	connection at W8 beams	Not Good	OK
1st	NS	connection at W8 beams	Not Good	OK
1st	EW	Bridge support shear anchor connection at CMU wall at HOA Building	Not Good	Not Good
1st	NS	Bridge support shear anchor connection at CMU wall at HOA Building	Not Good	Not Good

SUMMARY OF TIER 2 ANALYSIS

Tier 2 deficiency-based evaluation has indicated the following deficiencies:

<u>City Hall Building</u>

Insufficient capacity of roof diaphragm due to the use of 1x6 Tongue and Groove diagonal sheathing with inadequate diaphragm nail spacing and inadequate nailing pattern at diaphragm boundary zones and diaphragm and edge zones.
Insufficient capacity of top chord of roof trusses near supports.
Insufficient capacity of drag trusses at CMU vaults.
Insufficient capacity of connection between drag trusses and CMU vaults.

Higher than code allowed shear stress and inadequate shear load capacity in plywood shear walls and masonry walls resisting ASCE 41-17 specified lateral loads at the second floor east-west and north-south directions.

Potentially inadequate spacing of sill plate anchor bolts at plywood shear walls resisting ASCE 41-17 specified lateral loads at the second floor east-west and north-south directions.

Higher than code allowed shear stress and inadequate shear load capacity in masonry walls in resisting ASCE 41-17 specified lateral loads at the first floor North-South Direction.

Inadequate strength of connection between masonry shear walls and diaphragms and between masonry shear walls and foundations to transfer lateral seismic forces and maintain integrity of direct load path.

Vertical elements in seismic-force-resisting system are not continues to the foundation. Vault wall at west end of second floor has an offset with the masonry wall below.

Insufficient capacity of Tension Tie-Rod in both orthogonal directions at the Fire Hose Tower at the first-floor level.

Insufficient shear capacity of embed anchors of the pedestrian bridge beam support at the CMU wall of the HOA Building.

<u>HOA Building</u>

Higher than code allowed shear stress and inadequate shear load capacity in plywood shear walls and masonry walls resisting ASCE 41-17 specified lateral loads at the second floor east-west and north-south directions.

Inadequate spacing of sill plate anchor bolts at plywood shear walls resisting ASCE 41-17 specified lateral loads at the second floor east-west and north-south directions.

Higher than code allowed shear stress and inadequate shear load capacity in masonry walls in resisting ASCE 41-17 specified lateral loads at the first floor east-west and north-south directions.

Original drawings do not specify wood blocking for the roof diaphragm, nor it was accessible during our site visit to investigate the presence of wood blocking. If unblocked, it is required to add blocking or replace the existing sheathing.

RECOMMENDATIONS

Figures 22 and 23 show a schematic plan for the recommended seismic upgrades of City Hall and HOA buildings, respectively. The following conceptual upgrades are recommended (numbers in parentheses correspond the work item identified in Figures 22 and 23:

1. Replace diagonal sheathing at the roof of the City Hall Building with straight sheathing running perpendicular to 2 x 8 roof rafters and provide adequate roof diaphragm nailing (2.01).
2. Replace roof sheathing and provide roof diaphragm blocking and adequate nailing to meet or exceed seismic force demand at the roof of HOA building (2.01).
3. Modify steel truss cross section to meet structural load demand at the Top chord of roof steel trusses (2.02).
4. Modify steel truss cross section to meet structural load demand at the Bottom chord of Roof Drag Steel trusses (2.03).
5. Modify connection between Roof Steel Truss #4 and Truss #5 to transfer seismic lateral forces to the CMU vault located between lines 3 and 4 (2.04).
6. Modify connection between Roof Steel Truss #9 to transfer seismic lateral forces to the CMU vault located between lines 6 and 7 (2.04).

7. Provide additional lateral support at Bottom chords to transfer seismic lateral forces to the CMU vaults in the east-west direction (2.05).
8. Provide additional lateral shear stiffness to plywood shear walls to meet or exceed allowable shear stress demand at the City Hall Building and the HOA Building (2.06 and 2.07).
9. Provide adequate anchorage at the base of plywood shear walls to meet or exceed allowable shear demand at the City Hall Building and the HOA Building (2.09).
10. Increase lateral shear stiffness of masonry shear walls to meet or exceed allowable shear demand at the City Hall Building (2.08).
11. Provide additional anchorage at the base and at the diaphragm level of masonry shear walls to meet or exceed allowable shear demand at the City Hall Building and the HOA Building (2.09).
12. Provide adequate lateral bracing at Fire Hose Tower in both directions.
13. Provide additional lateral shear connection supports at the pedestrian bridge on the side of the Homeowner Association Building (2.11).

Figure 22 ? City Hall Seismic Recommended Upgrades

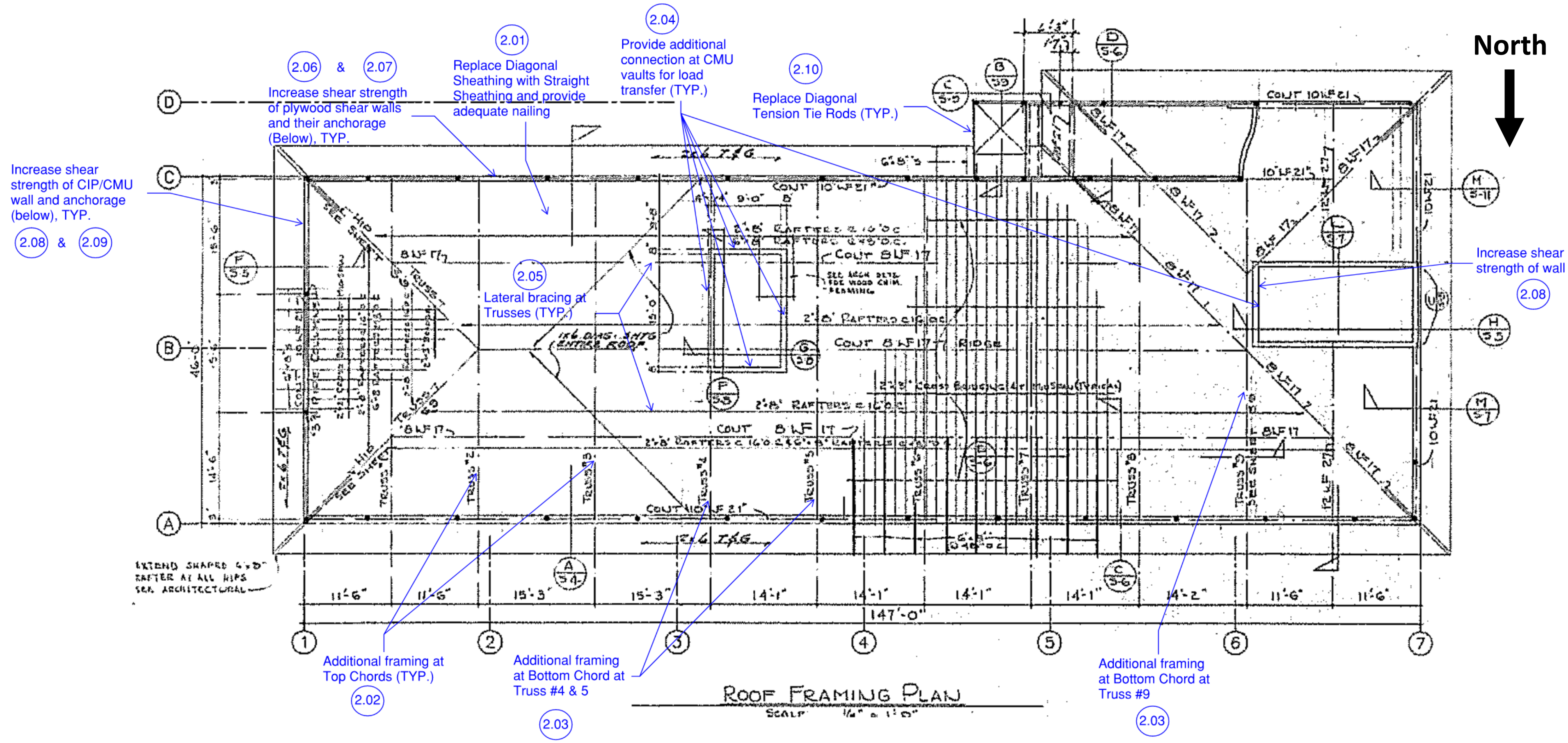
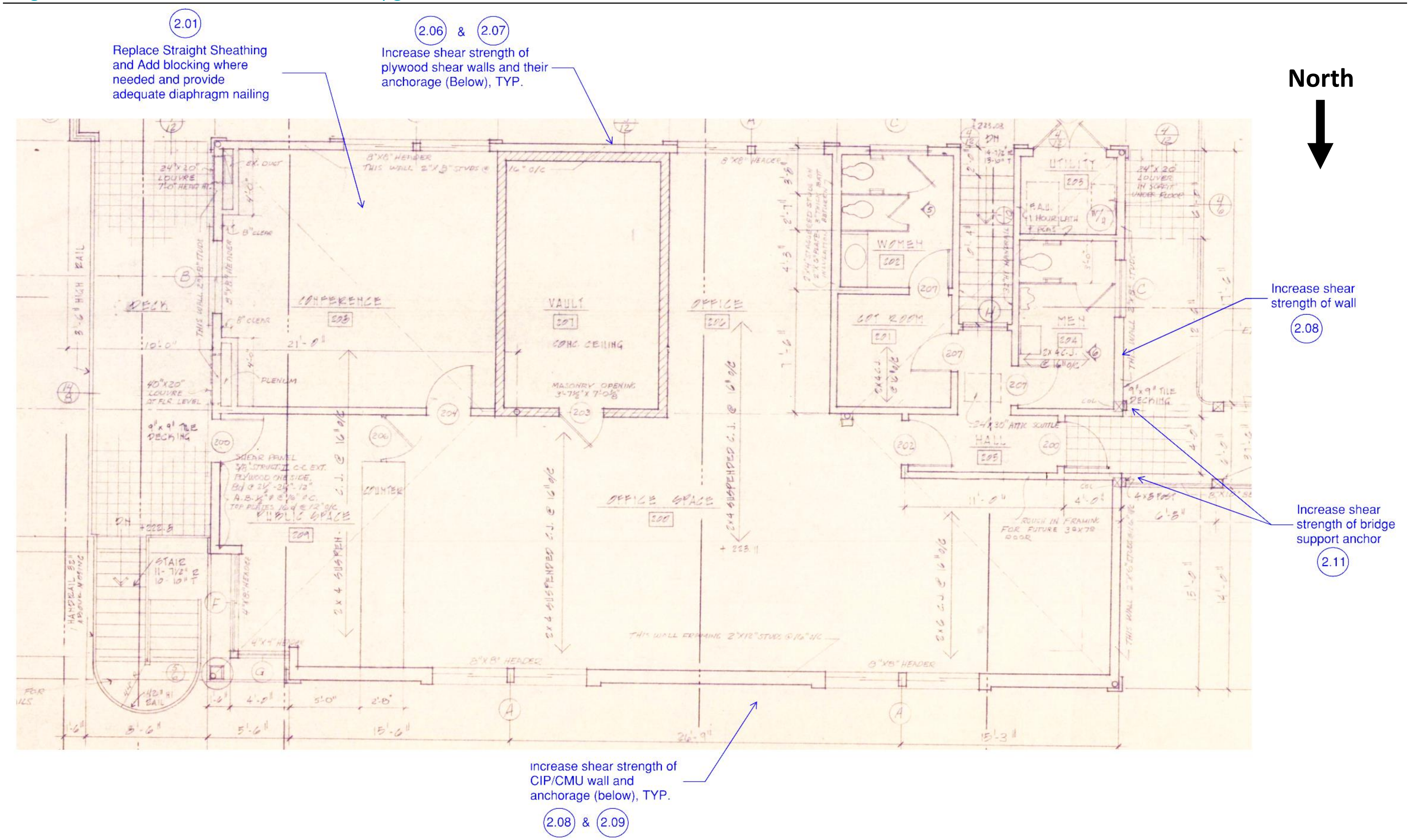


Figure 23 HOA Seismic Recommended Upgrades



OPINION OF PROBABLE COSTS

Table 6 provides our opinion of probable construction costs for the recommended upgrades for seismic rehabilitation as described in this report. The costs were developed using pricing from our database of repair for similar type projects competitively bid in the Los Angeles area.

With the development of repair programs such as in this report, contingency funds must be anticipated and included in any budget for repairs to account for concealed, unknown, or unanticipated conditions. For this type of work, we recommend that a 20% contingency be set aside for potential changes due to unknown conditions. This contingency cost is included in the project costs. The cost estimates are based on 3rd Quarter 2023 dollars.

Table 6 - Opinion of Probable Costs for Conceptual Seismic Upgrades

Work Item	Work Item Description	Estimated Cost
1	**General Conditions**	
1.1	Mobilization & General Conditions	$25,000
2	**Seismic Structural Repairs**	
2.01	Replace Roof diaphragm sheathing and Clay Tiles at the City Hall Building and Roof diaphragm sheathing at the HOA Building	$456,944
2.02	Add additional steel members at Truss top chord members	$72,000
2.03	Add additional steel members at Truss bottom chord members	$30,000
2.04	Additional connections at CMU vaults for lateral force transfer in both east-west and north-south direction	$30,000
2.05	Additional steel bracing members at Roof Trusses at their bottom chords	$36,000
2.06	Strengthening of Plywood shear walls	$199,680
2.07	Strengthening of Plywood shear wall connections	$30,250
2.08	Strengthening of Masonry/Brick Shear walls	$236,400
2.09	Strengthening of Masonry Brick wall connections	$30,250
2.10	Strengthening of Fire Hose Tower lateral bracing	$26,000
2.11	Strengthening of Shear connections at the Bridge	$30,000
	Repair Subtotal	**$1,202,524**
	Prevailing Wage Variance (%40)	$481,009
	Recommended Contingency (20%)	$240,505
	Permit, Insurance Bonding (5%)	$60,126
	GC Profit (10%)	$120,252
	Geotechnical investigation	$50,000
	Engineering services: Prepare construction documents for seismic repairs	$192,404
	Project Total	**$2,346,820**

Photo 1 City Hall North Elevation (BA1-479)

Photo 2 City Hall South Elevation (BA1-479)

Photo 3 City Hall East Elevation and Bridge Between City Hall and HOA Buildings (BA1-524)

Photo 4 City Hall West Elevation (BA1-524)

Photo 5 – HOA Building North Elevation (BA1-541)

Photo 6 – HOA Building South and Partial West Elevation (BA1-575)

Photo 7 HOA Building East Elevation (BA1-548)

Photo 8 HOA Building West Elevation and Bridge Between City Hall and HOA Buildings (BA1-536)

Photo 9 ⍰ The Fire Hose Tower (BA1-496)

APPENDIX-A - TIER 1 CALCULATIONS

ASCE 41-17 Quick Check procedures

City Hall Building Shear Wall Stress Check - Immediate Occupancy (IO)

Seismic Forces - BSE-1E

Base Shear - Equation 4-1 of ASCE 41-17: V = CSaW

C=	1.3		C for Shear wall -1-story to consider max. value for various structural systems
Sxs=	0.78		Seismic Maps
Sx1=	0.432		Seismic Maps
Ct=	0.02		Coefficient for Wood Structures
h_n=	21.5	ft	Height of building
Beta=	0.75		Coefficient for Concrete Shear Wall, Plywood Wood Structures
$T = C_t h_n^{\beta}$ T=	0.20	Sec	Fundamental period Eq. 4-4
$S_a = \frac{S_{X1}}{T}$ Sa	0.78		Response spectral acceleration Shall not exceed Sxs
C.Sa=	1.01		
Total Seismic Weight (W)=	1397	kips	Including roof and exterior walls
Base Shear	1417	kips	

Level	Wx	hx	Wx.hx^k	Wx.hx^k/ΣWx.hx^k	Fx (kips)	Vj (kips)
Roof	458	21.5	9847	0.48	675.6	675.6
2nd	939	11.5	10798.5	0.52	740.9	1416.6
			20646			

	Length of Shear Walls in EW Dir. (ft)				Length of Shear Walls in NS Dir. (ft)			
	Concrete	Brick	CMU	Plywood	Concrete	Brick	CMU	Plywood
2nd				101.3			25.5	30.0
1st	65.5	239.0				120.0		
Basement	284.0				86.7			

2nd Floor:

East West Direction

At the second floor only plywood shear wall resist lateral loads

Plywood wall Length EW= 101.3 ft

Wood wall strength= 1000 lb/ft Table 17-6 ASCE 41-17

Shear Force= 675.6 kips

Force in Plywood Shear Walls - Immediate Occupancy

$$v_j^{avg} = \frac{1}{M_s}\left(\frac{V_j}{A_w}\right)$$ ASCE 41-17 Eq. 4-9

Modification Factor (Ms.)= 1.5 Immediate Occupancy ASCE 41-17 Table 4-8

V/m= 450 kips

orces in Plywood Shear Walls in EW Dir: Vij= 2965 lb/ft > 1000 lb/ft Not Good

North-South Direction

At the second floor plywood shear wall and two CMU vault walls resist lateral loads

Plywood Wall Length NS= 30.0 ft

Plywood Tributary Area= 1150 ft^2

Roof Area= 7347 ft^2

Plywood wall shear force= 105.8 kips

V/m= 71 kips

orces in Plywood Shear Walls in NS Dir: Vij= 1567 lb/ft > 1000 lb/ft Not Good

Force in CMU Shear Walls - Life Safety

CMU Wall Length NS= 25.5 ft

CMU Tributary Area= 6197 ft^2

CMU Shear Force= 380 kips

Area of Shear Walls in NS= 1591 in^2

Stress in CMU Shear Walls in NS Dir:

$$v_j^{avg} = \frac{1}{M_s}\left(\frac{V_j}{A_w}\right)$$ ASCE 41-17 Eq. 4-9

Modification Factor (Ms.)= 1.5 Immediate Occupancy ASCE 41-17 Table 4-8

Vij= 159 > 70 psi Not Good

1st Floor:

East West Direction

At the first floor concrete and brick shear walls resist lateral loads in the east-west direction

Force in Shear Walls - Immediate Occupancy

Story Shear= 1416.6 kips

Stress in Shear Walls in EW Dir:

Concrete Wall Length EW=	65.5	ft	
Brick Wall Length EW=	239.0	ft	
Area of Concrete Walls in EW=	7860	in^2	
Area of Brick Walls in EW=	25811	in^2	

$$v_j^{avg} = \frac{1}{M_s}\left(\frac{V_j}{A_w}\right)$$ ASCE 41-17 Eq. 4-9

Modification Factor (Ms.)=	1.5	Life Safety		ASCE 41-17 Table 4-8			
Vij=	28	psi	<	70	psi	OK	

North South Direction

At the first d brick shear walls resist lateral loads in the north-south direction

Force in Shear Walls - Immediate Occupancy

Story Shear= 1416.6 kips

Stress in Shear Walls in NS Dir:

Brick Wall Length NS=	120.0	ft	
Area of Brick Walls in NS=	12960	in^2	

$$v_j^{avg} = \frac{1}{M_s}\left(\frac{V_j}{A_w}\right)$$ ASCE 41-17 Eq. 4-9

Modification Factor (Ms.)=	1.5	Immediate Occupancy		ASCE 41-17 Table 4-8			
Vij=	73	psi	>	70	psi	Not Good	

Basement:

East West Direction

At the basement concrete walls resist lateral loads in the east-west direction

Force in Shear Walls - Immediate Occupancy

Story Shear= 1416.6 kips

Stress in Shear Walls in EW Dir:

Concrete Walls Length EW=	284.0	ft	
Area of Concrete Walls in EW=	36286	in^2	

$$v_j^{avg} = \frac{1}{M_s}\left(\frac{V_j}{A_w}\right)$$ ASCE 41-17 Eq. 4-9

Modification Factor (Ms.)=	1.5	Immediate Occupancy		ASCE 41-17 Table 4-8			
Vij=	26	psi	<	100	psi	OK	

North-South Direction

At the basement concrete and brick walls resist lateral loads in the north-south direction

Force in Shear Walls - Immediate Occupancy

Story Shear= 1416.6 kips

Stress in Shear Walls in NS Dir:

Brick Walls Length NS=	120.0	ft	
Concrete Walls Length NS=	86.7	ft	
Area of Brick Walls in NS=	12960	in^2	
Area of Concrete Walls in NS=	10399	in^2	

$$v_j^{avg} = \frac{1}{M_s}\left(\frac{V_j}{A_w}\right)$$ ASCE 41-17 Eq. 4-9

Modification Factor (Ms.)=	1.5	Immediate Occupancy		ASCE 41-17 Table 4-8			
Vij=	40	psi	<	70	psi	OK	

City Hall Building Shear Wall Stress Check - Life Safety - LS

Seismic Forces - BSE-2E

Base Shear - Equation 4-1 of ASCE 41-17: V = CSaW

	C=	1.3		C for Shear wall -1-story to consider max. value for various structural systems
	Sxs=	1.627		Seismic Maps
	Sx1=	0.862		Seismic Maps
	Ct=	0.02		Coefficient for Wood Structures
	h_n=	21.5	ft	Height of building
	Beta=	0.75		Coefficient for Concrete Shear Wall, Plywood Wood Structures
$T = C_t h_n^\beta$	T=	0.20	Sec	Fundamental period Eq. 4-4
$S_a = \frac{S_{X1}}{T}$	Sa	1.627		Response spectral acceleration Shall not exceed Sxs
	C.Sa=	2.12		
	Total Seismic Weight (W)=	1397	kips	Including roof and exterior walls
	Base Shear	2955	kips	

Level	Wx	hx	$Wx.hx^k$	$Wx.hx^k/\Sigma Wx.hx^k$	Fx (kips)	Vj (kips)
Roof	458	21.5	9847	0.48	1409.3	1409.3
2nd	939	11.5	10798.5	0.52	1545.5	2954.8
			20646			

	Length of Shear Walls in EW Dir. (ft)				Length of Shear Walls in NS Dir. (ft)			
	Concrete	Brick	CMU	Plywood	Concrete	Brick	CMU	Plywood
2nd				101.3			25.5	30.0
1st	65.5	239.0				120.0		
Basement	284.0				86.7			

2nd Floor:

East West Direction

At the second floor only plywood shear wall resist lateral loads

Plywood wall Length EW= 101.3 ft

Wood wall strength= 1000 lb/ft Table 17-6 ASCE 41-17

Shear Force= 1409.3 kips

Force in Plywood Shear Walls - Life Safety

$$v_j^{avg} = \frac{1}{M_s}\left(\frac{V_j}{A_w}\right)$$ ASCE 41-17 Eq. 4-9

Modification Factor (Ms.)= 3 Life Safety ASCE 41-17 Table 4-8

V/m= 470 kips

Forces in Plywood Shear Walls in EW Dir: Vij= 1546 lb/ft > 1000 lb/ft Not Good

North-South Direction

At the second floor plywood shear wall and two CMU vault walls resist lateral loads

Plywood Wall Length NS= 30.0 ft

Plywood Tributary Area= 1150 ft^2

Roof Area= 7347 ft^2

Plywood wall shear force= 220.6 kips

V/m= 74 kips

Forces in Plywood Shear Walls in NS Dir: Vij= 817 lb/ft < 1000 lb/ft OK

Force in CMU Shear Walls - Life Safety

CMU Wall Length NS= 25.5 ft

CMU Tributary Area= 6197 ft^2

CMU Shear Force= 396 kips

Area of Shear Walls in NS= 1591 in^2

Stress in CMU Shear Walls in NS Dir:

$$v_j^{avg} = \frac{1}{M_s}\left(\frac{V_j}{A_w}\right)$$ ASCE 41-17 Eq. 4-9

Modification Factor (Ms.)= 3 Life Safety ASCE 41-17 Table 4-8

Vij= 83 > 70 psi Not Good

1st Floor:

East West Direction

At the first floor concrete and brick shear walls resist lateral loads in the east-west direction

Force in Shear Walls - Life Safety

Story Shear= 2954.8 kips

Stress in Shear Walls in EW Dir:

Concrete Wall Length EW= 65.5 ft

Brick Wall Length EW= 239.0 ft

Area of Concrete Walls in EW= 7860 in^2

Area of Brick Walls in EW= 25811 in^2

$$v_j^{avg} = \frac{1}{M_s}\left(\frac{V_j}{A_w}\right)$$ ASCE 41-17 Eq. 4-9

Modification Factor (Ms.)= 3 Life Safety ASCE 41-17 Table 4-8

Vij= 29 psi < 70 psi OK

North South Direction

At the first d brick shear walls resist lateral loads in the north-south direction

Force in Shear Walls - Life Safety

Story Shear= 2954.8 kips

Stress in Shear Walls in NS Dir:

Brick Wall Length NS= 120.0 ft

Area of Brick Walls in NS= 12960 in^2

$$v_j^{avg} = \frac{1}{M_s}\left(\frac{V_j}{A_w}\right)$$ ASCE 41-17 Eq. 4-9

Modification Factor (Ms.)= 3 Life Safety ASCE 41-17 Table 4-8

Vij= 76 psi > 70 psi Not Good

Basement:

East West Direction

At the basement concrete walls resist lateral loads in the east-west direction

Force in Shear Walls - Life Safety

Story Shear= 2954.8 kips

Stress in Shear Walls in EW Dir:

Concrete Walls Length EW= 284.0 ft

Area of Concrete Walls in EW= 36286 in^2

$$v_j^{avg} = \frac{1}{M_s}\left(\frac{V_j}{A_w}\right)$$ ASCE 41-17 Eq. 4-9

Modification Factor (Ms.)= 3 Life Safety ASCE 41-17 Table 4-8

Vij= 27 psi < 100 psi OK

North-South Direction

At the basement concrete and brick walls resist lateral loads in the north-south direction

Force in Shear Walls - Life Safety

Story Shear= 2954.8 kips

Stress in Shear Walls in NS Dir:

Brick Walls Length NS= 120.0 ft

Concrete Walls Length NS= 86.7 ft

Area of Brick Walls in NS= 12960 in^2

Area of Concrete Walls in NS= 10399 in^2

$$v_j^{avg} = \frac{1}{M_s}\left(\frac{V_j}{A_w}\right)$$ ASCE 41-17 Eq. 4-9

Modification Factor (Ms.)= 3 Life Safety ASCE 41-17 Table 4-8

Vij= 42 psi < 70 psi OK

HOA Building Shear Wall Stress Check - Immediate Occupancy (IO)

Seismic Forces - BSE-1E

2nd Floor Plywood Shear Walls:

Base Shear - Equation 4-1 of ASCE 41-17: V = CSaW

	C=	1.3		Shear wall -1-story above concrete podium (rigid slab)			
	Sxs=	0.78		Seismic Maps			Figure 9
	Sx1=	0.432		Seismic Maps			Figure 9
	Ct=	0.02		Coefficient for Wood Structures			
	hn=	10	ft	Height of walls			
	Beta=	0.75		Coefficient for Wood Structures			
	T=	0.11	Sec	Fundamental period $T = C_t h_n^{\beta}$			Eq. 4-4
$S_a = \frac{S_{X1}}{T}$	Sa	0.78		Response spectral acceleration shall not exceed Sxs			
	C.Sa=	1.01					
Second Floor Sesmic Weight (W)=		180	kips	Including roof and exterior walls			
Second Floor Shear (V)=		182.5	kips				
Plywood Wall Length NS=		54	ft				
Plywood wall Length EW=		48	ft				
Wood wall strength=		1000	lb/ft	Table 17-6 ASCE 41-17			

Force in Shear Walls - Immediate Occupancy

$$v_j^{avg} = \frac{1}{M_s}\left(\frac{V_j}{A_w}\right)$$ ASCE 41-17 Eq. 4-9

Modification Factor (Ms)=		1.5	Immediate Occupancy			ASCE 41-17 Table 4-8	
	V/m=	122					
Forces in Shear Walls in NS Dir:	vj=	2253	lb/ft	>	1000	lb/ft	Not Good
Forces in Shear Walls in EW Dir:	vj=	2535	lb/ft	>	1000	lb/ft	Not Good

1st Floor Conceret and Masonery Walls:

Base Shear - Equation 4-1 of ASCE 41-17: V = CSaW

	C=	1.3		Shear wall -1-story			
	Sxs=	0.78		Seismic Maps			
	Sx1=	0.432		Seismic Maps			
	Ct=	0.02		Coefficient for Structures with Conceret and Masonry Shear Walls			
	h_n=	17.5	ft	Height of walls			
	Beta=	0.75		Coefficient for Wood Structures			
	T=	0.17	Sec	Fundamental period $T = C_t h_n^{\beta}$			Eq. 4-4
$S_a = \frac{S_{X1}}{T}$	Sa	0.78		Response spectral acceleration shall not exceed Sxs			
	C.Sa=	1.01					
Second Floor Sesmic Weight (W)=		473	kips	Including roof and exterior walls			
Second Floor Shear (V)=		479.6	kips				
Area of Shear Walls in NS=		8076	in^2				
Area of Shear Walls in EW=		7032	in^2				

Force in Shear Walls - Immediate Occupancy

$$v_j^{avg} = \frac{1}{M_s}\left(\frac{V_j}{A_w}\right)$$ ASCE 41-17 Eq. 4-9

Modification Factor (Ms)=		1.5	Immediate Occupancy			ASCE 41-17 Table 4-8	
Forces in Shear Walls in NS Dir:	vj=	40		<	70	psi	OK
Forces in Shear Walls in EW Dir:	vj=	45		<	70	psi	OK

HOA Building Shear Wall Stress Check - Life Safety (LS)

Seismic Forces - BSE-2E

2nd Floor Plywood Shear Walls:

	Base Shear - Equation 4-1 of ASCE 41-17: V = CSaW					
	C=	1.3		Shear wall -1-story above concrete podium (rigid slab)		
	Sxs=	1.627		Seismic Maps		
	Sx1=	0.862		Seismic Maps		
	Ct=	0.02		Coefficient for Wood Structures		
	h_n=	8.5	ft	Height of walls		
	Beta=	0.75		Coefficient for Wood Structures		
$T = C_t h_n^{\beta}$	T=	0.10	Sec	Fundamental period		Eq. 4-4
$S_a = \frac{S_{X1}}{T}$	Sa	1.627		Response spectral acceleration Shall not exceed Sxs		
	C.Sa=	2.12				
	Second Floor Sesmic Weight (W)=	180	kips	Including roof and exterior walls		
	Second Floor Shear (V)=	380.7	kips			
	Plywood Wall Length NS=	54	ft			
	Plywood wall Length EW=	48	ft			
	Wood wall strength=	1000	lb/ft	Table 17-6 ASCE 41-17		

Force in Shear Walls - Life Safety

$v_j^{avg} = \frac{1}{M_s}\left(\frac{V_j}{A_w}\right)$				ASCE 41-17 Eq. 4-9			
Modification Factor (Ms)=		3	Life Safety		ASCE 41-17 Table 4-8		
Forces in Shear Walls in NS Dir:	vj=	2350	lb/ft	>	1000	lb/ft	Not Good
Forces in Shear Walls in EW Dir:	vj=	2644	lb/ft	>	1000	lb/ft	Not Good

1st Floor Conceret and Masonery Walls:

	Base Shear - Equation 4-1 of ASCE 41-17: V = CSaW					
	C=	1.3		Shear wall -1-story		
	Sxs=	1.627		Seismic Maps		
	Sx1=	0.862		Seismic Maps		
	Ct=	0.02		Coefficient for Structures with Conceret and Masonry Shear Walls		
	h_n=	17.5	ft	Height of walls		
	Beta=	0.75		Coefficient for Wood Structures	$T = C_t h_n^{\beta}$	
	T=	0.17	Sec	Fundamental period		Eq. 4-4
$S_a = \frac{S_{X1}}{T}$	Sa	1.627		Response spectral acceleration Shall not exceed Sxs		
	C.Sa=	2.12				
	Second Floor Sesmic Weight (W)=	473	kips	Including roof and exterior walls		
	Second Floor Shear (V)=	1000.4	kips			
	Area of Shear Walls in NS=	8076	in3			
	Area of Shear Walls in EW=	7032	in^2			

Force in Shear Walls - Life Safety

$v_j^{avg} = \frac{1}{M_s}\left(\frac{V_j}{A_w}\right)$				ASCE 41-17 Eq. 4-9			
Modification Factor (Ms)=		3	Life Safety		ASCE 41-17 Table 4-8		
Forces in Shear Walls in NS Dir:	vj=	41		<	70	psi	OK
Forces in Shear Walls in EW Dir:	vj=	47		<	70	psi	OK

APPENDIX-B - TIER 2 CALCULATIONS

Deficiency Based Evaluation ASCE 41-17

City Hall Roof Weights:

Roof slope = 2.75:12		
horizontal projection	1.026	
Clay Tile Roofing	14	psf
2" (2"x6" T & G) roof sheathing	6	psf
Roof Rafters 2" x 8"	3	psf
Steel Roof Trusses @ 15'-3" OC	3.5	psf
W8 & W10 transverse Steel Beams	2.5	psf
Exterior walls	0	psf
Interior partition walls	0	psf
Sprinkler system	1.0	psf
Accoustical Ceiling	1	psf
HVAC duct work (8.0 psf)	5.0	psf
Miscellaneous	2.0	psf
Seismic Dead Weight	**38**	psf
Horizontal projection of DL	38.99	psf
Roof Live Load	**20.0**	psf
Interior partition walls	**15**	psf
Exterior walls	**20 - 25**	psf

City Hall Floor Weights:

4.5" thick Concrete Flooring	56.0	psf
Concretre beams	25	psf
Interior Partitions	0	psf
Exterior Partitions	0	psf
Floor covering	5	psf
Sprinkler system	1.0	psf
HVAC duct work (8.0 psf)	8.0	psf
Miscellaneous	3.0	psf
Seismic Dead Weight	**98.0**	psf
Floor Live Load	**40.0**	psf

Weight of City Hall Building Roof and Floor Diaphragms

Levels	Assembly	Unit Wt. (psf)	Area (sq-ft)	Weight (kips)	Story Wt. (kips)
Roof	Roof	38.99	9114	355.31	458.13
	Exterior Wall	23	4110	47.27	
	Interior Wall	15	7408	55.56	
2nd	Floor	98	7408	725.98	938.72
	Exterior Wall	23	4418	101.61	
	Interior Wall	15	7408	111.12	
Basement	Floor	0	7408	0.00	0.00
	Exterior Wall	0	4110	0.00	
	Interior Wall	0	7408	0.00	

Seismic Dead Wt. = 1396.85 kips

Weight of City Hall Building Fire Hose Tower

Levels	Assembly	Unit Wt. (psf)	Area (sq-ft)	Weight (kips)	Story Wt. (kips)
Roof	Roof	15.00	9	0.14	0.77
	Exterior Wall	30	42	0.63	
	Interior Wall	0	9	0.00	
2nd Tier	Floor	10	25	0.25	2.85
	Exterior Wall	20	130	2.60	
	Interior Wall	0	25	0.00	
1st Tier	Floor	10	49	0.49	6.37
	Exterior Wall	20	294	5.88	
	Interior Wall	0	49	0.00	

Seismic Dead Wt. = 9.99 kips

Table 13.5-1 Coefficients for Architectural Components

Other flexible components
High-deformability elements and attachments

a_p = 2.5
R_p = 3.5
Ω_o = 2.5
I_p = 1.5
h = 15.5 ft

Fire Hose Tower Seismic Forces (Two Orthogonal Directions)

Level	w_x (kips)	z (ft.)	F_p (k)	$F_{p\,(max)}$ (k)	$F_{p\,(min)}$ (k)	M_{OTM} (k-ft)
Top Tier	0.46	20.5	0.554	0.852	0.160	1.941
2nd Tier	1.87	17	1.996	3.501	0.656	14.917
1st Tier	7.67	10.5	6.038	14.358	2.692	78.313

DCR < 1.25 is OK for Immediate Occupancy

Level	Frame Width (ft.)	No. of Cols	P_{DL} (kips)	P_{comp} (kips)	P_n (kips)	Force in Tension Tie Rod (kips)	Tension Capacity of Tie Rod (kips)	DCR (Tension)
Top Tier	3.0	2.0	0.191	0.515	50.1			
2nd Tier	4.5	2.0	0.904	2.561	36.1			
1st Tier	6.5	2.0	2.496	8.520	38.1	17.177	11.137	1.542

ATC Mapped and Site-Specific Geotechnical Seismic Design Parameters

Hazard Level	Parameter	Mapped Value	Site Specific Value
BSE-1E	S_{xs}	0.78	
	S_{x1}	0.432	
BSE-2E	S_{xs}	1.356	
	S_{x1}	0.862	
BSE-1N	S_{xs}	1.234	
	S_{x1}	0.761	
BSE-2N	S_{xs}	1.851	
	S_{x1}	1.142	

City Hall Building Seismic Parameters

ASCE 41-17 Seismic Parameters (BSE-1E)

Seismic Importance Factor, I_E		1.25	
Response Modification Coefficient, R (North-South Direction)		5	
Response Modification Coefficient, R (East-West Direction)		5	
Deflection Amplification Factor, C_d		3.5	
Response spectral acceleration at short period, S_s =		1.170	g
Response spectral acceleration at a period 1sec, S_1 =		0.381	g
Soil Site Class		D	
Site Coefficient, F_a		1.0	
Site Coefficient, F_v		1.7	
Design response spectral acceleration at short period, S_{XS} =		0.780	g
Design response spectral acceleration at a period 1sec, S_{X1} =		0.432	g
BSE-1E conversion factor for S_s =		0.575	x BSE-2E S_s
BSE-1E conversion factor for S_1 =		0.501	x BSE-2E S_1
Seismic Design Category, SDC		D	
Approximate Fundamental Period, T_a =		0.20	sec
Calculated Time Period (North-South Direction) =		0.170	sec
Calculated Time Period (East-West Direction) =		0.170	sec
	T_s =	1.805	sec
	T_o =	0.361	sec
	B_1 =	1.0024	
	a =	60	(Site Class D)
	C_m =	1.0	(1-2 story concrete shear wall building)
	C_1C_2 =	1.147	
	C_1C_2 =	1.107	
	S_a =	0.778	g (North- South Direction)
	S_a =	0.778	g (East - West Direction)

DCR max=	1.25	DCR min =	1	$\mu_{strength}$ =	1.250	(North- South Direction)
DCR max=	1.25	DCR min =	1	$\mu_{strength}$ =	1.250	(East - West Direction)
C_1 =	1.144	C_2 =	1.003			(North- South Direction)
C_1 =	1.104	C_2 =	1.003			(East - West Direction)
J =	1	C_1C_2 J=	1.147			(North- South Direction)
J =	1	C_1C_2 J=	1.107			(East - West Direction)

Seismic Response Coefficients	0.893		(North- South Direction)
ASCE 41-17, equation 7-21	0.862		(East - West Direction)
Pseudo Seismic Base Shear, Vy =	**1239.25**	kips	(North-South Direction)
Pseudo Seismic Base Shear, Vx =	**1195.92**	kips	(East-West Direction)

Story Forces (North-South Direction)

Level	w_x (kips)	Area (sq-ft)	ΣW_x (kips)	h_x (ft.)	$w_x h_x$ (k-ft)	C_{vx}	F_x (k)	ΣF_x (kips)
Roof	449.23	9114	449.23	21.5	9658.42	0.472	585.18	585.18
2nd	938.74	7408	1387.97	11.5	10795.53	0.528	654.07	1239.25
Σ =	1387.97				20453.96	1.00	1239.25	

Diaphragm Forces (North-South Direction)

Level	F_{px} (k)	F_{px} (min) (k)	F_{px} (max) (k)	F_x/w_x	F_{px}/w_x	Trib. Wt. f_p (psf)
Roof	585.18	652.69	1305.37	1.303	1.303	64.207
2nd	838.16	1363.90	2727.81	0.697	0.893	113.142

Story Forces (East-West Direction)

Level	w_x (kips)	Area (sq-ft)	ΣW_x (kips)	h_x (ft.)	$w_x h_x$ (k-ft)	C_{vx}	F_x (k)	ΣF_x (kips)
Roof	449.23	9114.00	449.23	21.5	9658.42	0.472	564.72	564.72
2nd	938.74	7408.00	1387.97	11.5	10795.53	0.528	631.20	1195.92
Σ =	1387.97				20453.96	1.00	1195.92	

Diaphragm Forces (East-West Direction)

Level	F_{px} (k)	F_{px} (min) (k	F_{px} (max) (l	F_x/w_x	F_{px}/w_x	Trib. Wt. f_p (psf)
Roof	564.72	652.69	1305.37	1.257	1.257	61.962
2nd	808.85	1363.90	2727.81	0.672	0.862	109.186

Diaphragm Chord Steel Calculations (between lines 1 and 7) at line A and C
@ Line 3

Level	w_p	R_p	M_{p1}	T/C =	As (in^2)	Area of W10x21	Tension Chord Axial, T_{CE} (kips)	DCR (Chord)
Roof	2.954	139.94	3314.41	80.06	0.51	6.49	235.59	0.34
2nd	5.205	246.59	5840.52	141.08	0.90	6.49	235.59	0.60

Diaphragm Chord Steel Calculations (at lines 1 and 7)
@ Line 1 and 7

Level	w_p	R_p	M_{p1}	T/C =	As (in^2)	Area of W10x21	Tension Chord Axial, T_{CE} (kips)	DCR (Chord)
Roof	9.108	209.49	2409.16	18.21	0.12	6.49	210.28	0.09
2nd	16.050	369.16	4245.32	32.09	0.21	6.49	210.28	0.15

LATERAL SEISMIC FORCES & DIAPHRAGM FORCES			
Floor Levels	Building Factored Lateral Story Forces (kips)	Factored Diaphragm Forces (kips)	Factored Maximum Diaphragm Forces (kips)
City Hall Building North-South Direction (Seismic)			
Roof	585.18	585.18	
2nd	654.07	838.16	
Ground	0	0	
Base	1239.25	-	
City Hall Building - East-West Direction (Seismic)			
Roof	564.72	564.72	
2nd	631.20	808.85	
Ground	0	0	
Base	1195.92	-	

City Hall Building - Roof Trusses

Roof slope (2.75: 12) =		2.75	
Roof slope angle,	θ =	12.91	degree
Roof Truss span =		46.0	ft
Truss spacing =		15.3	ft
Roof pitch height, h =		5.27	ft
Roof Truss Dead Load, W_D =		39	psf
Uniform Truss DL, w_D =		594.75	lbs/ft

Determine Strength level Vertical forces at Roof Truss ends

Design response spectral acceleration at short period, S_{XS} =	0.780	g
Factored minimum Roof Dead Load, W_{u_DL} =	535.28	lbs/ft
Factored maximum Roof Dead Load, W_{u_DL} =	654.23	lbs/ft
Factored Horizontal Seismic force (ASCE 41-17, BSE-1E) at Truss #4 =	54.83	kips
Tension uplift force due to seismic lateral load = at a Roof Truss end	-9.03	kips
Compression Thrust force due to seismic lateral load = at a Roof Truss end	18.33	kips

Determine (ASD) Service level Vertical forces at Roof Truss ends

Design response spectral acceleration at short period, S_{XS} =	0.780	g
Service Level minimum Roof Dead Load, W_{DL} =	535.28	lbs/ft
Service level maximum Roof Dead Load, W_{DL} =	654.23	lbs/ft
ASD Horizontal Seismic force at Truss #4 & 5 =	39.17	lbs
Tension uplift force due to seismic lateral load = at a Roof Truss end	-9.97	lbs
Compression Thrust force due to seismic lateral load = at a Roof Truss end	17.39	lbs

knowledge factor =	1.0	
Factor of expected Strength =	1.1	
Fy =	33	ksi

Design of Clip Angle and weld at Roof Truss #4 (typical) - section B-B/S3

m_{IO} = 1.25 (for braces in tension)

Level	Angle Plate thickness (in.)	Assumed Length of Gusset Plate (in.)	Weld Size (in.)	Assumed Weld Length (in.)	Seismic Story Force (kips)
Roof	0.375	5.75	0.25	13.75	585.18

	Tension Force at Roof Truss (kips)	Tension Force at Weld (kips)	Tension Capacity of Gusset PL. (kips)	1/4" filled weld Tension Capacity (kips)	DCR (Tension) Gusset PL	DCR (Tension) Weld
Roof	9.35	9.35	78.27	66.35	0.12	0.14

Design of Truss and W10x21 edge beam connection (i.e, section B-B/S3)

A-307 bolts F_{nt} = 45ksi F_{nv} = 27 ksi

Level	Cap Plate thickness (in.)	Width of Cap Plate (in.)	Size of Anchor Bolts (in)	Number of Anchor Bolts	Tension Force at Roof Truss (kips)	Comp. Force at Diagonal Brace (kips)
Roof	0	0	0.625	4	9.35	0.00

	Tension Force at Anchor Bolts at Cap PL (kips)	Shear Force at Anchor Bolts at Cap PL (kips)	Tension Stress in Anchor Bolts (ksi)	Shear stress in Anchor Bolts (ksi)	Tension Capacity of Anchor Bolts (kip-ft)	Shear Capacity of Anchor Bolts (kips)	DCR (tension) at Bolts	DCR (Shear) at Bolts
Roof	9.4	27.42	7.62	22.34	18.22	15.93	0.51	1.721
3		Stress limit	4.5	2.7				

1934 - 1967 - ASTM A9 (building) , Fy = 33ksi
Area of 2 L 4 x 3 1/2 = 5.36 in^2

Roof Diaphragm In-plane Drag Forces at Truss #4 & 5 (i.e., drag trusses parallel to CMU vault below roof trusses)

Level	CMU vault Length (ft.)	Diaphragm Force (kips)	Seismic Story Force (kips)	Truss #4 & 5 Collector Force (kips)	Diaphragm Force at Truss #4 & #5 (kips)	Bottom Chord Truss Force at Truss #4	Drag Force at two sides of CMU vault	Drag Force at exterior end CMU vault	Drag Steel (in^2)	Additional Drag Steel Req'd. (in^2)	Anchor bolts & Steel Conn. per G/S-8	No. of sides of CMU vault with shear	Truss Shear Transfer AB's (kip/ft)
Roof	16.33	564.72	564.72	298.32	298.32	3.24	149.16	0.00	4.57	0.00	5.06	2	9.134

Roof Diaphragm In-plane Drag Force at Truss #9 (i.e., drag truss parallel to CMU vault below roof truss)

Level	CMU vault Length (ft.)	Diaphragm Force (kips)	Seismic Story Force (kips)	Truss #9 Collector Force (kips)	Diaphragm Force at Truss #9 (kips)	Chord Truss Force at Truss #9 (kip/ft)	Drag Force at CMU vault	Drag Force at ends of CMU vault	Drag Steel (in^2)	Additional Drag Steel Req'd. (in^2)	Anchor bolts & Steel Conn. per H/S-5	sides of CMU vault with shear dowels	Truss Shear Transfer AB's (in^2/ft)
Roof	11.33	564.72	564.72	164.32	164.32	2.93	81.43	81.43	2.49	0.00	5.57	1	14.503

Design of OCBF Beam

DCR < 1.25 is OK for Immediate Occupancy

knowledge factor = 1.0 Factor of expected Strength = 1.1 Fy = 33 ksi

Level	Beam Length (ft.)	Diaphragm Force (kips)	Seismic Story Force (kips)	Axial Comp. Force in Beam	Shear Force in Beam (kips)	Positive Moment at Beam (kip-ft)	Negative Moment at Beam (kip-ft)	Axial Capacity of Beam (kips)	Shear Capacity of Beam (kips)	Positive Moment Capacity of Beam (kip-	Negative Moment Capacity of Beam (kip-	DCR (Axial)	DCR (Shear)	DCR (Flexure) +	DCR (Flexure) -
Roof	15.25	564.72	564.72	39.60	19.50	33.97	26	212.03	35.44	42.11	42.11	0.19	0.55	0.81	0.617

Design of OCBF Column

DCR < 1.25 is OK for Immediate Occupancy

Level	Column Length (ft.)	Diaphragm Force (kips)	Seismic Story Force (kips)	Axial Tension Force in Column (kips)	Axial Comp. Force in Column (kips)	Shear Force in Column (kips)	Positive Moment in Column (kip-ft)	Negative Moment in Column (kip-ft)	Axial Comp. Capacity of Column	Axial Tension Capacity of Column (kips)	Shear Capacity of Column (kips)	Positive Moment Capacity of Column	Negative Moment Capacity of Column	DCR (Axial Comp.)	DCR (Axial Tension)	DCR (Shear)	DCR (Flexure) +	DCR (Flexure) -
Roof-2nd	15.25	564.72	564.72	1.50	35.36	0.25	0.5	0.50	38.10	45.08	4.704	7.05	7.05	0.93	0.03	0.053	0.071	0.071

Design of Tension Only Brace

DCR < 1.25 is OK for Immediate Occupancy

Level	Brace Frame Length (ft.)	Diaphragm Force (kips)	Seismic Story Force (kips)	Tension Force at Diagonal Brace	Diaphragm Force at Diagonal Brace (kips)	Area of Diagonal Brace (in^2)	Tension Capacity of Diagonal	ASCE 41-17 Brace average axial stress	1/4" weld connecti on yield	DCR (Tension)	DCR (Stress)	DCR (weld at gusset PL)	Status of Tension Brace
Roof-2nd	15.25	564.72	564.72	13.67	13.67	0.625	13.61	22.12	31.37	1.00	0.61	0.44	OK

Design of Gusset Plate and weld at Diagonal Brace at OCBF Beam-Column

Level	Gusset Plate thickness (in.)	Assumed Length of Gusset Plate (in.)	Weld Size (in.)	Assumed Weld Length (in.)	Seismic Story Force (kips)	Tension Force at Gusset Plate	Tension Force at Weld (kips)	Tension Capacity of Gusset PL. (kips)	1/4" filled weld Tension	DCR (Tension) Gusset PL	DCR (Tension) Weld
Roof-2nd	0.375	6	0.25	6.5	564.72	16.16	16.16	61.26	31.37	0.26	0.52

Design of Beam-Column connection of an OCBF

A-307 bol Fnt = 45ksi Fnv = 27 ksi

Level	Cap Plate thickness (in.)	Width of Cap Plate (in.)	Size of Anchor Bolts (in)	Number of Anchor Bolts	Tension Force at Diagonal Brace (kips)	Comp. Force at Diagonal Brace (kips)	Tension Force at Anchor Bolts at Cap PL	Shear Force at Anchor Bolts at Cap PL	Tension Stress in Anchor Bolts (ksi)	Shear stress in Anchor Bolts (ksi)	Tension Capacity of Anchor Bolts (kip-ft)	Shear Capacity of Anchor Bolts (kips)	DCR (tension) at Bolts	DCR (Shear) at Bolts
Roof	0.75	6	0.75	4	16.16	0.00	11.4	11.42	6.46	6.46	32.65	24.16	0.35	0.473
								Stress limit	4.5	2.7				

Design of Base Plate and Anchor Bolts of OCBF Pipe Column

Level	Base Plate thickness (in.)	Width of Base Plate (in.)	Size of Anchor Bolts (in)	Number of Anchor Bolts	Tension Force at Pipe Column (kips)	sion Force at Pipe Column	Force at Pipe Base Plate (kips)	Flexure Load at Base Plate (kip-ft)	Capacity of Anchor Bolts	Shear Capacity of Anchor Bolts (kips)	Flexure Capacity of Base Plate (kip-ft)	Capacity of Base Plate (kips)	DCR (tension) at Bolts	DCR (Shear) at Bolts	DCR (Flexure) at Base Plate	DCR (Shear) at Base Plate
Roof-2nd	0.75	6	0.75	2	1.50	35.36	1.1	0.13	19.24	26.51	1.276	65.34	0.08	0.040	0.098	0.011

Note: Ordinary Concentric Braced Frames are good only for 25% of the applied seismic forces based on the strength of OCBF members and capacities of their connections at joints and at the base.

Sesimic Forces on East-West Direction Plywood Shear Walls

Level Roof	F_x (k) =	585.18 kips	Area =	9114 (sq-ft)	f_x =	64.21 psf	(ASCE 41-17 BSE-1E, Seismic Story Force)
		202.19 kips		9114 (sq-ft)		22.18 psf	(ASCE 7-16 Seismic Story Force)

Shear wall system = 0.75 (Wood and CMU shear walls take 75% of the total story load)
OCBF system = 0.25 (Steel braced frames take 25% of the total story load)

ASCE 41-17 to ASCE 7-16 1.00 2.8942 (Enter this value if using ASCE 7-16 forces otherwise use 1)

Wood Shear walls shear distribution factor =	0.593	Shear force at all wood structural panels =	346.91	kips	Wood Structural wall panels
					m_{IO} = 1.7
CMU Shear walls shear distribution factor =	0.16	Shear force at all CMU shear walls =	91.97	kips	m_{LS} = 3.8
					m_{CP} = 4.5
OCBF System =	0.25	Shear force at all Ordinary Braced Frames =	146.29	kips	
		Total =	**585.18**	kips	ASCE 41-17 Seismic Story force

East-West direction Shear walls loads at the 2nd Level (Roof to 2nd) Wall heights = 9.15 ft

Wall (grid line)	Wall Trib. Area (ft^2)	ASCE 41-17 forces: ΣF_x (lbs)	ASCE 41-17 forces: F_{tot} (lbs)	Shear wall Length, b (ft.)	Wall Aspect Ratio	Wall Shear Load, v = $F_{tot}/(1.4*b)$ lbs/ft	Sheathing 1 or 2 sides	Allowable Shear 10d (2", 2", 12") (lbs/ft)	Edge Nail Spacing (in.)	Wall DCR (Shear)	Shear Status Check for IO	COLA/UCI Data, Tested Wall strength (lbs)	Applied Load to Tested Load Ratio
C1	1026	49407.06	49407.06	12.67	0.72	2785.38	1	870	2	3.20	Overstress	24034.99	2.056
C2	207.11	9973.51	9973.51	12.67	0.72	562.27	1	870	2	0.65	OK	24034.99	0.415
C3	1270.76	61193.37	61193.37	12.67	0.72	3449.85	1	870	2	3.97	Overstress	24034.99	2.546
C4	1518.20	73108.90	73108.90	11.42	0.80	4572.74	2	1740	2	2.63	Overstress	21663.74	3.375
A1	1026	49407.06	49407.06	12.67	0.72	2785.38	1	870	2	3.20	Overstress	24034.99	2.056
A2	331.38	15957.61	15957.61	12.67	0.72	899.63	1	870	2	1.03	OK	24034.99	0.664
A3	804.71	38750.84	38750.84	12.67	0.72	2184.62	1	870	2	2.51	Overstress	24034.99	1.612
A4	1019.91	49113.68	49113.68	12.67	0.72	2768.84	1	870	2	3.18	Overstress	24034.99	2.043
Σ	7204.07	346912.03	346912.03	100.11									

Sesimic Forces on North-South Direction Plywood Shear Walls

Level Roof	F_x (k) =	585.18 kips	Area =	9114 (sq-ft)	f_x = 64.21 psf	(ASCE 41-17 BSE-1E, Seismic Story Force)
		202.19 kips		9114 (sq-ft)	22.18 psf	(ASCE 7-16 Seismic Story Force)

Shear wall system =	0.75	(Wood shear walls take 75% of the tributary area based portion of total story load)			
OCBF system =	0.25	(Steel braced frames take 25% of the tributary area based portion of total story load)			
CMU walls =		(CMU shear walls take tributary area based portion of total story load)			
ASCE 41-17 to ASCE 7-16	1.00	2.8942 ← (Enter this value if using ASCE 7-16 forces otherwise use 1)			
Wood Shear walls shear distribution	0.137	Shear force at all wood structural panels =	80.30	kips	Wood Structural wall panels
					m_{IO} = 1.7
CMU Shear walls shear distribution factor =	0.82				m_{LS} = 3.8
		Shear force at all CMU shear walls =	478.11	kips	m_{CP} = 4.5
OCBF System =	0.25	Shear force at all Ordinary Braced Frames =	26.77	kips	
		Total =	**585.18**	kips	ASCE 41-17 Seismic Story force

North-South direction Shear walls loads at the 2nd Level (Roof to 2nd) Wall heights = 9.15 ft

Wall (grid line)	Wall Trib. Area (ft^2)	ASCE 41-17 forces ΣF_x (lbs)	ASCE 41-17 forces F_{tot} (lbs)	Shear wall Length, b (ft.)	Wall Aspect Ratio	Wall Shear Load, v = $F_{tot}/(1.4*b)$ lbs/ft	Sheathing 1 or 2 sides	Allowable Shear 10d (2", 2", 12") (lbs/ft)	Edge Nail Spacing (in.)	Wall DCR (Shear)	Shear Status Check for IO	COLA/UCI Data, Tested Wall strength (lbs)	Applied Load to Tested Load Ratio
1A	833.76	40149.74	40149.74	14.5	0.63	1977.82	1	870	2	2.27	Overstress	27506.5	1.460
1C	833.76	40149.74	40149.74	15.5	0.59	1850.22	1	870	2	2.13	Overstress	29403.5	1.365
Σ	1667.52	80299.48	80299.48	30.00									

Sesimic Forces on East-West Direction Reinforced Concrete Masonry Shear Walls

Level Roof

F_x (k) = 585.18 kips	Area = 9114 (sq-ft)	f_x = 64.21 psf	(ASCE 41-17 BSE-1E, Seismic Story Force)
202.19 kips	9114 (sq-ft)	22.18 psf	(ASCE 7-16 Seismic Story Force)

Shear wall system = 0.75 (Wood and CMU shear walls take 75% of the total story load)
OCBF system = 0.25 (Steel braced frames take 25% of the total story load)

ASCE 7-16 to ASCE 41-17 1 2.8942 (Enter this value if using ASCE 7-16 forces otherwise use 1)

Masonry Expected Strength factor = 1.3
Compressive Strength, f'm = 2500 psi
Allowable shear stress, f_{av} = 65 psi

Brick/Masonry walls
m_{IO} = 2
m_{LS} = 2
m_{CP} = 3

East-West direction Shear wall loads at 2nd Level (Roof to 2nd) Wall heights = 10.00 ft

Wall (grid line)	Wall Trib. Area (ft^2)	Wall Type	ASCE 41-17 forces ΣF$_x$ (lbs)	ASCE 41-17 forces F$_{tot}$ (lbs)	Shear wall Length, b (ft.)	Shear wall thickness, t (in)	Wall Aspect Ratio	Wall Shear Load, v = F$_{tot}$/(1.4*b) lbs/ft	Wall Shear Stress, f_v (psi)	DCR (Wall Shear)	Shear Stress Check	Wall Vertical Dowels Capacity (lbs)	DCR Wall Vertical (Dowels)	Slab Dowels Capacity (lbs)	DCR Slab (Dowels)
B1	460.08	CMU	22155.17	22155.17	10.5	8	0.95	1507.15	15.70	0.338	OK	38115	0.581	63525	0.349
B2	645.04	CMU	31061.68	31061.68	10.5	8	0.95	2113.04	22.01	0.474	OK	38115	0.815	63525	0.489
B3	324.12	CMU	15608.18	15608.18	21.75	8	0.46	512.58	5.34	0.115	OK	78952.5	0.198	131587.5	0.119
B4	480.68	CMU	23147.28	23147.28	21.75	8	0.46	760.17	7.92	0.171	OK	78952.5	0.293	131587.5	0.176
Σ	1909.92		91972.31	91972.31	64.50										

Sesimic Forces on North-South Direction Reinforced Concrete Masonry Shear Walls

Level Roof F_x (k) = 585.18 kips Area = 9114 (sq-ft) f_x = 64.21 psf (ASCE 41-17 BSE-1E, Seismic Story Force)
202.19 kips 9114 (sq-ft) 22.18 psf (ASCE 7-16 Seismic Story Force)

Shear wall system = 1 (Wood shear walls take 75% of the tributary area based portion of total story load)
OCBF system = 0.25 (Steel braced frames take 25% of the total story load)

ASCE 7-16 to ASCE 41-17 1 2.8942 (Enter this value if using ASCE 7-16 forces otherwise use 1)

Masonry Expected Strength factor = 1.3
Compressive Strength, f'm = 2500 psi
Allowable shear stress, f_{av} = 65 psi

Brick/Masonry walls
m_{IO} = 2
m_{LS} = 2
m_{CP} = 3

North-South direction Shear wall loads at 2nd Level (Roof to 2nd) Wall heights = 10.00 ft

Wall (grid line)	Wall Trib. Area (ft^2)	Wall Type	ASCE 41-17 forces ΣF$_x$ (lbs)	ASCE 41-17 forces F_{tot} (lbs)	Shear wall Length, b (ft.)	Shear wall thickness, t (in)	Wall Aspect Ratio	Wall Shear Load, v = F_{tot}/(1.4*b) lbs/ft	Wall Shear Stress, f_v (psi)	DCR (Wall Shear)	Shear Stress Check	Wall Vertical Dowels Capacity (lbs)	DCR Wall Vertical (Dowels)	Slab Dowels Capacity (lbs)	DCR Slab (Dowels)
3B	1799	CMU	115507.87	115507.87	16.5	8	0.61	5000.34	52.09	1.122	OK	59895	1.929	99825	1.157
3.5B	2051.00	CMU	131687.96	131687.96	16.5	8	0.61	5700.78	59.38	1.279	OK	59895	2.199	99825	1.319
6B	2644.48	CMU	169793.36	169793.36	11	8	0.91	11025.54	114.85	2.474	Not Good	39930	4.252	66550	2.551
7B	952.00	CMU	61124.79	61124.79	11	8	0.91	3969.14	41.35	0.891	OK	39930	1.531	66550	0.918
Σ	7446.48		478113.97	478113.97	55.00										

Torsional Analysis of Rigid Diaphragm

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DESCRIPTION: City Hall - CMU and Brick wall rigidities

General Information

Calculations per IBC 2012, CBC 2013, ASCE 7-1

Applied Lateral Force in "X" Direction	100.0 k	Center of Shear Application :	
Applied Lateral Force in "Y" Direction	100.0 k	Distance from "X" datum point	71.0 ft
		Distance from "Y" datum point	25.0 ft
Note: These loads are resolved into X & Y components when applied to the system of elements at angular increments		Accidental Torsion values per ASCE 7-05 12.8.4.2	
		Ecc. as % of Maximum Dimension	5.00 %
Load Orientation Angular Increment	30.0 deg	Maximum Dimensions :	
Load Location Angular Increment	15.0 deg	Along "X" Axis	142.0 ft
		Along "Y" Axis	43.330 ft
Center of Rigidity Location (calculated) . . .			
"X" dist. from Datum	72.296 ft		
"Y" dist. from Datum	18.223 ft		

Accidental Eccentricity +/- from "Y" Coord. of Center of Load Application : 7.10 ft
Accidental Eccentricity +/- from "X" Coord. of Center of Load Application : 2.167 ft

Torsional Analysis of Rigid Diaphragm

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DESCRIPTION: City Hall - CMU and Brick wall rigidities

Wall Information

Label : C5		X Wall C.G. Location	51.1875 ft	Length	7.9583 ft
		Y Wall C.G. Location	42.96 ft	Height	11.5 ft
Wall Deflections (Stiffness) for 1.0 kip load :		Wall Angle CCW	0 deg	Thickness	9 in
Along Wall "y" Dir	5.2794E-004 in	Wall Fixity	Fix-Fix	E - Bending	1 Mpsi
Along Wall "x" Dir	3.7942E+003 in			E - Shear	1 Mpsi
Label : C6		X Wall C.G. Location	63.049 ft	Length	8.1042 ft
		Y Wall C.G. Location	42.96 ft	Height	11.5 ft
Wall Deflections (Stiffness) for 1.0 kip load :		Wall Angle CCW	0 deg	Thickness	9 in
Along Wall "y" Dir	5.0669E-004 in	Wall Fixity	Fix-Fix	E - Bending	1 Mpsi
Along Wall "x" Dir	3.7258E+003 in			E - Shear	1 Mpsi
Label : C7		X Wall C.G. Location	87.4358 ft	Length	12.33 ft
		Y Wall C.G. Location	42.96 ft	Height	11.5 ft
Wall Deflections (Stiffness) for 1.0 kip load :		Wall Angle CCW	0 deg	Thickness	9 in
Along Wall "y" Dir	2.1451E-004 in	Wall Fixity	Fix-Fix	E - Bending	1 Mpsi
Along Wall "x" Dir	2.4489E+003 in			E - Shear	1 Mpsi

ANALYSIS SUMMARY

Maximum shear forces applied to resisting elements. Eccentricity with respect to Center of Rigidity

	Max Shear along Member Local "y-y" Axis				Max Shear along Member Local "x-x" Axis			
Resisting Element	Load Angle	X-Ecc (ft)	Y-Ecc (ft)	Shear Force (k)	Load Angle	X-Ecc (ft)	Y-Ecc (ft)	Shear Force (k
1A	90	8.40	6.78	5.143	0	1.30	4.61	0.000
1B	90	8.40	6.78	0.332	0	1.30	4.61	0.000
1C	90	8.40	6.78	32.187	0	1.30	8.94	0.000
5A	90	8.40	6.78	51.971	0	1.30	8.94	0.000
7A	90	8.40	6.78	4.292	0	1.30	4.61	0.000
7B	90	8.40	6.78	0.277	0	1.30	4.61	0.000
7C	90	8.40	6.78	12.092	0	1.30	8.94	0.000
A1	0	8.40	6.78	2.685	90	8.40	6.78	0.000
A2	0	8.40	6.78	9.675	90	8.40	6.78	0.000
A3	0	8.40	6.78	9.675	90	8.40	6.78	0.000
A4	0	8.40	6.78	9.675	90	8.40	6.78	0.000
A5	0	8.40	6.78	2.679	90	-0.54	8.87	0.000
A6	0	8.40	6.78	2.679	90	-0.54	8.87	0.000
B1	0	1.30	8.94	2.029	90	8.40	6.78	0.000
B2	0	1.30	8.94	5.591	90	8.40	6.78	0.000
B3	0	1.30	8.94	37.421	90	8.40	6.78	0.000
C1	0	1.30	8.94	0.205	90	8.40	6.78	0.000
C2	0	1.30	8.94	0.611	90	8.40	6.78	0.000
C3	0	1.30	8.94	0.314	90	8.40	6.78	0.000
C4	0	1.30	8.94	8.559	90	8.40	6.78	0.000
C5	0	1.30	8.94	2.480	90	8.40	6.78	0.000
C6	0	1.30	8.94	2.584	90	8.40	6.78	0.000
C7	0	1.30	8.94	6.105	90	-0.54	8.87	0.000

Torsional Analysis of Rigid Diaphragm

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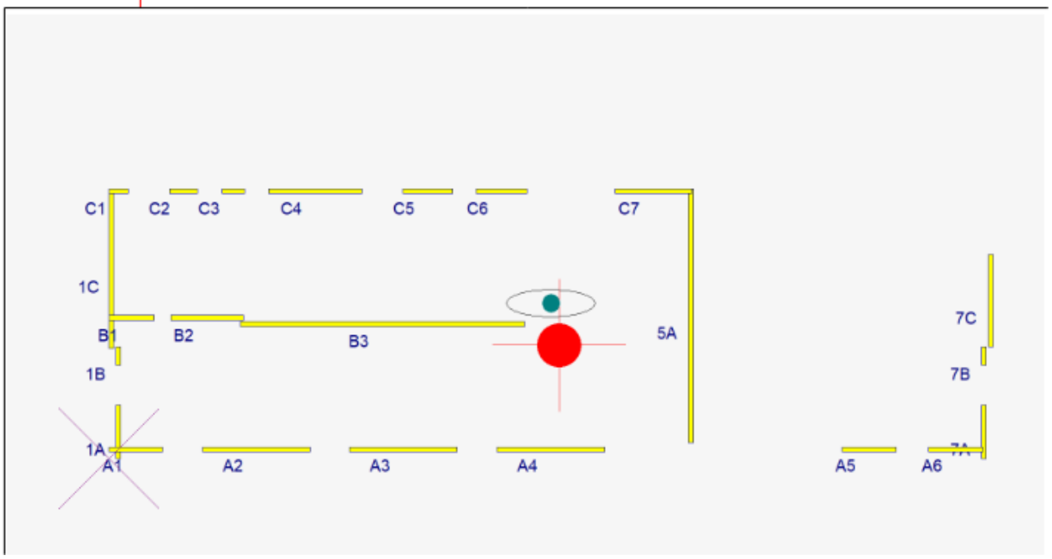
WALKER CONSULTANT

DESCRIPTION: City Hall - CMU and Brick wall rigidities

Layout of Resisting Elements

Legend : Defined Wall | X Datum
Center of Rigidity | Center of Mass | Accidental eccentricity application boundary



Sesimic Forces on East-West Direction Brick Shear Walls and Reinforced Concrete Masonry Shear walls

Level 2nd

F_x (k) = 1195.92 kips   Area = 7408 (sq-ft) f_x = 161.44 psf (ASCE 41-17 BSE-1E, Seismic Story Force)
428.19 kips   7408 (sq-ft)   57.80 psf (ASCE 7-16 Seismic Story Force)

Shear wall system = 1 (Brick and CMU shear walls take shear loads based on wall lateral stiffness or wall rigidities)

ASCE 7-16 to ASCE 41-17   1   2.7930   (Enter this value if using ASCE 7-16 forces otherwise use 1)

Masonry Expected Strength factor = 1.3
Compressive Strength, f'm = 2500 psi
Allowable shear stress, f_{av} = 65 psi (Brick and Masonry walls)
Allowable shear stress, f_{av} = 90 psi (CIP walls) with Expected Strength factor = 1.5

Brick/Masonry walls
m_{IO} = 2
m_{LS} = 2
m_{CP} = 3

East-West direction Shear wall loads at 1st Level (2nd to 1st)   Wall heights = 11.50 ft

			ASCE 41-17 forces												
Wall (grid line)	Wall Trib. Area (ft^2)	Wall Type	Shear Distribution Factor	F_{tot} (lbs)	Shear wall Length, b (ft.)	Shear wall thickness, t (in)	Wall Aspect Ratio	Wall Shear Load, v = $F_{tot}/(1.4*b)$ lbs/ft	Wall Shear Stress, f_v (psi)	DCR (Wall Shear)	Shear Stress Check	Wall Vertical Dowels Capacity (lbs)	DCR Wall Vertical (Dowels)	Slab Dowels Capacity (lbs)	DCR Slab (Dowels)
C1		CMU	0.00205	2451.64	3.06	9	3.76	572.28	5.30	0.114	OK	11107.8	0.221	18513	0.132
C2		CMU	0.00611	7307.08	4.54	9	2.53	1149.63	10.64	0.229	OK	16480.2	0.443	27467	0.266
C3		CMU	0.00314	3755.19	3.56	9	3.23	753.45	6.98	0.150	OK	12922.8	0.291	21538	0.174
C4		CMU	0.08559	102358.88	14.96	9	0.77	4887.26	45.25	0.975	OK	54304.8	1.885	90508	1.131
C5		CMU	0.02480	29658.84	7.96	9	1.44	2661.42	24.64	0.531	OK	28894.8	1.026	48158	0.616
C6		CMU	0.02584	30902.60	8.1	9	1.42	2725.10	25.23	0.543	OK	29403	1.051	49005	0.631
C7		CMU	0.06105	73010.98	12.33	9	0.93	4229.58	39.16	0.844	OK	44757.9	1.631	74596.5	0.979
B1		CIP	0.02029	24265.24	7.25	10	1.59	2390.66	19.92	0.310	OK	26317.5	0.922	43862.5	0.553
B2		CIP	0.05591	66863.94	11.67	10	0.99	4092.54	34.10	0.531	OK	42362.1	1.578	70603.5	0.947
B3		CIP	0.37421	447525.59	45.67	10	0.25	6999.37	58.33	0.907	OK	165782.1	2.699	276303.5	1.620
A1		CMU	0.02685	32110.48	8.67	9	1.33	2645.45	24.49	0.528	OK	31472.1	1.020	52453.5	0.612
A2		CMU	0.09675	115705.36	17.32	9	0.66	4771.75	44.18	0.952	OK	62871.6	1.840	104786	1.104
A3		CMU	0.09675	115705.36	17.32	9	0.66	4771.75	44.18	0.952	OK	62871.6	1.840	104786	1.104
A4		CMU	0.09675	115705.36	17.32	9	0.66	4771.75	44.18	0.952	OK	62871.6	1.840	104786	1.104
A5		CMU	0.02679	32038.72	8.66	9	1.33	2642.59	24.47	0.527	OK	31435.8	1.019	52393	0.612
A6		CMU	0.02679	32038.72	8.66	9	1.33	2642.59	24.47	0.527	OK	31435.8	1.019	52393	0.612
Σ	0.00		1.03	1231403.96	197.05										

Sesimic Forces on North-South Direction Brick Shear Walls and Reinforced Concrete Masonry Shear walls

Level 2nd

F_x (k) =	1239.25 kips	Area =	7408 (sq-ft)	f_x =	167.29 psf	(ASCE 41-17 BSE-1E, Seismic Story Force)
	428.19 kips		7408 (sq-ft)		57.80 psf	(ASCE 7-16 Seismic Story Force)

Shear wall system = 1 (Brick and CMU shear walls take shear loads based on wall lateral stiffness or wall rigidities)

ASCE 7-16 to ASCE 41-17 | 1 | 2.8942 (Enter this value if using ASCE 7-16 forces otherwise use 1)

Masonry Expected Strength factor =	1.3		Brick/Masonry walls	
Compressive Strength, f'm =	2500	psi	m_{IO} =	2
Allowable shear stress, f_{av} =	65	psi	m_{LS} =	2
			m_{CP} =	3

North-South direction Shear wall loads at 1st Level (2nd to 1st)

Wall heights = 11.50 ft

Wall (grid line)	Wall Trib. Area (ft^2)	Wall Type	ASCE 41-17 forces: Shear Distribution Factor	ASCE 41-17 forces: F_{tot} (lbs)	Shear wall Length, b (ft.)	Shear wall thickness, t (in)	Wall Aspect Ratio	Wall Shear Load, v = $F_{tot}/(1.4*b)$ lbs/ft	Wall Shear Stress, f_v (psi)	DCR (Wall Shear)	Shear Stress Check	Wall Vertical Dowels Capacity (lbs)	DCR Wall Vertical (Dowels)	Slab Dowels Capacity (lbs)	DCR Slab (Dowels)
1A		CMU	0.0514	63734.85	8.66	9	1.33	5256.92	48.68	1.048	OK	34293.6	1.859	57156	1.115
1B		CMU	0.0033	4114.32	3	9	3.83	979.60	9.07	0.195	OK	11880	0.346	19800	0.208
1C		CMU	0.3219	398878.78	25.58	9	0.45	11138.13	103.13	2.221	Not Good	101296.8	3.938	168828	2.363
5A		CMU	0.5197	644052.84	40.75	9	0.28	11289.27	104.53	2.251	Not Good	161370	3.991	268950	2.395
7A		CMU	0.0429	53188.79	8.66	9	1.33	4387.07	40.62	0.875	OK	34293.6	1.551	57156	0.931
7B		CMU	0.0028	3432.73	3	9	3.83	817.32	7.57	0.163	OK	11880	0.289	19800	0.173
7C		CMU	0.1209	149850.63	14.83	9	0.78	7217.54	66.83	1.439	Not Good	58726.8	2.552	97878	1.531
Σ			1.06	1317252.95	104.48										

Sesimic Forces on East-West Direction Brick Shear Walls and Reinforced Concrete Masonry Shear walls

Level 1st	F_x (k) =	1195.92 kips	Area =	7408 (sq-ft)	f_x = 161.44 psf	(ASCE 41-17 BSE-1E, Seismic Story Force)
		428.19 kips		7408 (sq-ft)	57.80 psf	(ASCE 7-16 Seismic Story Force)

Shear wall system = 1 (Brick and CMU shear walls take shear loads based on wall lateral stiffness or wall rigidities)

ASCE 7-16 to ASCE 41-17 1 2.7930 ← (Enter this value if using ASCE 7-16 forces otherwise use 1)

Masonry Expected Strength factor =	1.3		Brick/Masonry walls	
Compressive Strength, f'm =	2500	psi	m_{IO} =	2
Allowable shear stress, f_{av} =	65	psi (Brick and Masonry walls)	m_{LS} =	2
Allowable shear stress, f_{av} =	90	psi (CIP walls) with Expected Strength factor = 1.5	m_{CP} =	3

East-West direction Shear wall loads at 1st Level (1st to Fdn.) Wall heights = 10.00 ft

			ASCE 41-17 forces												
Wall (grid line)	Wall Trib. Area (ft^2)	Wall Type	Shear Distribution Factor	F_{tot} (lbs)	Shear wall Length, b (ft.)	Shear wall thickness, t (in)	Wall Aspect Ratio	Wall Shear Load, v = $F_{tot}/(1.4*b)$ lbs/ft	Wall Shear Stress, f_v (psi)	DCR (Wall Shear)	Shear Stress Check	Foundation Dowels Capacity (lbs)	DCR Foundation (Dowels)	Slab Dowels Capacity (lbs)	DCR Slab (Dowels)
C1		CIP	0.33000	394653.93	74	10	0.16	3809.40	31.75	0.684	OK	805860	0.490	537240	0.735
B1		CIP	0.35000	418572.34	92.68	12	0.12	3225.94	22.40	0.483	OK	1345713.6	0.311	672856.8	0.622
A1		CIP	0.37000	442490.76	94.68	10	0.12	3338.24	27.82	0.599	OK	1031065.2	0.429	687376.8	0.644
Σ	0.00		1.05	1255717.03	261.36										

Sesimic Forces on North-South Direction Brick Shear Walls and Reinforced Concrete Masonry Shear walls

Level 1st	F_x (k) = 1239.25 kips	Area = 7408 (sq-ft)	f_x = 167.29 psf	(ASCE 41-17 BSE-1E, Seismic Story Force)
	428.19 kips	7408 (sq-ft)	57.80 psf	(ASCE 7-16 Seismic Story Force)

Shear wall system = 1 (Brick and CMU shear walls take shear loads based on wall lateral stiffness or wall rigidities)

ASCE 7-16 to ASCE 41-17 1 2.8942 (Enter this value if using ASCE 7-16 forces otherwise use 1)

Masonry Expected Strength factor =	1.3		Brick/Masonry walls	
Compressive Strength, f'm =	2500	psi	m_{IO} =	2
Allowable shear stress, f_{av} =	65	psi (Brick and Masonry walls)	m_{LS} =	2
Allowable shear stress, f_{av} =	90	psi (CIP walls) with Expected Strength factor = 1.5	m_{CP} =	3

North-South direction Shear wall loads at 1st Level (1st to Fdn.) Wall heights = 10.00 ft

			ASCE 41-17 forces													
Wall (grid line)	Wall Trib. Area (ft^2)	Wall Type	Shear Distribution Factor	F_{tot} (lbs)	Shear wall Length, b (ft.)	Shear wall thickness, t (in)	Wall Aspect Ratio	Wall Shear Load, v = $F_{tot}/(1.4*b)$ lbs/ft	Wall Shear Stress, f_v (psi)	DCR (Wall Shear)	Shear Stress Check	Foundation Dowels Capacity (lbs)	DCR Foundation (Dowels)	Slab Dowels Capacity (lbs)	DCR Slab (Dowels)
1		CIP	0.3800	470916.63	47.33	10	0.24	7106.89	59.22	1.276	OK	515423.7	0.914	312378	1.508
5		CMU	0.6500	805515.29	77.33	9	0.15	7440.42	68.89	1.484	Not Good	306226.8	2.630	510378	1.578
5		CIP	0.6500	805515.29	77.33	10	0.15	7440.42	62.00	1.335	OK	842123.7	0.957	510378	1.578
Σ			1.03	2081947.20	124.66										

HOA Building Roof Weights:

Roof slope = 2.75:12		
horizontal projection	1.026	
Clay Tile Roofing	14	psf
15/32" 3-ply (OSB/plywood) sheathing	2.625	psf
Premanufactured Wood Trusses	3.5	psf
Insulation (fibrous glass)	1.5	psf
Sprinkler system	1.0	psf
Accoustical Ceiling	1.0	psf
Miscellaneous	2.0	psf
HVAC duct work (8.0 psf)	8.0	psf
Seismic Dead Weight	**33.625**	psf
Horizontal projection of DL	34.50	psf
Roof Live Load	**20.0**	psf
Interior partition walls	**15**	psf
Exterior walls	**20 - 25**	psf

HOA Building Floor Weights:

Flooring	1.5	psf
2 1/2" thick topping overlay - (light wt. concrete)	23.96	psf
10" deep hollow core slab	68	psf
Steel Beams and Columns	2.5	psf
Insulation (fibrous glass)	0	psf
Sprinkler system	1.0	psf
Ceiling plaster	0.0	psf
Miscellaneous	2.0	psf
HVAC duct work (8.0 psf)	0.0	psf
Seismic Dead Weight	**98.96**	psf
Floor Live Load	**40.0**	psf

Weight of HOA Building Roof and Floor Diaphragms

Levels	Assembly	Unit Wt. (psf)	Area (sq-ft)	Weight (kips)	Story Wt. (kips)
HOA Roof	Roof	34.5	3135	108.16	163.74
	Exterior Wall	45	854	38.43	
	Interior Wall	15	2287	17.15	
Bridge Roof	Roof	34.50	270	9.31	16.96
	Exterior Wall	45	170	7.65	
	Interior Wall	0	120	0.00	
HOA 2nd	Floor	99.00	2287	226.41	299.15
	Exterior Wall	45	854	38.43	
	Interior Wall	15	2287	34.31	
Bridge 2nd	Floor	99.00	120	11.88	19.53
	Exterior Wall	45	170	7.65	
	Interior Wall	0	120	0.00	

Seismic Dead Wt. = 499.38 kips

HOA Building Seismic Parameters (Roof Level)

ASCE 41-17 Seismic Parameters (BSE-1E)

Seismic Importance Factor, I_E					1.25	
Response Modification Coefficient, R (North-South Direction)					6.5	
Response Modification Coefficient, R (East-West Direction)					6.5	
Deflection Amplification Factor, C_d					4.0	
Response spectral acceleration at short period, S_s =					1.170	g
Response spectral acceleration at a period 1sec, S_1 =					0.381	g
Soil Site Class					D	
Site Coefficient, F_a					1.0	
Site Coefficient, F_v					1.7	
Design response spectral acceleration at short period, S_{XS} =					0.780	g
Design response spectral acceleration at a period 1sec, S_{X1} =					0.432	g
BSE-1E conversion factor for S_s =					0.575	x BSE-2E S_s
BSE-1E conversion factor for S_1 =					0.501	x BSE-2E S_1
Seismic Design Category, SDC					D	
Approximate Fundamental Period, T_a =					0.12	sec
Calculated Time Period (North-South Direction) =					0.143	sec
Calculated Time Period (East-West Direction) =					0.143	sec
				T_s =	1.805	sec
				T_o =	0.361	sec
				B_1 =	1.0024	
				a =	60	(Site Class D)
				C_m =	1.0	(1-2 story concrete shear wall building)
				C_1C_2 =	1.208	
				C_1C_2 =	1.208	
DCR max=	1.25	DCR min =	1	$\mu_{strength}$ =	1.250	(North- South Direction)
DCR max=	1.25	DCR min =	1	$\mu_{strength}$ =	1.250	(East - West Direction)
		C_1 =	1.204	C_2 =	1.004	(North- South Direction)
		C_1 =	1.204	C_2 =	1.004	(East - West Direction)
		J =	1	C_1C_2 J=	1.208	(North- South Direction)
		J =	1	C_1C_2 J=	1.208	(East - West Direction)
				S_a =	0.778	g (North- South Direction)
				S_a =	0.778	g (East - West Direction)
Seismic Response Coefficients					0.940	(North- South Direction)
ASCE 41-17, equation 7-21					0.940	(East - West Direction)
Pseudo Seismic Base Shear, Vy =					**169.93**	kips (North-South Direction)
Pseudo Seismic Base Shear, Vx =					**169.93**	kips (East-West Direction)

Story Forces (North-South Direction)

Level	w_x (kips)	Area (sq-ft)	ΣW_x (kips)	h_x (ft.)	$w_x h_x$ (k-ft)	C_{vx}	F_x (k)	ΣF_x (kips)
Roof	180.70	3405	180.70	11.5	2078.09	1.000	169.93	169.93
Σ =	180.70				2078.09	1.00	169.93	

Diaphragm Forces (North-South Direction)

Level	F_{px} (k)	F_{px} (min) (k)	F_{px} (max) (k)	F_x/w_x	F_{px}/w_x	Trib. Wt. f_p (psf)
Roof	169.93	262.54	525.09	0.940	0.940	49.907

Story Forces (East-West Direction)

Level	w_x (kips)	Area (sq-ft)	ΣW_x (kips)	h_x (ft.)	$w_x h_x$ (k-ft)	C_{vx}	F_x (k)	ΣF_x (kips)
Roof	180.70	3405.00	180.70	11.5	2078.09	1.000	169.93	169.93
Σ =	180.70				2078.09	1.00	169.93	

Diaphragm Forces (East-West Direction)

Level	F_{px} (k)	F_{px} (min) (k)	F_{px} (max) (k)	F_x/w_x	F_{px}/w_x	Trib. Wt. f_p (psf)
Roof	169.93	262.54	525.09	0.940	0.940	49.907

Diaphragm Chord Steel Calculations (between lines 0.1 and 0.6) at line A and B.1
@ between center of Line 0.3 and 0.4

Level	w_p	R_p	M_{p1}	T/C =	As (in^2)	Area of W21x68 (in^2)	Tension Chord Axial, T_{CE} (kips)	DCR (Chord)
Roof	1.996	69.87	1479.49	41.10	0.16	20.00	726.00	0.06

Diaphragm Chord Steel Calculations (at lines 0.1 and 0.6)
@ Line 0.1 and 0.6

Level	w_p	R_p	M_{p1}	T/C =	As (in^2)	Area of 2x8 Chord Member (in^2)	Tension Chord Axial, T_{CE} (kips)	DCR (Chord)
Roof	3.843	76.86	768.56	11.09	0.04	21.75	20.01	0.55

HOA Building - Roof Trusses

Roof slope (2.75: 12) =		2.75
Roof slope angle,	θ =	12.91 degree
Roof Truss span =		40.0 ft
Truss spacing =		2.0 ft
Roof pitch height, h =		4.58 ft
Roof Truss Dead Load, W_D =		31.5 psf
Uniform Truss DL, w_D =		63 lbs/ft

Determine Strength level Vertical forces at Roof Truss ends

Design response spectral acceleration at short period, S_{XS} =	0.780 g
Factored minimum Roof Dead Load, W_{u_DL} =	56.70 lbs/ft
Factored maximum Roof Dead Load, W_{u_DL} =	69.30 lbs/ft
Factored Horizontal Seismic force (ASCE 41-17, BSE-1E) at a typical Roof Truss =	5.27 kips
Tension uplift force due to seismic lateral load = at a Roof Truss end	-0.77 kips
Compression Thrust force due to seismic lateral load = at a Roof Truss end	1.75 kips

Determine (ASD) Service level Vertical forces at Roof Truss ends

Design response spectral acceleration at short period, S_{XS} =	0.780 g
Service Level minimum Roof Dead Load, W_{DL} =	56.70 lbs/ft
Service level maximum Roof Dead Load, W_{DL} =	69.30 lbs/ft
ASD Horizontal Seismic force at Truss #4 & 5 =	3.77 lbs
Tension uplift force due to seismic lateral load = at a Roof Truss end	-0.88 lbs
Compression Thrust force due to seismic lateral load = at a Roof Truss end	1.64 lbs

Vertical Load Effect on HOA Pedestrian Bridge

Ev =	5.71	kips	ASCE 41-17 Level load per ASCE 7-16 section 12.4.2.2
Upward load on cantilever projection only	7.322	kips	ASCE 7-16 section 12.4.4
Sideway load on cantilever projection	7.322	kips	
Seismic load for a simply supported bridge at Roof Level	6.35	kips	ASCE 41-17 Level load per section 7.2.10 & 7.2.11.1
Seismic load for a simply supported bridge at 2nd Level	13.66	kips	ASCE 41-17 Level load per section 7.2.10 & 7.2.11.1

Shear load at bridge support = at Roof level =	1.12	kips per connection
Shear load at bridge support = at 2nd level	2.41	kips per anchor bolt see 24/S-4 and 25/S-4
3/4" dia. Anchor Bolt Steel Shear capacity	8.97	kips per anchor bolt
3/4" dia. Anchor Bolt shear capacity at top of grouted CMU wall	0.5	kips (Governs)
DCR (Shear) - anchors =	4.823	(Not Good)

Sesimic Forces on East-West Direction Plywood Shear Walls

Level Roof	F_x (k) =	169.93 kips	Area =	3135 (sq-ft)	f_x = 54.20 psf	(ASCE 41-17 BSE-1E, Seismic Story Force)
		36.11 kips		3135 (sq-ft)	11.52 psf	(ASCE 7-16 Seismic Story Force)

Shear wall system = 1 (Wood shear walls take 100% of the total story load)

ASCE 41-17 to ASCE 7-16 1.00 4.7058 ← (Enter this value if using ASCE 7-16 forces otherwise use 1)

Wood Shear walls shear distribution factor = 1.086 Shear force at all wood structural panels = 184.57 kips

Wood Structural wall panels Total = **184.57** kips ASCE 41-17 Seismic Story force

m_{IO} = 1.7
m_{LS} = 3.8
m_{CP} = 4.5

East-West direction Shear walls loads at the 2nd Level (Roof to 2nd) Wall heights = 8.5 ft

		ASCE 41-17 forces											
Wall (grid line)	Wall Trib. Area (ft^2)	ΣF_x (lbs)	F_{tot} (lbs)	Shear wall Length, b (ft.)	Wall Aspect Ratio	Wall Shear Load, v = $F_{tot}/(1.4*b)$ lbs/ft	Sheathing 1 or 2 sides	Allowable Shear 8d (2.5", 2.5", 12") (lbs/ft)	Edge Nail Spacing (in.)	Wall DCR (Shear)	Shear Status Check for IO	COLA/UCI Data, Tested Wall strength (lbs)	Applied Load to Tested Load Ratio
B.1A	389.92	21135.61	21135.61	8.42	1.01	1792.98	1	640	2.5	2.80	Overstress	15972.74	1.323
B.1B	1177.58	63830.72	63830.72	13	0.65	3507.18	1	640	2.5	5.48	Overstress	24661	2.588
A1	547.42	29672.90	29672.90	9.92	0.86	2136.59	1	640	2.5	3.34	Overstress	18818.24	1.577
A2	601.88	32624.90	32624.90	16.42	0.52	1419.21	1	640	2.5	2.22	Overstress	31148.74	1.047
A3	688.2	37303.88	37303.88	8.92	0.95	2987.18	1	640	2.5	4.67	Overstress	16921.24	2.205
Σ	3405.00	184568.01	184568.01	56.68									

Sesimic Forces on North-South Direction Plywood Shear Walls

Level Roof	F_x (k) =	169.93 kips	Area =	3135 (sq-ft)	f_x =	54.20 psf	(ASCE 41-17 BSE-1E, Seismic Story Force)
		36.11 kips		3135 (sq-ft)		11.52 psf	(ASCE 7-16 Seismic Story Force)

Shear wall system = 1 (Wood shear walls take 100% of the total story load)

ASCE 41-17 to ASCE 7-16 1.00 4.7058 (Enter this value if using ASCE 7-16 forces otherwise use 1)

Wood Shear walls shear distribution 1.086 Shear force at all wood structural panels = 184.57 kips

Wood Structural wall panels Total = **184.57** kips ASCE 41-17 Seismic Story force

m_{IO} = 1.7
m_{LS} = 3.8
m_{CP} = 4.5

North-South direction Shear walls loads at the 2nd Level (Roof to 2nd) Wall heights = 8.5 ft

Wall (grid line)	Wall Trib. Area (ft^2)	ASCE 41-17 forces: ΣF_x (lbs)	ASCE 41-17 forces: F_{tot} (lbs)	Shear wall Length, b (ft.)	Wall Aspect Ratio	Wall Shear Load, v = $F_{tot}/(1.4*b)$ lbs/ft	Sheathing 1 or 2 sides	Allowable Shear 8d (2.5", 2.5", 12") (lbs/ft)	Edge Nail Spacing (in.)	Wall DCR (Shear)	Shear Status Check for IO	COLA/UCI Data, Tested Wall strength (lbs)	Applied Load to Tested Load Ratio
0.1A	696.67	37762.99	37762.99	6.25	1.36	4315.77	1	640	2.5	6.74	Overstress	11856.25	3.185
0.1B	409.25	22183.39	22183.39	7.33	1.16	2161.70	1	640	2.5	3.38	Overstress	13905.01	1.595
0.1C	461.5	25015.61	25015.61	6.25	1.36	2858.93	1	640	2.5	4.47	Overstress	11856.25	2.110
0.6A	857.75	46494.34	46494.34	13.25	0.64	2506.43	1	640	2.5	3.92	Overstress	25135.25	1.850
0.6B	527	28566.03	28566.03	6.75	1.26	3022.86	1	640	2.5	4.72	Overstress	12804.75	2.231
0.6C	452.83	24545.65	24545.65	6.5	1.31	2697.32	1	640	2.5	4.21	Overstress	12330.5	1.991
Σ	3405.00	184568.01	184568.01	46.33									

HOA Building Seismic Parameters (2nd Level)

ASCE 41-17 Seismic Parameters (BSE-1E)

Seismic Importance Factor, I_E					1.25	
Response Modification Coefficient, R (North-South Direction)					5	
Response Modification Coefficient, R (East-West Direction)					5	
Deflection Amplification Factor, C_d					3.5	
Response spectral acceleration at short period, S_s =					1.170	g
Response spectral acceleration at a period 1sec, S_1 =					0.381	g
Soil Site Class					D	
Site Coefficient, F_a					1.0	
Site Coefficient, F_v					1.7	
Design response spectral acceleration at short period, S_{XS} =					0.780	g
Design response spectral acceleration at a period 1sec, S_{X1} =					0.432	g
BSE-1E conversion factor for S_s =					0.575	x BSE-2E S_s
BSE-1E conversion factor for S_1 =					0.501	x BSE-2E S_1
Seismic Design Category, SDC					D	
Approximate Fundamental Period, T_a =					0.17	sec
Calculated Time Period (North-South Direction) =					0.200	sec
Calculated Time Period (East-West Direction) =					0.200	sec
				T_s =	1.805	sec
				T_o =	0.361	sec
				B_1 =	1.0024	
				a =	60	(Site Class D)
				C_m =	1.0	(1-2 story concrete shear wall building)
				C_1C_2 =	1.106	
				C_1C_2 =	1.106	
DCR max=	1.25	DCR min =	1	$\mu_{strength}$ =	1.250	(North- South Direction)
DCR max=	1.25	DCR min =	1	$\mu_{strength}$ =	1.250	(East - West Direction)
		C_1 =	1.104	C_2 =	1.002	(North- South Direction)
		C_1 =	1.104	C_2 =	1.002	(East - West Direction)
		J =	1	C_1C_2 J=	1.106	(North- South Direction)
		J =	1	C_1C_2 J=	1.106	(East - West Direction)
				S_a =	0.778	g (North- South Direction)
				S_a =	0.778	g (East - West Direction)
Seismic Response Coefficients					0.861	(North- South Direction)
ASCE 41-17, equation 7-21					0.861	(East - West Direction)
Pseudo Seismic Base Shear, Vy =					**429.96**	kips (North-South Direction)
Pseudo Seismic Base Shear, Vx =					**429.96**	kips (East-West Direction)

Story Forces (North-South Direction)

Level	w_x (kips)	Area (sq-ft)	ΣW_x (kips)	h_x (ft.)	$w_x h_x$ (k-ft)	C_{vx}	F_x (k)	ΣF_x (kips)
2nd	499.38	2407	499.38	17.15	8564.37	1.000	429.96	429.96
Σ =	499.38				8564.37	1.00	429.96	

Diaphragm Forces (North-South Direction)

Level	F_{px} (k)	F_{px} (min) (k)	F_{px} (max) (k)	F_x/w_x	F_{px}/w_x	Trib. Wt. f_p (psf)
2nd	429.96	725.55	1451.10	0.861	0.861	178.629

Story Forces (East-West Direction)

Level	w_x (kips)	Area (sq-ft)	ΣW_x (kips)	h_x (ft.)	$w_x h_x$ (k-ft)	C_{vx}	F_x (k)	ΣF_x (kips)
2nd	499.38	2407.00	499.38	17.15	8564.37	1.000	429.96	429.96
Σ =	499.38				8564.37	1.00	429.96	

Diaphragm Forces (East-West Direction)

Level	F_{px} (k)	F_{px} (min) (k)	F_{px} (max) (k)	F_x/w_x	F_{px}/w_x	Trib. Wt. f_p (psf)
2nd	429.96	725.55	1451.10	0.861	0.861	178.629

Diaphragm Chord Steel Calculations (between lines 0.1 and 0.6) at line A and B.1
@ between center of Line 0.3 and 0.4

Level	w_p	R_p	M_{p1}	T/C =	As (in^2)	Area of chord bars (in^2)	Tension Chord Axial, T_{CE} (kips)	DCR (Chord)
2nd	7.145	250.08	5295.46	147.10	1.13	0.88	31.94	4.60

Diaphragm Chord Steel Calculations (at lines 0.1 and 0.6)
@ Line 0.1 and 0.6

Level	w_p	R_p	M_{p1}	T/C =	As (in^2)	Area of chord bars (in^2)	Tension Chord Axial, T_{CE} (kips)	DCR (Chord)
2nd	13.754	275.09	2750.89	39.70	0.30	0.44	15.97	2.49

Torsional Analysis of Rigid Diaphragm

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DESCRIPTION: HOA Bldg - CIP and CMU wall rigidities

General Information Calculations per IBC 2012, CBC 2013, ASCE 7-1

Applied Lateral Force in "X" Direction 100.0 k
Applied Lateral Force in "Y" Direction 100.0 k

Note: These loads are resolved into X & Y components when applied to the system of elements at angular increments.

Load Orientation Angular Increment 30.0 deg
Load Location Angular Increment 15.0 deg

Center of Rigidity Location (calculated) . . .
"X" dist. from Datum 24.334 ft
"Y" dist. from Datum 13.419 ft

Center of Shear Application :
Distance from "X" datum point 35.750 ft
Distance from "Y" datum point 18.750 ft

Accidental Torsion values per ASCE 7-05 12.8.4.2
Ecc. as % of Maximum Dimension 5.00 %

Maximum Dimensions :
Along "X" Axis 71.50 ft
Along "Y" Axis 37.50 ft

Accidental Eccentricity +/- from "Y" Coord. of Center of Load Application : 3.575 ft
Accidental Eccentricity +/- from "X" Coord. of Center of Load Application : 1.875 ft

Torsional Analysis of Rigid Diaphragm

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DESCRIPTION: HOA Bldg - CIP and CMU wall rigidities

Wall Information

Label : B.1B		X Wall C.G. Location	64 ft	Length	9.67 ft
Wall Deflections (Stiffness) for 1.0 kip load :		Y Wall C.G. Location	37.165 ft	Height	17.15 ft
Along Wall "y" Dir	9.6333E-004 in	Wall Angle CCW	0 deg	Thickness	8 in
Along Wall "x" Dir	1.4697E+004 in	Wall Fixity	Fix-Fix	E - Bending	1 Mpsi
				E - Shear	1 Mpsi

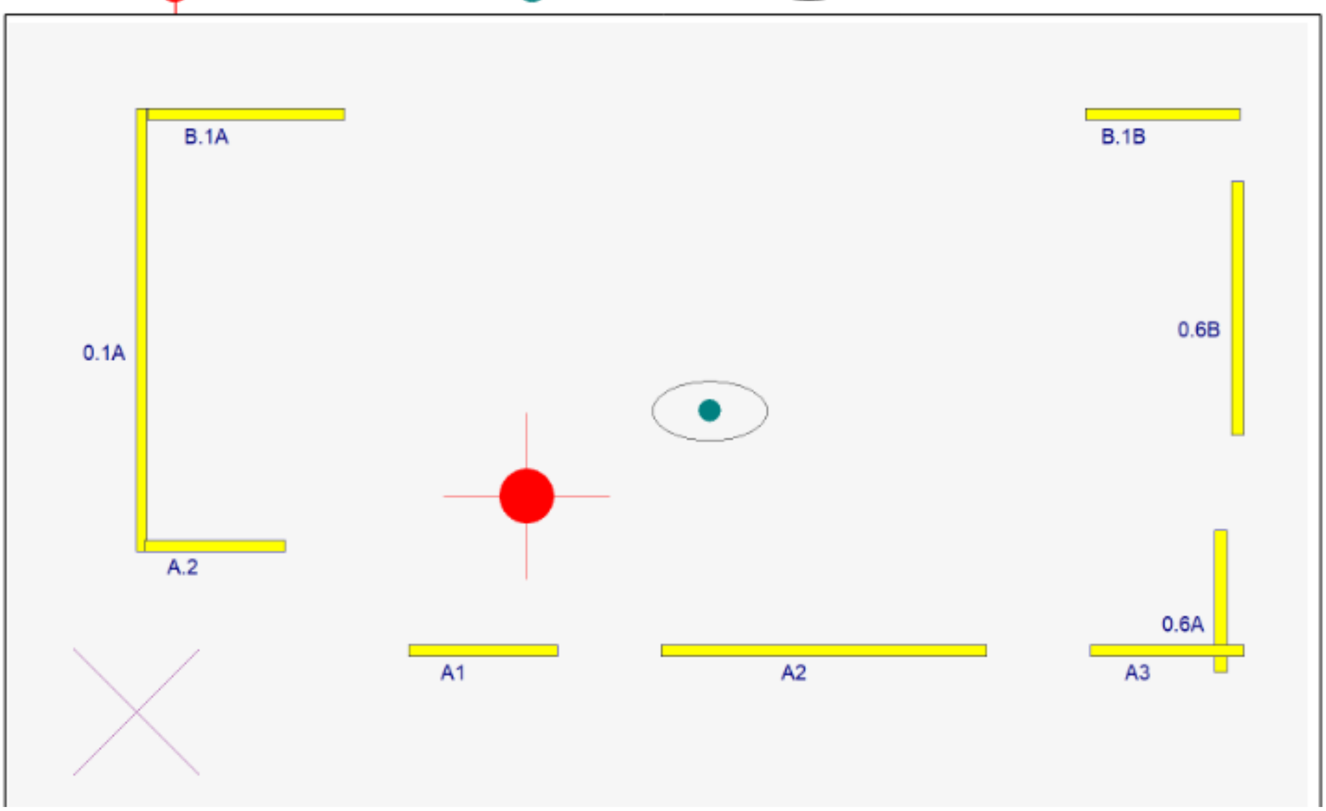
ANALYSIS SUMMARY

Maximum shear forces applied to resisting elements. Eccentricity with respect to Center of Rigidity

	Max Shear along Member Local "y-y" Axis				Max Shear along Member Local "x-x" Axis			
Resisting Element	Load Angle	X-Ecc (ft)	Y-Ecc (ft)	Shear Force (k)	Load Angle	X-Ecc (ft)	Y-Ecc (ft)	Shear Force (k)
0.1A	90	-7.84	5.33	64.743	0	-11.42	7.21	0.000
0.6A	90	-14.99	5.33	11.387	0	-11.42	3.46	0.000
0.6B	90	-14.99	5.33	42.670	0	-11.42	7.21	0.000
A.2	0	-7.84	5.33	7.682	90	-7.84	5.33	0.000
A1	0	-7.84	5.33	8.828	90	-7.84	5.33	0.000
A2	0	-7.84	5.33	46.747	90	-14.99	5.33	0.000
A3	0	-7.84	5.33	9.484	90	-14.99	5.33	0.000
B.1A	0	-11.42	7.21	19.441	90	-7.84	5.33	0.000
B.1B	0	-11.42	7.21	10.999	90	-14.99	5.33	0.000

Layout of Resisting Elements

Legend : Defined Wall X Datum
Center of Rigidity Center of Mass Accidental eccentricity application boundary



Sesimic Forces on North-South Direction Reinforced Concrete Masonry Shear walls

Level 2nd

F_x (k) =	429.96 kips	Area =	2407 (sq-ft)	f_x = 178.63 psf	(ASCE 41-17 BSE-1E, Seismic Story Force)
	178.36 kips		2407 (sq-ft)	74.10 psf	(ASCE 7-16 Seismic Story Force)

Shear wall system = 1 (CMU shear walls take shear loads based on wall lateral stiffness or wall rigidities)

ASCE 7-16 to ASCE 41-17 | 1 | 2.4106 | (Enter this value if using ASCE 7-16 forces otherwise use 1)

Masonry Expected Strength factor = 1.3
Compressive Strength, f'm = 2500 psi
Allowable shear stress, f_{av} = 65 psi (Brick and Masonry walls)
Allowable shear stress, f_{av} = 90 psi (CIP walls) with Expected Strength factor = 1.5

Masonry walls
m_{IO} = 2
m_{LS} = 2
m_{CP} = 3

North-South direction Shear wall loads at 1st Level (2nd to fdn.)

Wall heights = 17.15 ft

			ASCE 41-17 forces												
Wall (grid line)	Wall Trib. Area (ft^2)	Wall Type	Shear Distribution Factor	F_{tot} (lbs)	Shear wall Length, b (ft.)	Shear wall thickness, t (in)	Wall Aspect Ratio	Wall Shear Load, v = $F_{tot}/(1.4*b)$ lbs/ft	Wall Shear Stress, f_v (psi)	DCR (Wall Shear)	Shear Stress Check	Wall Vertical Dowels Capacity (lbs)	DCR Wall Vertical (Dowels)	Slab Dowels Capacity (lbs)	DCR Slab (Dowels)
0.1		CIP	0.64740	278356.40	27.5	8	0.62	7230.04	75.31	1.622	OK	199650	1.394	332750.00	0.837
0.6A		CMU	0.11390	48972.50	8.83	9	1.94	3961.54	36.68	0.790	OK	49681.995	0.986	82803.33	0.591
0.6B		CMU	0.42678	183498.52	15.75	9	1.09	8321.93	77.05	1.660	Not Good	88617.375	2.071	147695.63	1.242
Σ	0.00		1.19	510827.42	52.08										

Sesimic Forces on East-West Direction Reinforced Concrete Masonry Shear walls

Level 2nd

F_x (k) =	429.96 kips	Area =	2407 (sq-ft)	f_x = 178.63 psf	(ASCE 41-17 BSE-1E, Seismic Story Force)
	178.36 kips		2407 (sq-ft)	74.10 psf	(ASCE 7-16 Seismic Story Force)

Shear wall system = 1 (CMU shear walls take shear loads based on wall lateral stiffness or wall rigidities)

ASCE 7-16 to ASCE 41-17 1 2.4106 (Enter this value if using ASCE 7-16 forces otherwise use 1)

Masonry Expected Strength factor =	1.3		Masonry walls	
Compressive Strength, f'm =	2500	psi (Brick and Masonry walls)	m_{IO} =	2
Allowable shear stress, f_{av} =	65	psi	m_{LS} =	2
Allowable shear stress, f_{av} =	90	psi (CIP walls) with Expected Strength factor = 1.5	m_{CP} =	3

East-West direction Shear wall loads at 1st Level (2nd to fdn.) Wall heights = 17.15 ft

Wall (grid line)	Wall Trib. Area (ft^2)	Wall Type	ASCE 41-17 forces: Shear Distribution Factor	ASCE 41-17 forces: F_{tot} (lbs)	Shear wall Length, b (ft.)	Shear wall thickness, t (in)	Wall Aspect Ratio	Wall Shear Load, v = $F_{tot}/(1.4*b)$ lbs/ft	Wall Shear Stress, f_v (psi)	DCR (Wall Shear)	Shear Stress Check	Wall Vertical Dowels Capacity (lbs)	DCR Wall Vertical (Dowels)	Slab Dowels Capacity (lbs)	DCR Slab (Dowels)
A1		CMU	0.0883	37956.91	9.25	8	1.85	2931.04	30.53	0.658	OK	52045.125	0.729	86741.88	0.438
A2		CMU	0.4675	200993.61	20.25	8	0.85	7089.72	73.85	1.591	Not Good	113936.63	1.764	189894.38	1.058
A3		CMU	0.0948	40777.45	9.5	8	1.81	3065.97	31.94	0.688	OK	53451.75	0.763	89086.25	0.458
A.2		CIP	0.0768	33029.56	9.5	8	1.81	2483.43	25.87	0.557	OK	68970	0.479	114950.00	0.287
B.1A		CMU	0.1944	83588.61	12.33	8	1.39	4842.35	50.44	1.086	OK	69374.745	1.205	115624.58	0.723
B.1B		CMU	0.1100	47291.35	9.67	8	1.77	3493.23	36.39	0.784	OK	54408.255	0.869	90680.43	0.522
Σ			1.03	443637.50	70.50										

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