Seismic Evaluation - Tier 1
Cupertino City Hall
10300 Torre Ave
Cupertino, California

Prepared For:
City of Cupertino

Prepared By:
MME Civil + Structural Engineering

MME Job No. 21143.P5

April 19, 2021

April 19, 2022

Susan Michael AIA, Leed AP
Capital Improvement Programs Manager
Public Works
10300 Torre Ave.
Cupertino, California 95014

Re: Cupertino City Hall Seismic Evaluation – Tier 1
MME Project No: 21143.P5

Dear Ms. Michael,

As requested, we have prepared the following building Tier 1 Seismic Evaluation report of the existing Cupertino City Hall located at 10300 Torre Ave., Cupertino, California. Our work includes a seismic evaluation of the existing building based on visual observations of the existing construction and provided documentation. We performed the seismic evaluation under the provisions of the American Society of Civil Engineers (ASCE) 41-17 Standard. We also performed a visual observation of the general condition of the exposed primary structural systems. We have relied solely on existing as-built drawings, technical specifications, and reports provided along with our visual observations of the existing building as the single source of detailed information about the structural components of the building. No removal of finishes or other data collection, such as non-destructive or destructive testing, was provided at this time. Our assessment intends to identify the seismic code conformance of the existing building.

Thank you for the opportunity to assist you with your project. Should you have any questions or comments or require further assistance, please call.

Respectfully yours,

Robert Riley, SE
Senior Structural Engineer

Dale Hendsbee, S.E.
Principal

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Executive Summary

The structural deficiencies noted in this report indicate that the building is likely to sustain major damage and not be functionally operable if a significant seismic event were to occur. If damaged, timely delivery of services to the community that are provided using this building would be impacted. Additionally, occupants of the building (public and staff) are at a higher risk of injury compared against a similar occupancy in a building that did not have these deficiencies.

Based on a review of the existing design and subsequent evaluation reports, the current building is very vulnerable to seismic damage. The original design from 1965 was before vast improvements in the science of earthquake engineering was incorporated into the building codes. The extensive remodel in 1986 failed to bring the building into conformance with the improved seismic codes at that time. The building relies on concrete shear walls for lateral load resistance and a combination of concrete walls and isolated concrete columns to support the gravity loads. These elements do not have sufficient ductility to resist seismic lateral displacements without sustaining significant damage. Damage to these critical structural gravity load-resisting elements could result in collapse of the roof structure. The life safety and economic risk could be substantial.

Two scenarios of seismic strengthening have been discussed for the Cupertino City Hall, located at 10300 Torre Ave, Cupertino, CA. The two scenarios correspond to the building's possible risk category classification according to the California Building Code (CBC) table 1604.5. Scenario one is based on its current occupancy as the Emergency Operation Center (EOC) and is designated an essential facility and therefore classified as risk category IV. Scenario two is a reduced risk category of the building where the EOC would be removed and relocated to a different location. This risk category II is similar to the category that is typically used for offices.

We used the ASCE 41-17 Standard for Seismic Evaluation and Retrofit of Existing Buildings, Tier 1 Evaluation in conjunction with the review of previous reports, original 1965 plans, and retrofit 1986 plans to develop the following structural findings and recommendations for improvement.

For our Tier 1 Evaluation, we have included the heavy clay tile roofing in our calculations for the weight of the building. One area that would help reduce seismic loads and therefore strengthening would be to remove and replace the clay tile with a lighter roofing type.

We found that the building does not comply with either the risk category IV or II evaluation criteria unless a seismic strengthening is undertaken. Our findings are similar to the findings in the previous reports. Based on these findings, we recommend that a Tier 2 Deficiency Based Evaluation be performed to investigate a number of these deficiencies to see if any can be waived and to provide a basis for the detailed design of the remediation work. After completion of the Tier 2 evaluation, any remaining deficiencies identified should be retrofitted. We have separated the structural deficiencies into two groups. Group One are items that in our opinion would not benefit from a Tier 2 evaluation. Group Two are items that may benefit from Tier 2 evaluation.

<u>Structural – Scenario 1 Risk Category IV - Immediate Occupancy</u>
This list is a combination of both our Tier 1 Evaluation and the deficiencies that other reports have identified. The structural deficiencies that have been identified are:

<u>Group One</u> – Unlikely that a Tier 2 evaluation would remove the need to upgrade

1. Roof Diaphragm Shear Capacity
2. Roof Diaphragm Collector Splice Capacity
3. Anchor Bolt Connections at top of Shear Walls
4. Out of Plane Connection of Veranda Beam
5. Upper Floor Concrete Shear Wall Shear Capacity
6. Upper Floor Concrete Shear Wall Flexural Capacity
7. Concrete Shear Wall Boundary Members

<u>Group Two</u> – A Tier 2 evaluation **may** remove the need to upgrade

8. Continuous Cross Ties at Upper Floor Shear Wall
9. Upper Floor Concrete Shear Wall Adjacent to Diaphragm Openings Concrete
10. Ground Floor Wall Reinforcing at Openings
11. Concrete Column Reinforcement for Confinement
12. Concrete Column Splices and Girder Stirrups
13. Wall Foundation Dowels Capacity

<u>Structural – Scenario 2 Risk Category II - Collapse Prevention</u>
This list is only the items that we identified in our Tier 1 Evaluation. It does not include items from previous reports. The reduced amount of deficiencies listed below for risk category II are primarily a reflection of the lower safety standards associated with risk category II and therefore fewer items are required to be checked in the Tier 1 Evaluation. Many of the Scenario 1 items would still be deficient in Scenario 2 if they were required to be checked. The structural deficiencies that have been identified are:

1. Upper Floor Concrete Shear Wall Shear Capacity
2. Out of Plane Connection of Veranda Beam
3. Concrete Column Splices and Girder Stirrups
4. Upper Floor Concrete Shear Wall Adjacent to Diaphragm Openings Concrete
5. Column Reinforcement for Confinement
6. Continuous Cross Ties at Upper Floor Shear Wall

Nonstructural

Nonstructural elements were not included in the scope of our Tier 1 analysis. However, several nonstructural items were noted in the previous reports and are summarized in this report for your consideration (See Appendix G).

A. Equipment anchorage capacities are unknown and would require verification and or installation of anchorage and bracing. Equipment that should be considered includes the following:
   - Emergency Generator, including isolators
   - Emergency Generator flexible connections for conduit, fuel, and coolant piping
   - Rooftop HVAC Equipment
   - Elevator Equipment
   - Electrical Transformers, Panels, Switchgear, Cabinets, etc.
   - Suspended Light Fixtures
   - Ductwork and Piping Supports and Bracing
   - Electrical Conduits, Trapezes, Banks, and Trays
   - Fire Sprinkler Piping
   - Accessibility

B. Anchorage and bracing for the existing suspended ceilings and interior partitions

C. Exterior cladding and glazing system

D. Deteriorated veranda fascia on the south elevation

Seismic strengthening noted in our report is not typically required by the CBC unless certain changes are proposed for the building. These changes include occupancy changes, renovations, additions, and loading changes. Our understanding is that none of these changes is being considered at this time. Barring a City of Cupertino requirement that is more rigorous than the CBC, the proposed strengthening that has been recommended is considered voluntary. Scenario 2 could be a change in occupancy and may trigger these nonstructural improvements.

Geotechnical

No geotechnical report has been provided for our review. Foundation improvements may be required and if this is the case, we recommend obtaining a report by a licensed geotechnical engineer.

For our Tier 1 evaluation, we used the City of Cupertino GIS Property Information web-based application to identify Geologic Hazards. For the City Hall location, there are no mapped Liquefaction, Fault Rupture, or Slope Instability issues at this site (Appendix B).

Introduction

The purpose of this evaluation is to review and evaluate the structural systems of the subject building using criteria provided by ASCE 41-17. Because this building has been structurally evaluated several times in the last 10 years, we were able to use the ASCE 41 evaluation to corroborate previous findings. In areas where the previous evaluations were more in-depth than our evaluation, we have reviewed their findings and included them as part of the recommendations. The ASCE 41 evaluation criteria have been tailored for specific building types and desired levels of building performance. This standard provides a means to identify general deficiencies based on the anticipated behavior of specific building types.

The evaluation begins with a Screening Phase (Tier 1) to assess primary components and connections in the seismic force-resisting system using standard checklists and simplified structural calculations. If the element is compliant, it is anticipated to perform adequately under seismic loading without additional review or strengthening. Items indicated as non-compliant in a Tier 1 checklist are considered potential deficiencies that require further analysis.

A limited, deficiency-based Evaluation Phase (Tier 2) can then be used to review in more detail the items determined to be potential deficiencies by Tier 1 checklists and simplified calculations. Non-compliant items are evaluated for calculated linear seismic demand as determined by ASCE 41-17. If the elements are compliant per Tier 2 analysis, the Tier 1 deficiency is waived. However, if the element remains non-compliant after the more detailed Tier 2 analysis, repair or remediation of the deficiency is recommended.

Evaluation Overview

This seismic evaluation report for the existing building located at 10300 Torre Ave, Cupertino, CA, is based on the following:

- The American Society of Civil Engineers/ Structural Engineering Institute (ASCE/SEI 41-17) Standard for Seismic Evaluation and Retrofit of Existing Buildings - Tier 1, Immediate Occupancy and Collapse Prevention level structural evaluation criteria, including:
    - Checklists
    - Analysis
- One site visit for a general review of the structure was performed on August 08, 2021. No destructive testing or removal of finishes was performed or included in the scope.
- Review of the following original drawings dated October 01, 1965
    - Architectural plans (Partial) prepared by Wilfred E. Blessing F.A.I.A & Associates

  - Structural plans and calculations prepared by Kirk C. McFarland Structural Engineer
- Review of the Civic Center Improvement plans dated December 18, 1986
  - Architectural plans prepared by Holland, East & Duvivier
  - Structural plans prepared by CYGNA Consulting Engineers
- Existing material properties as indicated on sheet S10 of the 1965 structural plans. Properties are included in Appendix C.
- Review of the following reports and evaluations:
  - City Hall Seismic Report" by AKH Structural Engineers, 2006
  - "Report of Results from Structural Analysis and Evaluation of Existing Cupertino City Hall" by AKH Structural Engineers, 2011
  - "Final Cupertino ESF Analysis Rev 1", Multiple Project Participants, 2012
  - "Cupertino City Hall Alternates Study Structural Evaluation" by Tipping Mar, 2014
- No Geotechnical Report was available at the time this report was written. Sheet S10 of the original construction documents indicates that soil design information used in the design is from a soils report.
- Seismic review of non-structural elements is not included as part of our Tier 1 evaluation.

Structure Overview

General Site Description

The building is located on a relatively flat lot on the NW corner of Torre Avenue and Rodrigues Avenue in the City of Cupertino.

Structural Performance Objective

Per ASCE 41-17, a structural performance objective consists of a target performance level for structural elements in combination with a specific seismic hazard level. For the seismic assessment of the subject building, two Basic Performance Objective for Existing Buildings (BPOE) were selected.

Scenario 1:

The City Hall building is currently considered an "Essential Facility" by the City of Cupertino based on upgrades in 1986. This is a Risk Category IV as defined by ASCE 7:

ESSENTIAL FACILITIES: Buildings and other structures that are intended to remain operational in the event of extreme environmental loading from flood, wind, snow, or earthquakes.

For the Tier 1 review to the BPOE, the specified level of performance is Immediate Occupancy (1-B) at the BSE-1E seismic hazard level and Life Safety (3-D) at the BSE-2E seismic hazard level.

The Immediate Occupancy Performance Level as described by ASCE/SEI 41-17 is made up of two parts: the structural performance level and non-structural performance level. The number “1” designates the structural performance level defined as:

> *Structural Performance Level S-1, Immediate Occupancy, is defined as the post-earthquake damage state in which a structure remains safe to occupy and essentially retains its pre-earthquake strength and stiffness.*

The letter designation “B” in the BPOE indicates the nonstructural performance level and is defined as:

> *Position Retention Nonstructural Performance Level (N-B). Nonstructural Performance Level N-B, Position Retention, is the post-earthquake damage state in which nonstructural components might be damaged to the extent that they cannot immediately function but are secured in place so that damage caused by falling, toppling, or breaking of utility connections is avoided. Building access and Life Safety Systems, including doors, stairways, elevators, emergency lighting, fire alarms, and fire suppression systems, generally remain available and operable, provided that power and utility services are available.*

The Life Safety Performance Level as described by ASCE/SEI 41-17 is defined as:

> *Structural Performance Level S-3, Life Safety, is defined as the post-earthquake damage state in which a structure has damaged components but retains a margin of safety against the onset of partial or total collapse.*

The letter designation “D” in the BPOE is defined as:

> *Hazards Reduced Nonstructural Performance Level (N-D). Nonstructural Performance Level N-D, Hazards Reduced, shall be defined as the post-earthquake damage state in which nonstructural components are damaged and could potentially create falling hazards, but high hazard nonstructural components identified in Chapter 13, Table 13-1, are secured to prevent falling into areas of public assembly or those falling hazards from those components could pose a risk to life safety for many people. Preservation of egress, protection of fire suppression systems, and similar life-safety issues are not addressed in this Nonstructural Performance Level.*

Scenario 2:

To reduce the amount of strengthening required the City Hall building could be converted back to an occupancy that is typical for an office building. The primary function that would have to be removed is the EOC. The building could be considered a Risk Category II as defined by ASCE 7:

All buildings and other structures except those listed in Risk Categories I, III, and IV.

For the Tier 1 review to the BPOE, the specified level of performance is Collapse Prevention (5-D) at the BSE-2E seismic hazard level. ASCE/SEI 41-17 defines Collapse Prevention as:

Structural Performance Level S-5, Collapse Prevention, is defined as the post-earthquake damage state in which a structure has damaged components and continues to support gravity loads but retains no margin against collapse.

The letter designation "D" in the BPOE is defined above in Scenario 1

A Tier 1 evaluation of nonstructural elements was not included within the scope of this review.

<u>Site Seismicity</u>

Per ASCE 41-17, 'seismicity', or the potential for ground motion, is classified into regions defined as Low, Moderate, or High. These regions are based upon mapped site accelerations Ss and S1 which are then modified by site coefficients Fa and Fv to produce the Design Spectral Accelerations, SDS (short period), and SD1 (1-second period).

At the time of this report, no geotechnical investigation or report has been provided for the subject site. The soil profile of this building is therefore assumed the default and classified as Site Class D per ASCE 41-17 for use in the determination of site coefficients Fa and Fv.

Per the site values indicated by USGS data and evaluated using seismic acceleration equations and tables of ASCE 41-17, the site is located in a region of High Seismicity with a design short-period spectral response acceleration parameter (SDS) of 1.589g and a design spectral response acceleration parameter at a one-second period (SD1) of 0.623g. See Summary Data Sheet in Appendix D.

The spectral response parameters SS and S1 were obtained for the BSE-1E seismic hazard level for existing structures (BPOE). The acceleration values were adjusted for the maximum direction and site class per ASCE 41-17 Section 2.4.1, and compared to BSE-1N (used by current building code for design of new buildings) to determine the design values for the Tier 1 analysis, since values obtained for the BSE-1E hazard level need not exceed the hazard levels for new construction.

The successful performance of buildings in areas of high seismicity depends on a combination of strength, ductility of structural components, and the presence of a fully interconnected, balanced, and complete seismic force-resisting system.

General

Original 1965 Construction: The original building was a one-story structure above grade with a basement below grade. A 1985 remodel opened one side of the basement, introduced openings in the north basement wall, and created an elevated veranda slab on the north side of the building (Photo 1). These changes created a 2 story building. The building is generally rectangular in plan, with the long side oriented in the east-west direction. The building footprint including the roofed veranda is approximately 136 feet by 112 feet. The interior space is 120 feet by 96 feet and the two floors have a combined area of approximately 23,040 square feet.

The 1st floor is a reinforced elevated concrete slab, supported by concrete joists, beams, and columns. The structural floor from the 1965 drawings is shown in Figure 1. A Structural floor-framing plan of the 1st floor remodel from the 1986 plans is shown in Figure 2.

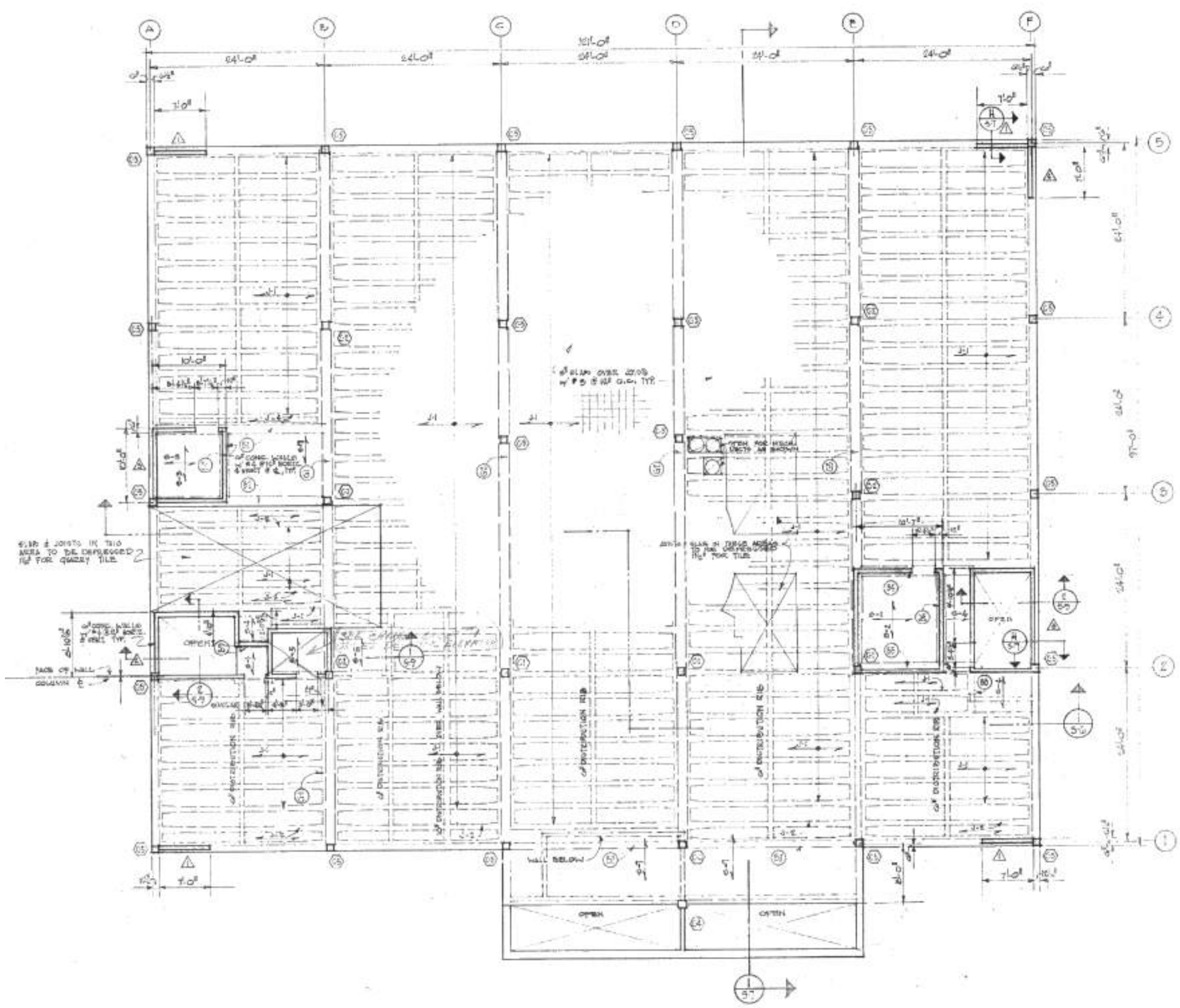


Figure 1 1st Floor Framing Plan, 1965 Structural Drawings

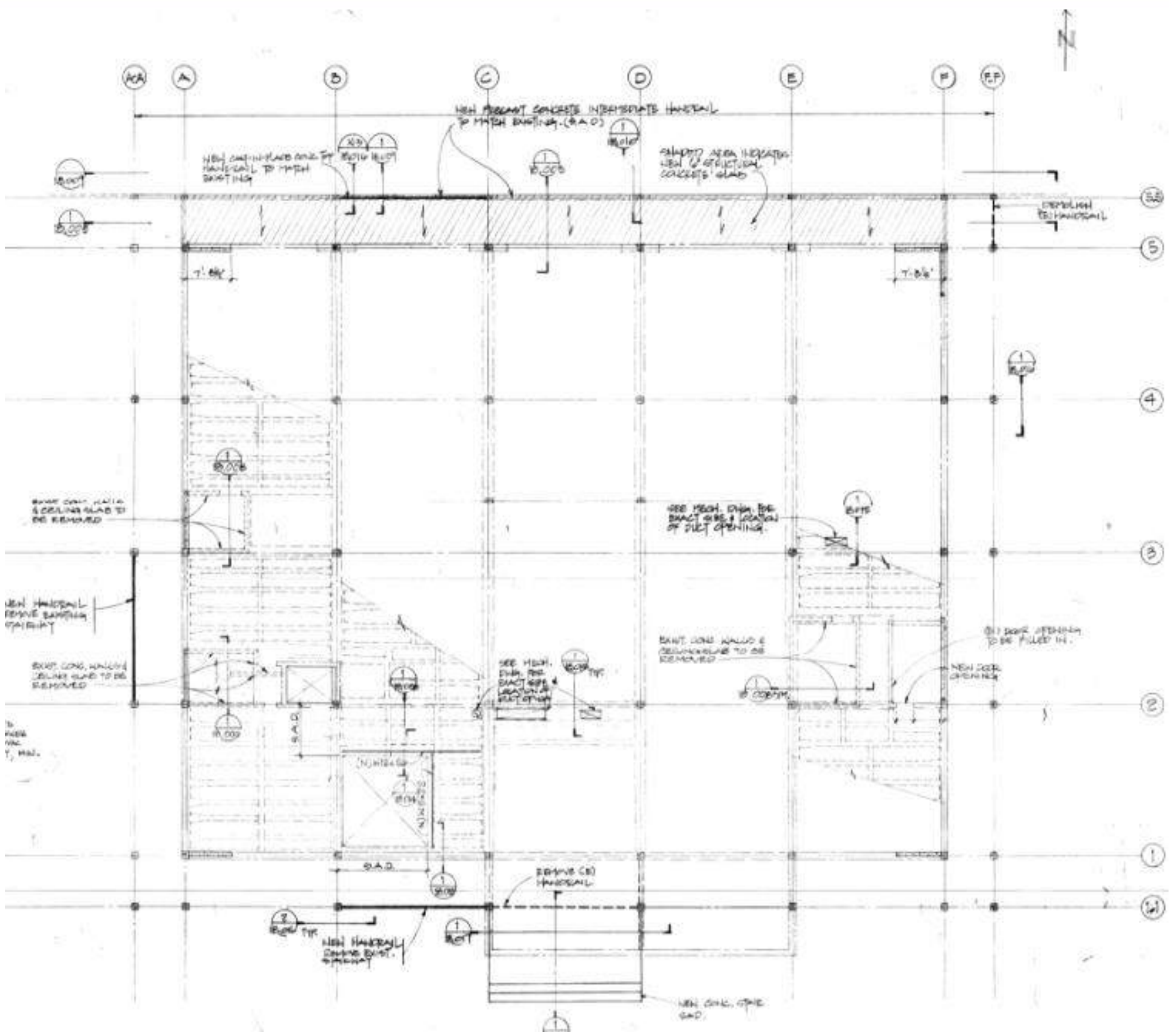


Figure 2 1st Floor Framing Plan, 1986 Structural Drawings

The roof is a mansard type with the lower hip portion having two slopes and the center portion being essentially flat. The hipped lower portion is framed with wood girders at 6’ on center, T&G decking overlaid with ½” plywood. The upper flat portion has rafters at 16” on center typical and sheathed with ½” plywood. Rafters and girders are supported by bearing walls, steel beams, or concrete beams. (Figure 3). The sloping roof and mansard are clay tile.

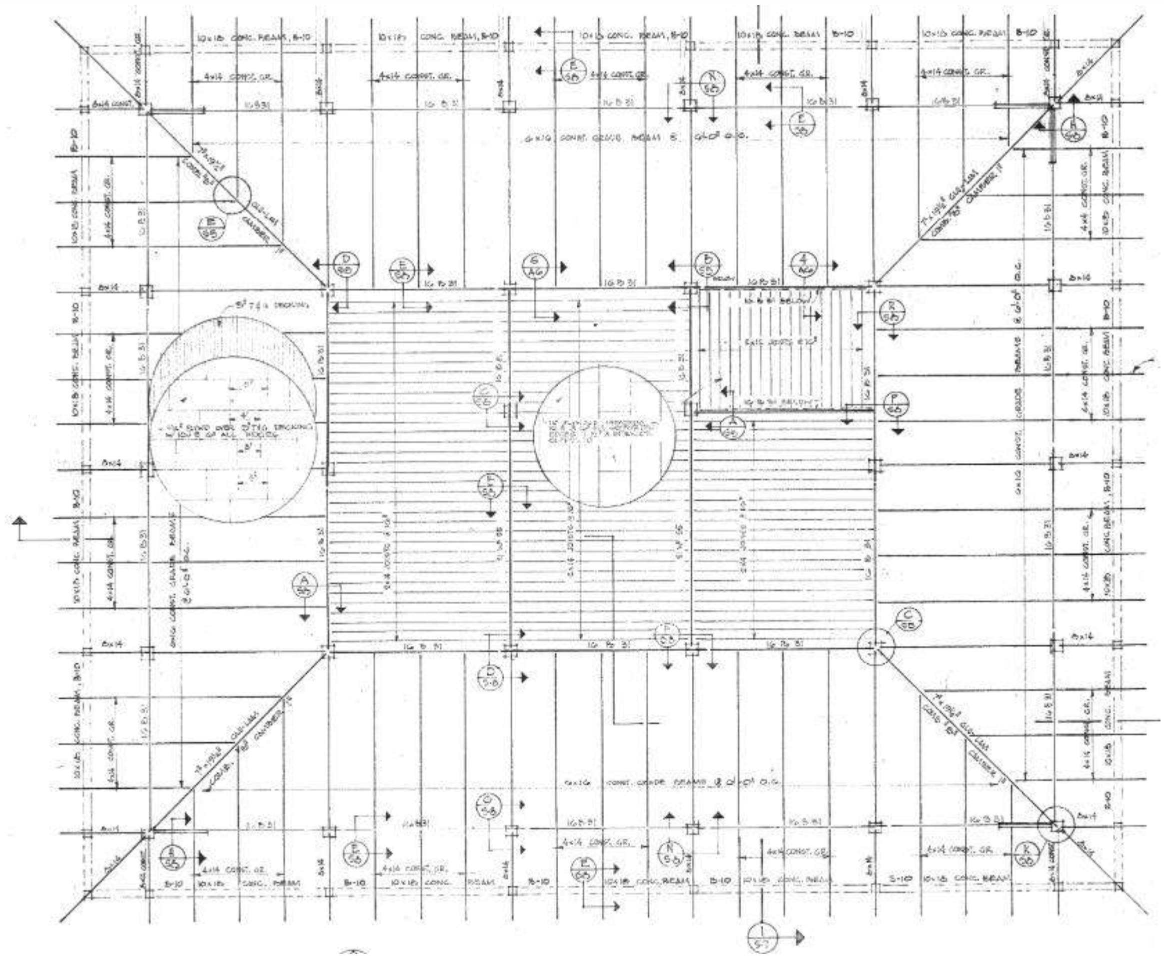


Figure 3 Roof Framing Plan, 1965 Structural Drawings

A full building section from the 1965 drawings is shown in Figure 4.

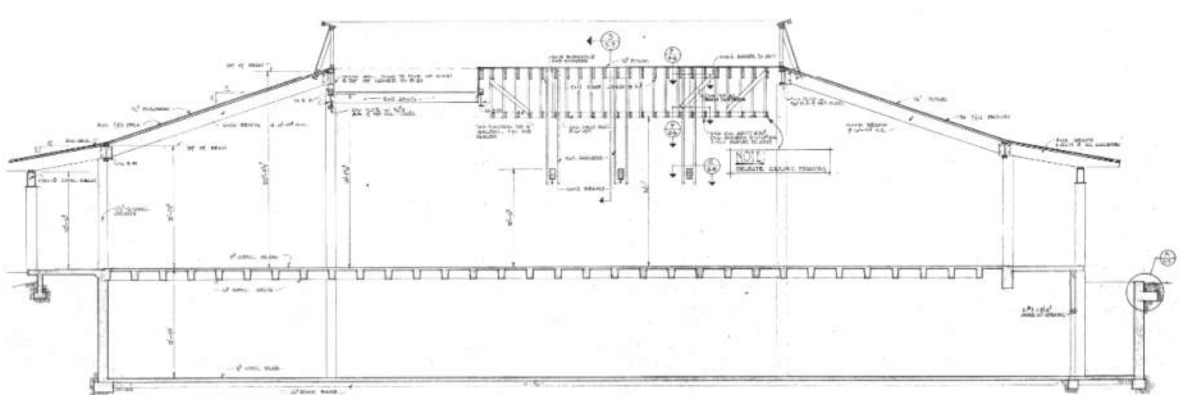


Figure 4 - Full Building Longitudinal Section from 1965 Structural Drawings

Walls

Ground floor/basement walls are reinforced concrete. Walls above the 1st floor elevated slab consist of relatively short shear concrete walls with wood-framed infill walls between the shear walls. Columns supporting beams are typically 12” square reinforced concrete.

<u>Seismic Force-Resisting System</u>

The lateral system of the building is reinforced concrete shear walls. The below-grade perimeter walls in the original plans were 12” thick with a single layer of vertical #6s at 12” and horizontal #5s at 10”. The 1986 remodel opened up the northern perimeter basement wall and added reinforcing and 6” to the thickness of the walls (Figure 5).

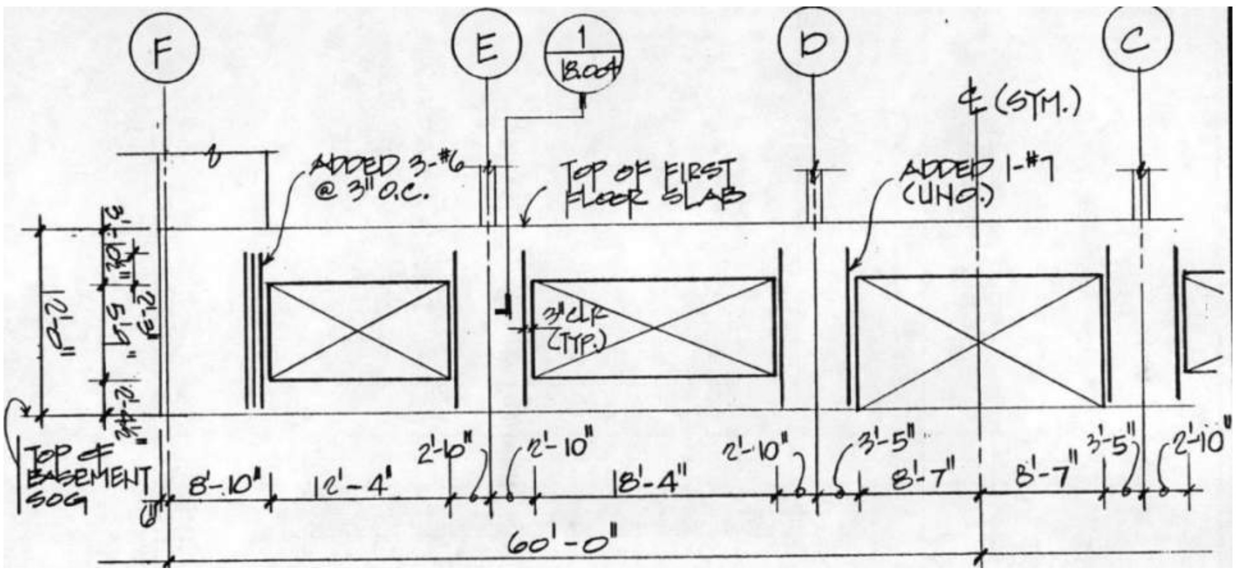


Figure 5 North Wall Elevation 1986 Structural Drawings

The first-floor shear walls are 6” thick reinforced concrete walls and are shown in red in Figure 6 from the 1965 1st Floor Framing Plan. The walls reinforcing and the top of wall anchor bolts are specified in the table shown in Figure 7.

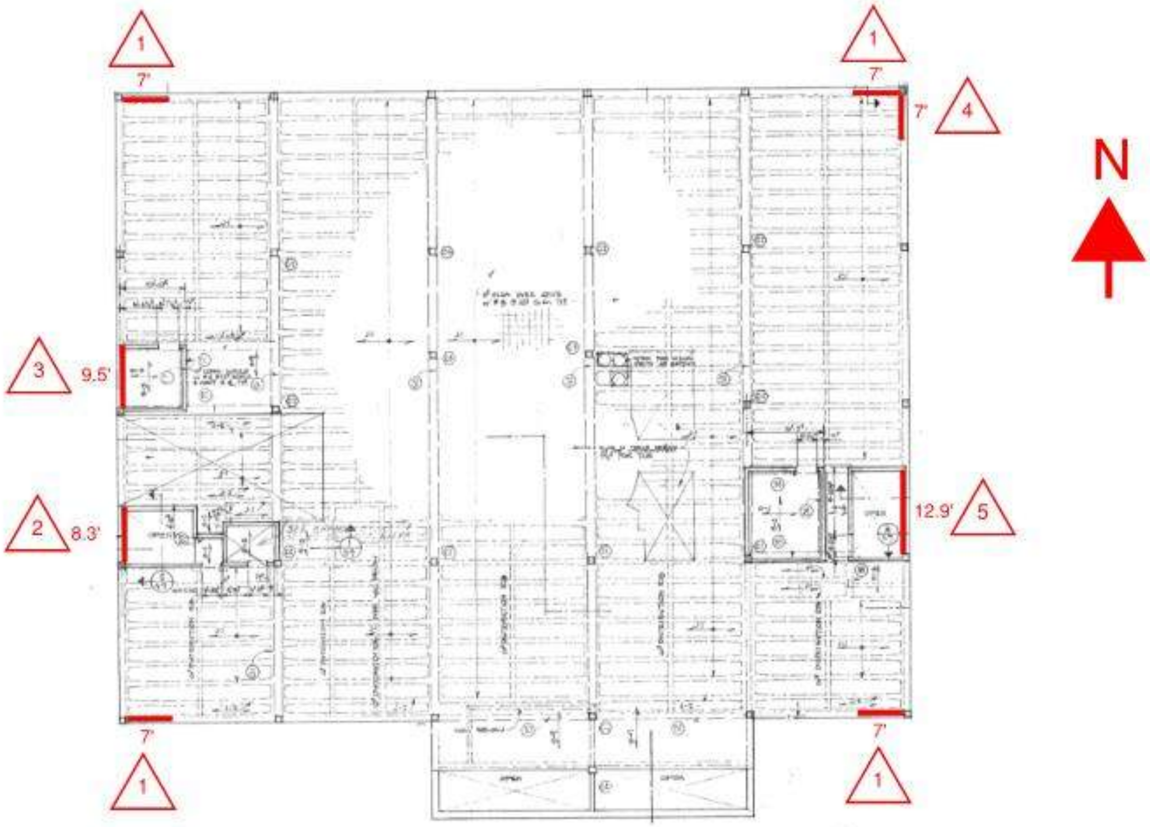


Figure 6 Shear walls from 1965 Structural Plans

SHEAR WALL SCHEDULE

MARK	WALL THICKNESS	A.B. & B.S. TO WALL		REBAR EA. END OF WALL		REMARKS
		AMOUNT	SIZE	AMOUNT	SIZE	
1	6"	7	7/8"φx12"	2	#8	TYP. WALL STEEL #4 @ 12" EA. WAY
2	6"	6	7/8"φx12"	1	#9	DO.
3	6"	8	7/8"φx12"	1	#10	DO.
4	6"	4	7/8"φx12"	1	#9	DO.
5	6"	10	7/8"φx12"	2	#8	DO.

NOTE:
REBAR @ EACH END OF WALL SHALL BE FULL HEIGHT w/ NO SPLICE @ TOP OF FOUNDATION. PROVIDE #5 DOWELS @ 6" FROM FOUNDATION FOR ENTIRE LENGTH OF WALL.

Figure 7 Shear wall Schedule from 1965 Structural Plans

Foundations

Foundations are generally shallow spread reinforced concrete interior columns and continuous concrete footings at the perimeter. A slab on grade is present over the entire footprint of the building.

Field Verification and Condition Assessment

A visual assessment was performed on August 08, 2021, by MME. The exterior and interior of the structure were observed; the interior review included a walkthrough of the ground and 1st floor.

The structure appeared to be in generally good structural condition with minimal structural damage or deterioration apparent (except as noted below) and appears to be constructed in general accordance with the provided structural drawings.

The veranda fascia on the south elevation has significant wood deterioration, Photo 4. The extent of the deterioration and if it affects the structural members should be investigated during the Tier 2 evaluation.

The veranda slab at the southwest corner has a significant crack and spalling adjacent to the building corner column, Photo 5. The most likely reason is the differential settlement between the building and the slab on grade.

Material Properties

Basic properties for existing structural materials were found on the existing building documentation or per ASCE 41 code prescribed minimum structural values utilized in the analysis calculations can be found in Appendix C.

Building Type

Per ASCE/SEI 41-17, this building can be classified as Building Type C2: Concrete Shear Walls with Stiff Diaphragms and C2a: Concrete Shear Walls with Flexible Diaphragms. There are no interior structural walls, but there are interior concrete columns on a grid pattern supporting the 1st floor and roof. The floor is a concrete slab supported on concrete joists and is classified as a stiff diaphragm. The roof framing consists of plywood sheathing over wood joists, girders, steel beams, and concrete columns. The plywood-sheathed diaphragm is classified as flexible. The foundation system consists of continuous perimeter footings and isolated interior footings. Seismic forces are resisted by concrete and wood diaphragms, and exterior concrete walls.

Historical Performance

In addition to classifying buildings by type of construction, ASCE 41 identifies 'Benchmark Buildings' for each building type. The detailing of seismic force-resisting systems in Benchmark Buildings is generally considered to meet the performance requirements of ASCE 41. A building can be determined to be compliant with the

Benchmark Building requirements after a thorough review of the existing building plans, field verification of construction, and a condition assessment. The evaluation of non-structural elements is still required.

For building types C2 and C2a evaluated to the Immediate Occupancy Structural and Life Safety Performance, the benchmark building code year is 2000 and 1994 respectively. Since the subject building was constructed in 1965 and remodeled in 1986, it does not meet the criteria of a Benchmark Building, and a Tier 1 analysis is required.

Findings and Recommendations

Structural

We performed the ASCE 41-17 Tier 1 Building Type Specific Checklists (Appendix D) based on two scenarios for the two different occupancies: scenario 1 - occupancy category IV and scenario 2 – occupancy category II. We found thirteen (13) and five (5) non-compliant items respectively. We have also included several non-structural non-compliant items either that were noted in previous reports or that we identified during our site visit. See Appendix D and E for retrofit details.

We have separated the structural deficiencies into two groups. The first group are items that in our opinion a Tier 2 evaluation **would not** alleviate the need for the seismic upgrade. The second group may benefit from additional analysis included in a Tier 2 evaluation.

Group One – Unlikely that a Tier 2 evaluation would remove the need to upgrade

1. Roof Diaphragm Shear Capacity: The AKH evaluation determined that the shear capacity of the roof diaphragm was over-stressed. They determined that even if the clay tile roof was removed and replaced with a lighter roofing system, the plywood nailing would need to be upgraded.
   Required for Scenario 1 – occupancy category IV
   Recommendation: The plywood nailing should be upgraded.
2. Roof Diaphragm Collector Splice Capacity: The AKH evaluation determined that the collector splices are over-stressed.
   Required for Scenario 1 – occupancy category IV
   Recommendation: The splice connections should be upgraded.
3. Anchor Bolt Connections at top of Shear Walls: The AKH evaluation and our Tier 1 quick checks determined that the anchor bolts are overstressed.
   Required for Scenario 1 – occupancy category IV
   Recommendation: The anchor bolt connections should be upgraded.

4. Out of Plane Connection of Veranda Beam: The Tier 1 evaluation determined that the connection from the veranda beam to the roof framing is inadequate for out-of-plane loads.
   Required for Scenario 1 – occupancy category IV and Scenario 2 – occupancy category II
   Recommendation: The out of plane connection at the veranda should be upgraded.
5. Upper Floor Concrete Shear Wall Shear Capacity: The Tier 1 evaluation determined that the existing shear walls are over-stressed. In addition, the AKH calculations, as well as the Tipping Mar calculations have shown that the shear walls will require additional capacity.
   Required for Scenario 1 – occupancy category IV and Scenario 2 – occupancy category II
   Recommendation: The shear walls should be upgraded. Upgrades to repair Items 6 through 10 in regards to shear wall retrofits can all be achieved at the same time.
6. Upper Floor Concrete Shear Wall Flexural Capacity: See #6 above
   Required for Scenario 1 – occupancy category IV
7. Concrete Shear Wall Boundary Members: See #6 above
   Required for Scenario 1 – occupancy category IV

<u>Group Two</u> – A Tier 2 evaluation **may** remove the need to upgrade

8. Continuous Cross Ties at Upper Floor Shear Wall: Continuous cross ties do not exist at locations of the upper floor shear walls.
   Required for Scenario 1 – occupancy category IV and Scenario 2 – occupancy category II
   Recommendation: A Tier 2 evaluation may determine that continuous cross ties for the full length of the building are not required.
9. Upper Floor Concrete Shear Wall adjacent to diaphragm openings: Several of the shear walls on the East and West elevations are adjacent to openings in the concrete floor diaphragm.
   Required for Scenario 1 – occupancy category IV and Scenario 2 – occupancy category II
   Recommendation: A Tier 2 evaluation may show that the current geometry is adequate and this does not need to be repaired.
10. Ground floor Wall Reinforcing at Openings: The 1986 remodel that created the openings in the lower level north wall placed additional vertical reinforcement at the openings but did not include horizontal reinforcement.
    Required for Scenario 1 – occupancy category IV
    Recommendation: A Tier 2 evaluation may provide relief from this requirement.

11. Concrete Column Reinforcement for Confinement: The Tier 1 evaluation and previous studies determined that there are not adequate column confinement ties around the longitudinal vertical bars.
    Required for Scenario 1 – occupancy category IV
    Recommendation: A Tier 2 evaluation may reduce some of the need for additional confinement. It is anticipated that some of the columns will still require modification to meet code requirements.
12. Concrete Column Splices and Girder Stirrups: The Tier 1 evaluation determined that the existing longitudinal bar splice lengths and the spacing of stirrups in the concrete beams at the floor level are inadequate.
    Required for Scenario 1 – occupancy category IV and Scenario 2 – occupancy category II
    Recommendation: A Tier 2 evaluation may reduce some of the need for these repairs
13. Wall Foundation Dowels: The Tier 1 evaluation identified that there are dowels into the foundation at the concrete walls. However, the capacity of the dowels needs to be verified.
    Required for Scenario 1 – occupancy category IV
    Recommendation: A Tier 2 evaluation may show that the dowels are adequate.

<u>Non-Structural</u>
We did not complete a Tier 1 evaluation of non-structural elements such as mechanical, electrical, and plumbing (MEP) anchorage and bracing. The previous reports have evaluated these items and have made recommendations for the seismic upgrade.

A. Equipment anchorage capacities are unknown and would require verification and or installation of anchorage and bracing. Equipment that should be considered includes the following:
    - Emergency Generator, including isolators
    - Emergency Generator flexible connections for conduit, fuel, and coolant piping
    - Rooftop HVAC Equipment
    - Elevator Equipment
    - Electrical Transformers, Panels, Switchgear, Cabinets, etc.
    - Suspended Light Fixtures
    - Ductwork and Piping Supports and Bracing
    - Electrical Conduits, Trapezes, Banks, and Trays
    - Fire Sprinkler Piping
B. Anchorage and bracing for the existing suspended ceilings and interior partitions

C. Exterior cladding and glazing system
D. Deteriorated veranda fascia on the south elevation
E. Accessibility

For our Tier 1 Evaluation, we have included the heavy clay tile roofing in our calculations for the weight of the building. One area that would help reduce seismic loads and therefore strengthening would be to remove and replace the clay tile with a lighter roofing type.

Reliability of Seismic Evaluations

In general, structural engineers cannot predict the exact damage to a building as a result of an earthquake. There will be a wide variation of damage from building to building due to the variations in ground motion and varying types and quality of construction. In addition, engineers cannot predict the exact ground motions of the earthquake that may strike a given building. Design and evaluation of buildings are performed using general guidelines and information from past earthquakes. Engineers and the codes used for design and evaluation have been conservative when attempting to ensure that building design meets minimum standards of Immediate Occupancy. This effort is based on science and technology as well as on observations made from actual seismic events. Building design and codes are constantly evolving to better meet performance targets. Continued research will improve predictive methods and facilitate performance-based engineering. It has been estimated that, given design ground motions, a small percent of new buildings and a slightly greater percent of retrofit buildings may fail to meet their expected performance.

This report is general and does not imply that the recommendations listed above are the only structural requirements that must be made to the existing structure to meet current code criteria.

We understand you may have questions regarding this evaluation and are available for comment and explanations. Please call with any questions you may have. Thank you for choosing MME Structural Engineers to assist you with this building seismic review.

APPENDIX A – Photographs

Photo 1 North Elevation with Elevated Veranda Slab

Photo e East Elevation

Photo 2 Veranda Concrete Beam

Photo 3 Damaged Veranda Fascia

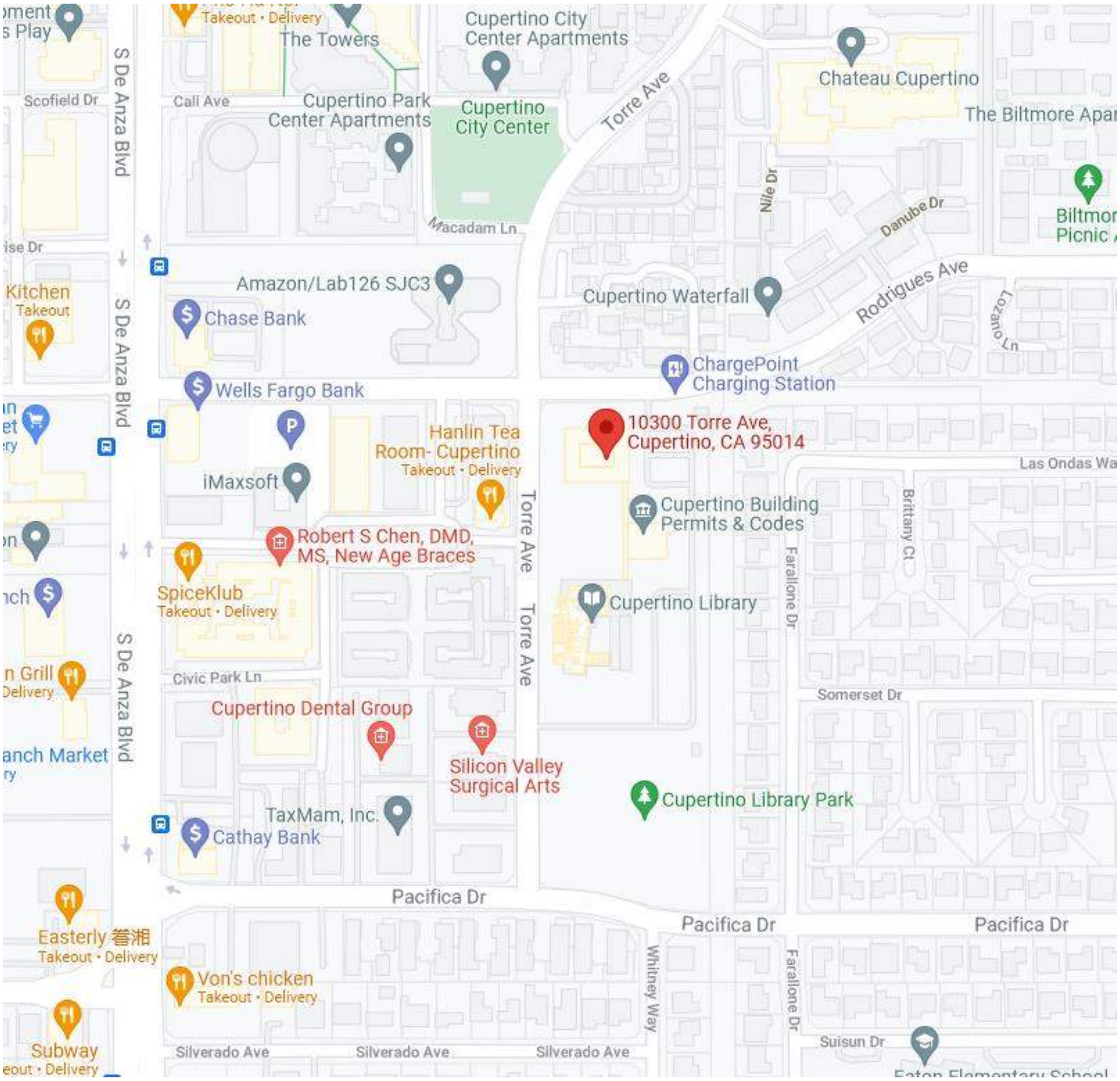
Photo 4 Veranda

Photo 5 Veranda

Photo 6 Veranda Damaged Slab on Grade

APPENDIX B – Maps

Location Map



Map 1 Location Map

Geologic Hazard Map

Per the Cupertino GIS Property Information Map, shown below, the subject site is not in a Fault Rupture or Liquefaction-Inundation Zone.

Map 2 Cupertino GIS Map W/ Geologic Hazards

Since no geotechnical report is available, the default class D soil type has been assumed for this investigation.

APPENDIX C – Materials

<u>FOUNDATIONS</u>: The bottom of all footings shall bear on native undisturbed material at least 12" below the present grade or 18" below the rough finish grade, whichever is lower. If excess excavation is made beneath the footings, the excess excavation shall be filled with concrete of the specified mix. The design soil pressure is ——psf dead load, —— psf dead and live load, and —— psf for all loads including wind and seismic. —see soil report

<u>BACKFILL</u>: Prior to backfilling, concrete forms shall have been stripped and together with all debris shall have been removed from the area. Material used in backfilling shall be free of wood scraps, rubbish, debris or rubble.

<u>CONCRETE</u>: All foundation concrete shall have an ultimate compressive strength of not less than 2,500 psi at 28 days and shall contain not more than 6.75 gallons of water for each 94 pound sack of cement. All concrete for columns, beams, girders, slabs above grade, stairs, etc. shall have an ultimate compressive strength of not less than 3,000 psi at 28 days and shall contain not more than 6.00 gallons of water for each 94 pound sack of cement.

The minimum clear distance from the reinforcing steel to the face of the concrete shall be:

- 3" where concrete is placed against earth
- 2" where concrete is exposed to earth but placed in forms
- 2" where concrete is exposed to weather
- 1½" for beams, girders, and columns
- ¾" for slabs and walls

<u>REINFORCING STEEL</u>: All reinforcing steel shall be deformed intermediate Grade Billet Steel in conformance with ASTM Designations A 15 and A 305. Splices shall be lapped not less than 40 diameters and laps in adjacent bars shall be staggered where practical.

<u>STRUCTURAL STEEL</u>: All structural steel shall be fabricated and erected in conformance with the American Institute of Steel Construction Specification for the Design, Fabrication and Erection of Structural Steel for Buildings.

All Structural steel shall be shop and field painted as described in the specifications. After erection all abraided or burned spots shall be retouched.

<u>CARPENTRY</u>: All framing lumber except sills shall be Coast Region Douglas Fir. Sills shall be Redwood and shall be the full width of the stud. Sills (unless otherwise noted) shall be anchored to the foundation with 5/8" × 12" bolts spaced not more than 4'-0" with one bolt not more than 9" nor less than 4" from each end of each piece of sill. Where sills are bored or notched exceeding one-third of the sill width, extra bolts shall be placed each side of the hole or notch as per ends of pieces. There shall be not less than two bolts in each piece of sill. Sills for structural walls shall be bedded in 1:2 cement mortar not less than one-half inch thick.

Photo 7 Material Properties for 1965 Structural Plans

APPENDIX D - AKH Details

Retrofit Details From "Cupertino City Hall Essential Services Facility Analysis Appendix" by AKH

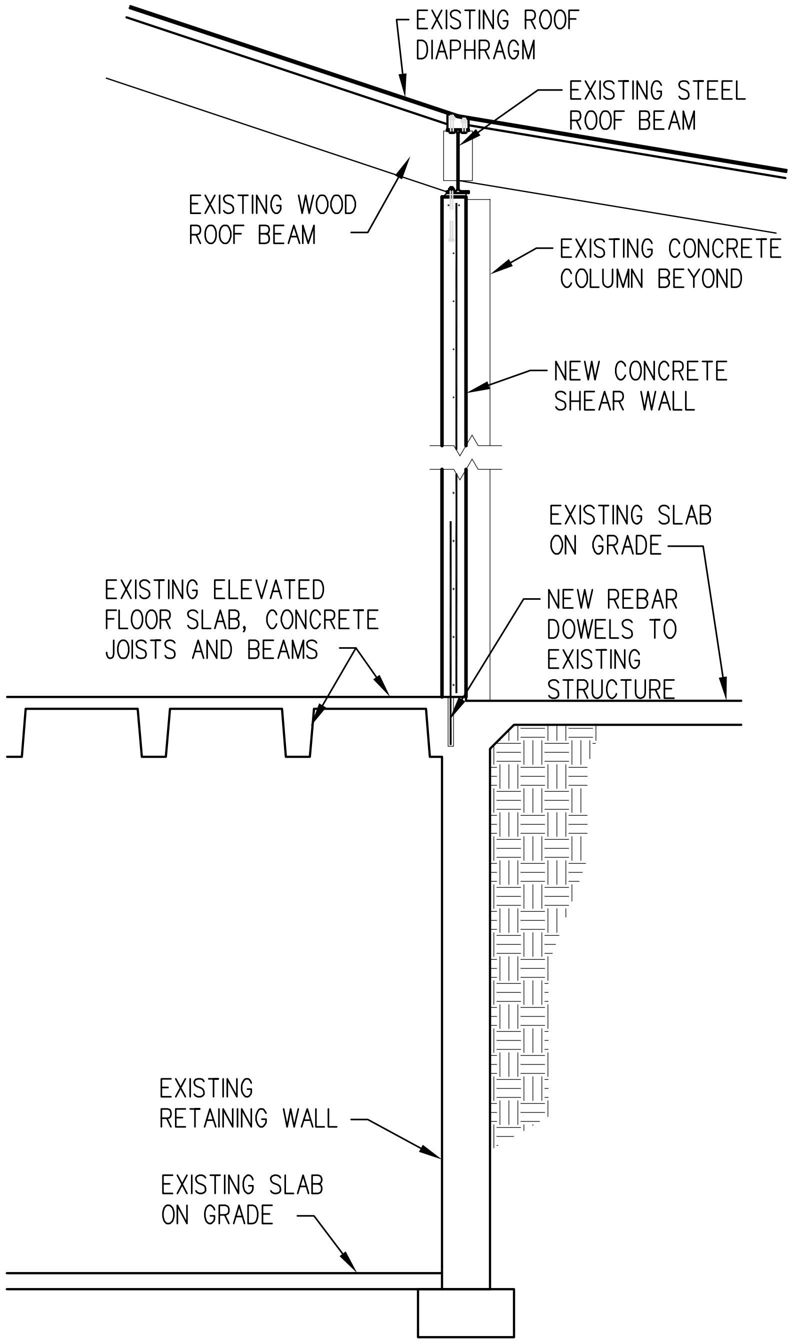


Figure 3.C: SECTION AT NEW SHEAR WALL

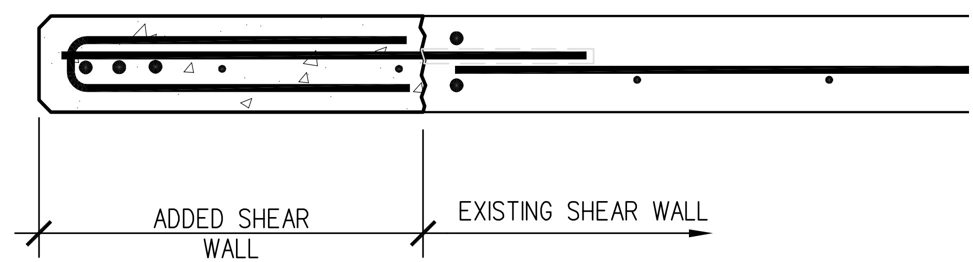


Figure 3.D: NEW SHEAR WALL AT EXIST. WALL

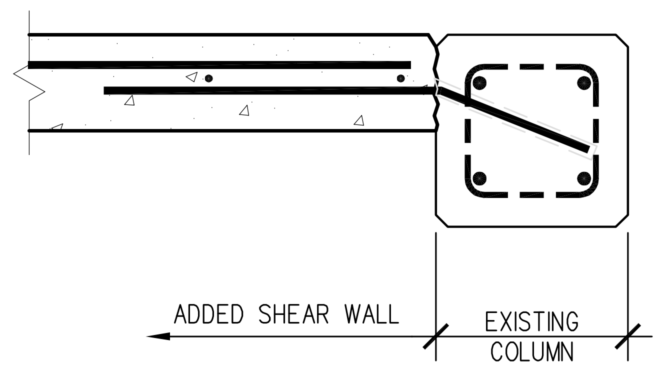


Figure 3.E: NEW SHEAR WALL AT EXIST. COL.

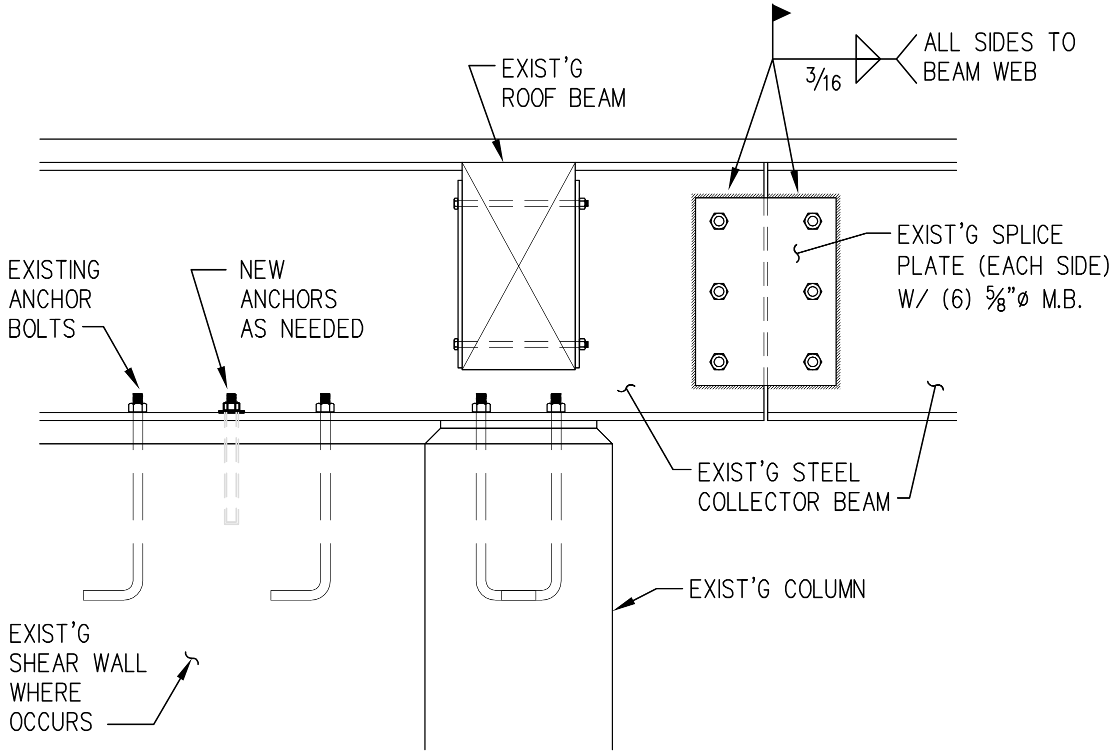


Figure 3.F: ELEVATION: EXISTING STEEL BEAM AT COLUMN AND SHEAR WALL

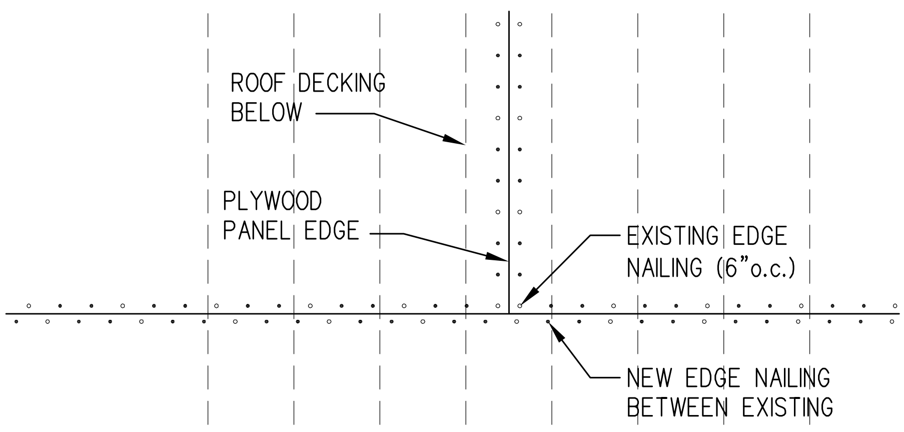
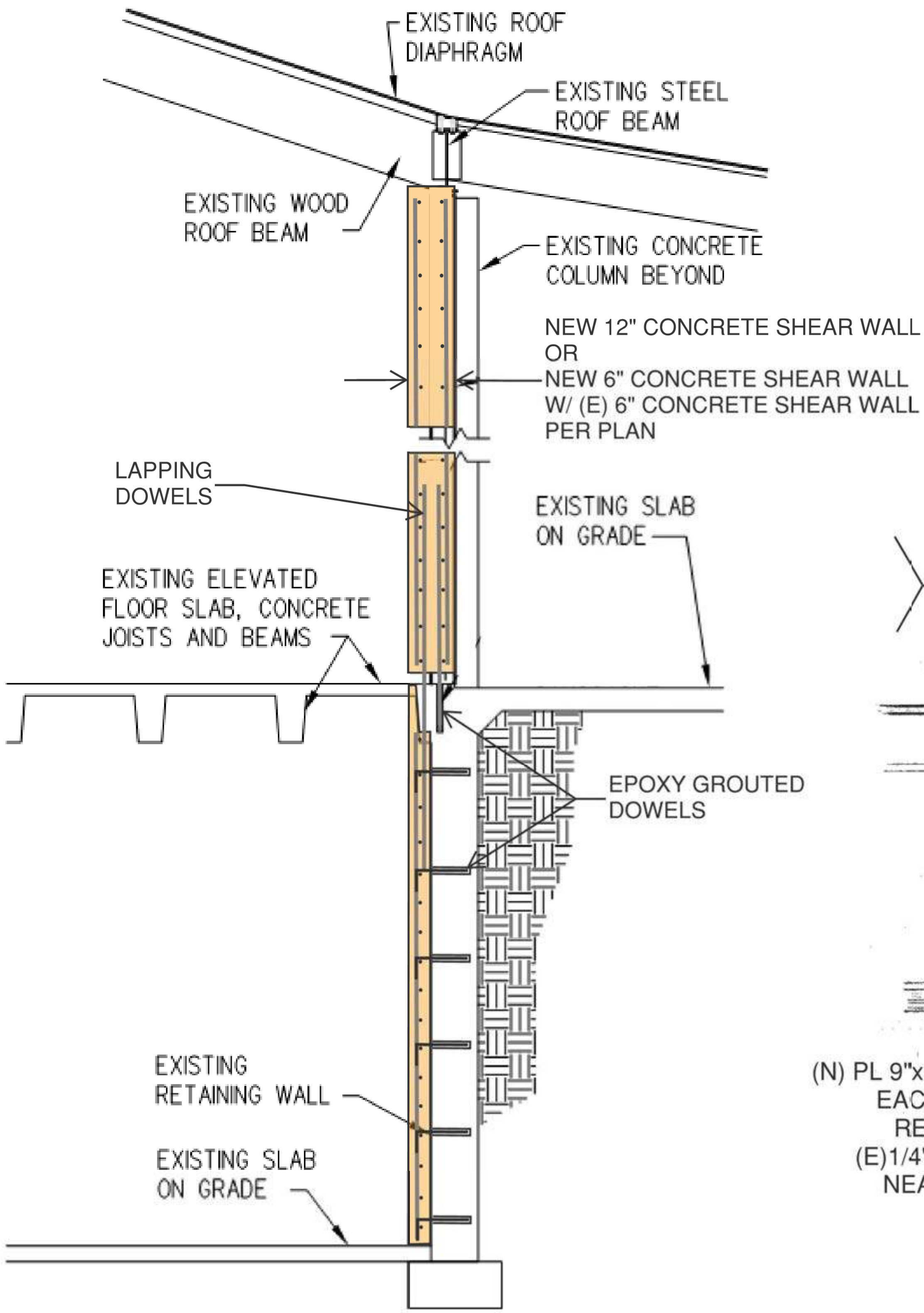


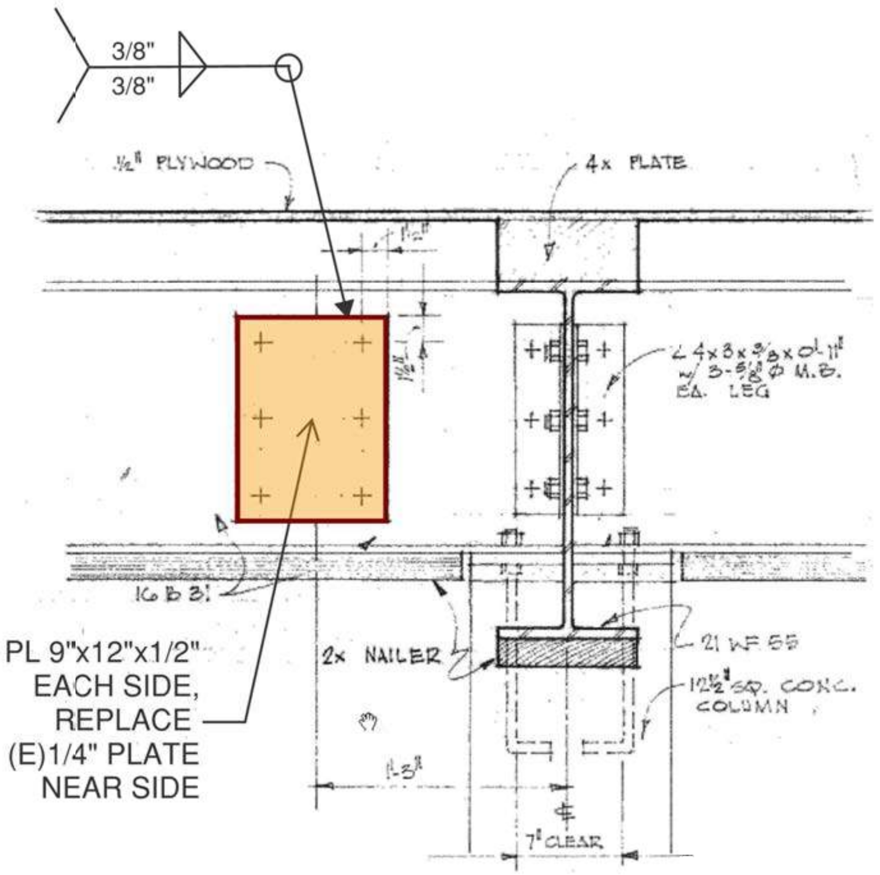
Figure 3.G: PLAN OF PLYWOOD PANEL EDGE NAILING

APPENDIX E - Tipping Mar Details

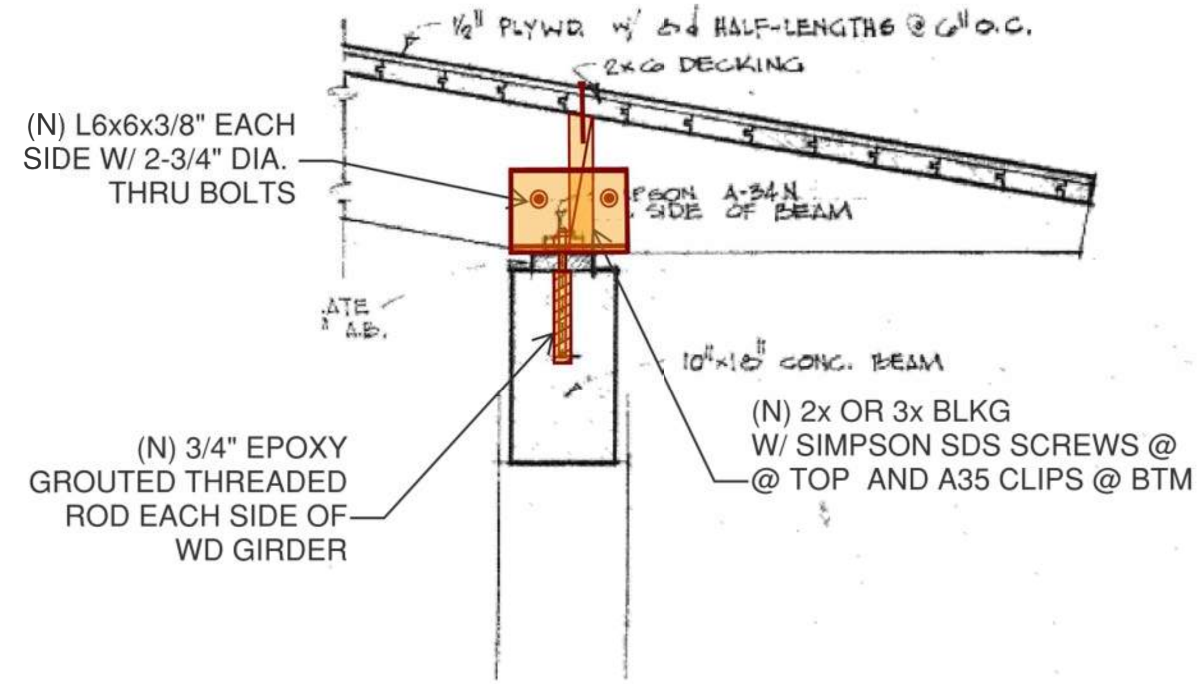
Retrofit Details From "Cupertino City Hall Essential Services Facility Analysis Appendix 11" by Tipping Mar



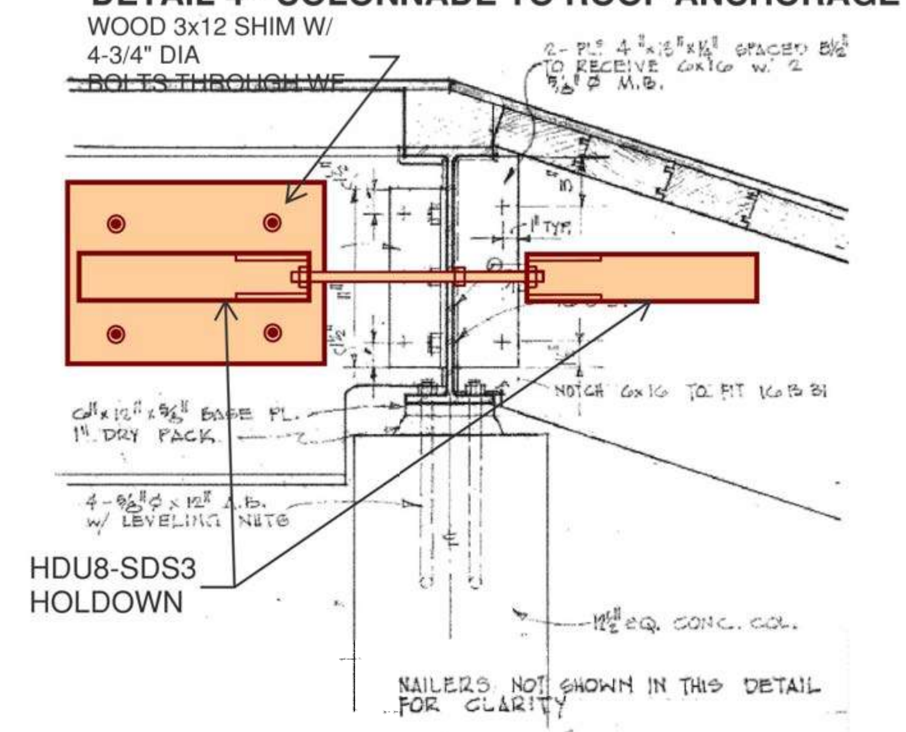
DETAIL 3 - SECTION AT NEW CONCRETE SHEAR WALL



DETAIL 2 - STEEL TO STEEL COLLECTOR SPLICE



DETAIL 4 - COLONNADE TO ROOF ANCHORAGE



DETAIL 1 - STEEL TO WOOD COLLECTOR CONN.

TIPPING | MAR
STRUCTURAL ENGINEERING
1906 Shattuck Ave. Berkeley, CA 94704
510 549-1906 510 549-1912 fax

Cupertino City Hall | Option B
Cupertino, CA

Retrofit Details
September 29 2014

TM Project: 2014,094 Scale: As Noted

S3

APPENDIX F – Tier 1 Checklists

Appendix C: Summary Data Sheet

BUILDING DATA

Building Name: Cupertino City Hall — Date: 10/12/2021
Building Address: 10300 Torre Ave, Cupertino, CA 95014
Latitude: 37.3188973 — Longitude: -122.0286498 — By:

Year Built: 1965 — Year(s) Remodeled: 19 — Original Design Code:
Area [ft² (m²)]: 23,040 — Length [ft (m)]: 120 — Width [ft (m)]: 96
No. of Stories: 2 — Story Height: 12'/17.1' — Total Height: 29.1

USE ☐ Industrial ☑ Office ☐ Warehouse ☐ Hospital ☐ Residential ☐ Educational ☐ Other: ______

CONSTRUCTION DATA

Gravity Load Structural System: Roof - Wood Framed & Floor - Concrete Slab and joists
Exterior Transverse Walls: Concrete Shearwalls — Openings? Not in shearwalls
Exterior Longitudinal Walls: Concrete Shearwalls — Openings? Not in Shearwalls
Roof Materials/Framing: Clay tile and built up - Wood framing
Intermediate Floors/Framing: Concrete Slab and joists
Ground Floor: Slab on Grade
Columns: Concrete — Foundation: Concrete Perimeter
General Condition of Structure: Good
Levels Below Grade? Ground floor is partially below grade
Special Features and Comments:

LATERAL-FORCE-RESISTING SYSTEM

	Longitudinal	Transverse
System:	Concrete shearwall	Concrete Shearwalls
Vertical Elements:	Concrete Columns	Concrete Columns
Diaphragms:	Roof - Plywood and Floor - Concrete	Roof - Plywood and Floor - Concrete
Connections:	Anchor Bolts	Anchor Bolts

EVALUATION DATA

BSE-1N Spectral Response Accelerations: S_{DS} = 1.589 — S_{D1} = .803
Soil Factors: Class = D-Default — F_a = 1.2 — F_v = 1.7
BSE-1E Spectral Response Accelerations: S_{XS} = 1.048 — S_{X1} = .623
Level of Seismicity: High — Performance Level: Immediate Occupancy
Building Period: T = .238
Spectral Acceleration: S_a = 1.048
Modification Factor: $C_mC_1C_2$ = 1 — Building Weight: W = 3014.7
Pseudolateral Force: V = $C_mC_1C_2S_aW$ = 3158.2

BUILDING CLASSIFICATION: C2 Concrete Shear Walls (with Stiff Diahp) C2a (with Flexible Diaph)

REQUIRED TIER 1 CHECKLISTS

	Yes	No
Basic Configuration Checklist	☑	☐
Building Type C2 Structural Checklist	☑	☐
Nonstructural Component Checklist	☐	☑

FURTHER EVALUATION REQUIREMENT: Tier 2 Deficiency Based Evaluation

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

17.1.2IO Basic Configuration Checklist

Table 17-3. Immediate Occupancy Basic Configuration Checklist

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference	Comments
Very 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	Load path exists, bu some are deficient in strength. IO checklist
C ☐ NC ☐ N/A ☒ U ☐	ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building is greater than 0.5% of the height of the shorter building in low seismicity, 1.0% in moderate seismicity, and 3.0% 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	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Status				Evaluation Statement	Tier 2 Reference	Commentary Reference	Comments
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	

Status				Evaluation Statement	Tier 2 Reference	Commentary Reference	Comments
Low Seismicity (Complete the Following Items in Addition to the Items for Very 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	Reference: Cupertino GIS map https://gis.cupertino.org/propertyinfo/
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	Reference: Cupertino GIS map https://gis.cupertino.org/propertyinfo/
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	Reference: Cupertino GIS map https://gis.cupertino.org/propertyinfo/

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Status				Evaluation Statement	Tier 2 Reference	Commentary Reference	Comments
Moderate and High Seismicity (Complete the Following Items in Addition to the Items for Low 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	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

17.12IO Structural Checklist for Building Types C2: Concrete Shear Walls with Stiff Diaphragms and C2a: Concrete Shear Walls with Flexible Diaphragms

Table 17-25. Immediate Occupancy Structural Checklist for Building Types C2 and C2a

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference	Comments
Very Low 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	The shear walls supporting the roof are significantly overstressed
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. The spacing of reinforcing steel is equal to or less than 18 in. (457 mm).	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	Some portions of connection and ability transfer loads will require additional investigation in Tier 2. Wall anchorage at veranda is undersized and requires repair
C ☐ NC ☐ N/A ☐ U ☒	TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of loads to the shear walls, and the connections are able to develop the lesser of the shear strength of the walls or diaphragms.	5.7.2	A.5.2.1	Diaphragms are connected to the shear-walls. Some portions of connection and ability transfer loads will require additional investigation in Tier 2.

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference	Comments
C ☐ NC ☐ N/A ☐ U ☒	FOUNDATION DOWELS: Wall reinforcement is doweled into the foundation, and the dowels are able to develop the lesser of the strength of the walls or the uplift capacity of the foundation.	5.7.3.4	A.5.3.5	Walls are doweled into the foundations. Additional capacity checks required in Tier 2
Foundation System				
C ☐ NC ☐ N/A ☒ U ☐	DEEP FOUNDATIONS: Piles and piers are capable of transferring the lateral forces between the structure and the soil.		A.6.2.3	
C ☐ NC ☐ N/A ☒ U ☐	SLOPING SITES: The difference in foundation embedment depth from one side of the building to another does not exceed one story.		A.6.2.4	

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference	Comments
Low, Moderate, and High Seismicity (Complete the Following Items in Addition to the Items for Very Low Seismicity)				
Seismic-Force-Resisting System				
C ☐ NC ☒ N/A ☐ U ☐	DEFLECTION COMPATIBILITY: Secondary components have the shear capacity to develop the flexural strength of the components and are compliant with the following items in Table 17-23: COLUMN-BAR SPLICES, BEAM-BAR SPLICES, COLUMN-TIE SPACING, STIRRUP SPACING, and STIRRUP AND TIE HOOKS.	5.5.2.5.2	A.3.1.6.2	Column bar splices are less than 50db. Girders do not have continuous stirrups
C ☐ NC ☐ N/A ☒ U ☐	FLAT SLABS: Flat slabs or plates not part of 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. Coupling beams have the capacity in shear to develop the uplift capacity of the adjacent wall.	5.5.3.2.1	A.3.2.2.3	
C ☒ NC ☐ N/A ☐ U ☐	OVERTURNING: All shear walls have aspect ratios less than 4-to-1. Wall piers need not be considered.	5.5.3.1.4	A.3.2.2.4	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

C	NC	N/A	U				
	X			CONFINEMENT REINFORCING: For shear walls with aspect ratios greater than 2-to-1, the boundary elements are confined with spirals or ties with spacing less than $8d_b$.	5.5.3.2.2	A.3.2.2.5	Spacing of ties is less than specified.
	X			WALL REINFORCING AT OPENINGS: There is added trim reinforcement around all wall openings with a dimension greater than three times the thickness of the wall.	5.5.3.1.5	A.3.2.2.6	No Top reinforcement above openings at ground level walls on north side
X				WALL THICKNESS: Thicknesses of bearing walls are not less than 1/25 the unsupported height or length, whichever is shorter, nor less than 4 in. (101 mm).	5.5.3.1.2	A.3.2.2.7	
Diaphragms (Stiff or Flexible)							
X				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	
	X			OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 15% of the wall length.	5.6.1.3	A.4.1.4	In two locations adjacent to stairway openings the concrete shear walls have an opening for the full length of the wall
		X		PLAN IRREGULARITIES: There is tensile capacity to develop the strength of the diaphragm at reentrant corners or other locations of plan irregularities.	5.6.1.4	A.4.1.7	
		X		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	
Flexible Diaphragms							
	X			CROSS TIES: There are continuous cross ties between diaphragm chords.	5.6.1.2	A.4.1.2	Continuous cross ties between chords are not present or detailed
		X		STRAIGHT SHEATHING: All straight-sheathed diaphragms have aspect ratios less than 1-to-1 in the direction being considered.	5.6.2	A.4.2.1	
X				SPANS: All wood diaphragms with spans greater than 12 ft (3.6 m) consist of wood structural panels or diagonal sheathing.	5.6.2	A.4.2.2	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

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C ☒	NC ☐	N/A ☐	U ☐	DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 30 ft (9.2 m) and aspect ratios less than or equal to 3-to-1.	5.6.2	A.4.2.3
C ☐	NC ☐	N/A ☒	U ☐	NONCONCRETE FILLED DIAPHRAGMS: Untopped metal deck diaphragms or metal deck diaphragms with fill other than concrete consist of horizontal spans of less than 40 ft (12.2 m) and have aspect ratios less than 4-to-1.	5.6.3	A.4.3.1
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; the pile cap reinforcement and pile anchorage are able to develop the tensile capacity of the piles.	5.7.3.5	A.5.3.8

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Project Name	Cupertino City Hall
Project Number	21143.P5 CP

17.12CP Structural Checklist for Building Types C2: Concrete Shear Walls with Stiff Diaphragms and C2a: Concrete Shear Walls with Flexible Diaphragms

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

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference	Comments
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	The shear walls supporting the roof are significantly overstressed
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	Some portions of connection and ability transfer loads will require additional investigation in Tier 2. Wall anchorage at veranda is undersized and requires repair
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	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

Status				Evaluation Statement	Tier 2 Reference	Commentary Reference	Comments
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	Tier 2 analysis is required
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	In two locations adjacent to stairway openings the concrete shear walls have an opening for the full length of
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	Continuous cross ties between chords are not present or detailed
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	

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown

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

Legend: C = Compliant, NC = Noncompliant, N/A = Not Applicable, U = Unknown