Attachment C

Seismic Evaluation Report

Monterey County Courthouse, Annex & Parking Garage

1200 Aguajito Road, Monterey, CA 93940

June 30, 2018

Prepared For:
County of Monterey

Prepared By:

RIM

ZFA STRUCTURAL ENGINEERS

RIM

ALASKA

CALIFORNIA

GUAM

HAWAI'I

June 30, 2018

Dave Pratt
Project Manager II
County of Monterey
Resource Management Agency
1441 Schilling Pl, 2^{nd} Floor
Salinas, CA 93901

Re: County of Monterey
Seismic Evaluation of 3 Facilities at 1200 Aquajito Road
Monterey, California

Dear Mr. Pratt,

We are pleased to submit the following Final Seismic Evaluation Report and Cost Estimate for review and comment.

Objective of Report

This Seismic Evaluation Report covers three buildings; the Monterey County Courthouse, the Courthouse annex, and the adjacent parking structure. These buildings have been reviewed for Life Safety performance level using the ASCE 41-13 Standard for Seismic Evaluation and Retrofit of Existing Buildings, Tier 1 and Tier 2 Evaluations. This standard is based on structural and non-structural damage that has occurred in previous earthquakes and provides a means to identify general deficiencies based on anticipate behavior of specific building types.

The objective of the Report is to:

1. Complete a Seismic Evaluation of these three structures to current ASCE/SEI Standards.
2. Identify necessary structural improvements to bring the building into compliance.
3. Identify the architectural, mechanical, electrical, and data scope impacts for anticipated structural improvements.
4. Prepare a conceptual cost estimate identifying the costs of all improvements.

Summary of Methodology

This Seismic Evaluation was initiated with a site investigation by RIM Architects and ZFA structural engineers on September 30, 2017. The purpose of the site visit was to visually identify existing deficiencies in the structures, and to further understand the design of the structure and existing Architectural and Engineering features that might be affected.

ZFA Structural Engineers completed their analysis, and provided structural recommendations on improvements to shear walls, columns, pre-cast panels, and other structural components. Based on the improvements within the building, RIM Architects provided architectural input as to what floor finishes, partitions, ceilings, and other specialty finishes were affected by the structural upgrades. Electrical, mechanical, and data scope were also identified.

From this information O'Conner Construction Management completed the cost estimate, working in collaboration with RIM Architects and ZFA Structural Engineers.

We are proud of the depth of analysis contained in this report. Please review and comment. We are open to any discussion on any aspect of our report. Thank you for the opportunity to provide you this service.

Sincerely,

RIM

Eric R. Nelson, AIA, NCARB
Principal

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Section 2: Conceptual Design Cost Estimate

Section 1
Seismic Evaluation Report

ZFA STRUCTURAL ENGINEERS

Final Seismic Evaluation Report

Monterey County Courthouse, Annex, and Parking Garage

1200 Aguajito Road, Monterey, CA 93940

ZFA Project: 17661

June 30, 2018

Prepared For:

RIM Architects

San Francisco, CA

Prepared By:

Chelsea Drenick, Senior Engineer

Ryan Bogart, Senior Associate

Mark A. Moore, Principal in Charge

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EXECUTIVE SUMMARY

The Monterey County Courthouse, Annex, and Parking Garage, located at 1200 Aguajito Road in Monterey, California, have been reviewed for a **Life Safety** performance level using the ASCE 41-13 Standard for Seismic Evaluation and Retrofit of Existing Buildings, Tier 1 and Tier 2 Evaluations. The buildings were reviewed using the original construction documents, structural Tier 1 checklists, and site visits. Nonstructural elements were not included in the scope of this review; however, one nonstructural item is included within the recommendations based on the as-built drawings and observations on site. Items indicated as "noncompliant" or "unknown" by Tier 1 checklists were reviewed using Tier 2 evaluation procedures, unless sufficient information was not available to perform a Tier 2 analysis (e.g. Liquefaction).

The review resulted in the following structural and geotechnical findings. Summary tables are provided for each structure indicating noncompliant and unknown items only.

COURTHOUSE AND ANNEX

As-built drawings were provided for the Courthouse only, not the adjacent Annex structure. However, due to the similar methods, materials, and date of construction, deficiencies noted for the Courthouse building can be reasonably assumed to be present in the Annex.

Structural Check	**Tier 1**	**Tier 2**
Basic Configuration Checklist		
Adjacent buildings	Noncompliant	n/a
Liquefaction	Unknown	n/a
Slope failure	Unknown	n/a
Surface fault rupture	Unknown	n/a
Checklist for Building Type C1		
Column shear stress	Noncompliant	Noncompliant
No shear failures	Noncompliant	Noncompliant
Strong column-weak beam	Noncompliant	Noncompliant
Beam bars	Noncompliant	Noncompliant
Column-bar splices	Noncompliant	Noncompliant
Beam-bar splices	Noncompliant	Noncompliant
Column-tie spacing	Noncompliant	Noncompliant
Stirrup spacing	Noncompliant	Noncompliant
Joint transverse reinforcing	Noncompliant	Compliant
Deflection Compatibility	Noncompliant	n/a

PARKING GARAGE:

Structural Check	Tier 1	Tier 2
Basic Configuration Checklist		
Torsion	Noncompliant	Noncompliant
Liquefaction	Unknown	n/a
Slope failure	Unknown	n/a
Surface fault rupture	Unknown	n/a
Ties between foundation elements	Unknown	n/a
Checklist for Building Type C1		
Column shear stress	Noncompliant	Noncompliant
Flat slab frames	Noncompliant	Compliant
No shear failures	Noncompliant	Noncompliant
Strong column-weak beam	Noncompliant	Compliant
Column-tie spacing	Noncompliant	Noncompliant
Stirrup spacing	Noncompliant	Compliant
Joint transverse reinforcing	Noncompliant	Compliant

A conceptual seismic retrofit was developed to address the deficiencies identified above. In general, the retrofit for all buildings includes the addition of new concrete shear walls for the full height of the building. The new concrete shear walls are supported by new concrete foundations. Fiber reinforced polymer jackets are installed on the existing concrete columns to provide additional ductility. Additional scope items include the improvement of the precast panel connection to the structure and localized collector strengthening.

The following evaluation report details our findings and recommendations.

INTRODUCTION

The purpose of this evaluation is to review and evaluate the structural systems of the subject buildings using criteria provided by ASCE 41-13. The evaluation criteria have been tailored for specific building types and desired levels of building performance. This standard is based on criteria developed from observation of structural and nonstructural damage occurring in previous earthquakes and provides a means to identify general deficiencies based on 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 through the use of standard checklists and simplified structural calculations. Checklist items are general in nature and are intended to highlight building components that do not exceed conservative construction guidelines. If the element is compliant, it is anticipated to perform adequately under seismic loading without additional review or strengthening. Items indicated as noncompliant 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 the items determined to be potential deficiencies by Tier 1 checklists and simplified calculations. Noncompliant items are evaluated for calculated linear seismic demands as determined by ASCE 41-13. If the element is compliant with Tier 2 analysis procedures, the Tier 1 deficiency is waived. However, if the element remains noncompliant after the more detailed Tier 2 analysis, repair or remediation of the deficiency is recommended.

In certain cases, a more detailed Systematic Evaluation (Tier 3) may be more appropriate for complex structures where a Tier 2 analysis may be considered significantly conservative. A Tier 3 structural evaluation generally requires a substantially greater level of effort than a Tier 2 review. A Tier 3 evaluation was not performed for these buildings.

EVALUATION OVERVIEW

This seismic evaluation report for the existing buildings located at 1200 Aguajito Road in Monterey, California is based on the following:

- The American Society of Civil Engineers / Structural Engineering Institute (ASCE/SEI 41-13) *Standard for Seismic Evaluation and Retrofit of Existing Buildings* - Tier 1 and Tier 2 (noncompliant items only), Life Safety-level structural evaluation criteria.

- One site visit for general review of structures performed on September 30, 2017. No destructive testing or removal of finishes was performed or included in scope.

- Review of following original drawings by Wallace Holm AIA Architect and Associates:
    - Structural and architectural drawings for the Courthouse, dated Nov. 29, 1966 (15 & 23 sheets)
    - Architectural drawings for the Annex, dated Feb. 1, 1965 (3 sheets)
    - Structural and architectural drawings for the Parking Garage, dated Jan. 2, 1973 (10 & 3 sheets)

- Existing material properties as indicated in Appendix C.

- A Geotechnical report was not available. Minimal Geotechnical information showing boring profiles is present on the Parking Garage architectural drawings.

- A formal Tier 1 evaluation of nonstructural elements is not included. (One nonstructural item is noted in the recommendations based on information in the as-built drawings.)

STRUCTURE OVERVIEW

General Site Description

The site is located on a gently sloping lot approximately 100 feet southeast of Highway 1 and approximately 1.5 miles west of the Monterey Regional Airport on Aguajito Road. The campus is located with the main entrance facing east towards Aguajito Road, and the Courthouse building on the north side of the site, the Annex immediately south of the Courthouse, and the Parking Garage approximately 100 feet south of the Annex. The site and structures are currently occupied by Monterey County and in use as office, health, and courthouse facilities.

Structural Performance Objective

Per ASCE 41-13, a structural performance objective consists of a target performance level for structural elements in combination with a specific seismic hazard level. For seismic assessment of the subject building, the Basic Performance Objective for Existing Buildings (BPOE) was selected. While the BPOE seeks safety for occupants with reasonable confidence, it allows existing structures to be assessed for seismic forces that are less than those required for the design of new structures under the current building code. Buildings meeting the BPOE are expected to experience nominal damage from relatively frequent, moderate earthquakes, but have the potential for significant damage and economic loss from the most severe, though less frequent, seismic events.

For the purposes of this review to the BPOE, the specified level of performance is **Life Safety (3-C)** for this non-essential structure (Risk Category II as defined by ASCE 7). The Life Safety Performance Level as described by ASCE/SEI 41-13: *'Structural Performance Level S-3 is defined as the post-earthquake damage state in which a structure has damaged components but retains a margin against the onset of partial or total collapse. Nonstructural Performance Level N-C is the post-earthquake damage state in which Nonstructural Components may be damaged, but the consequential damage does not pose a Life Safety threat.'*

Site Seismicity (Earthquake Activity)

Per ASCE 41-13, '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 S_s and S_1 which are then modified by site coefficients F_a and F_v to produce the Design Spectral Accelerations, S_{DS} (short period) and S_{D1} (1-second period). 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. Where buildings occur in lower levels of seismicity, the strength and ductility required for better performance is significantly reduced, and building components or connections with additional strength capacity can in some cases be adequate despite lacking ductility.

Since sufficient data for the determination of the building Site Class at the subject site was not provided for review, a soil profile of *Site Class D* shall be assumed per ASCE 41-13, Section 2.4.1.6.2, for use in determination of site coefficients F_a and F_v.

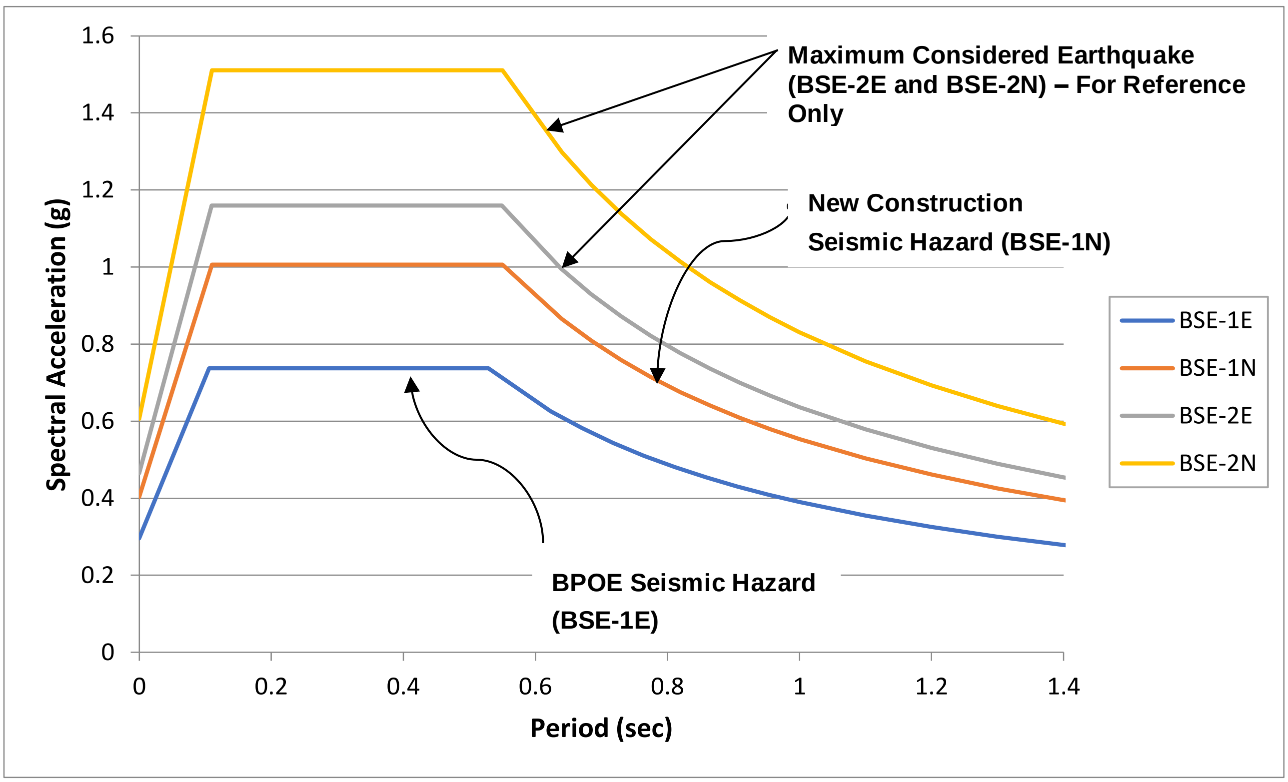
Per the site values indicated by USGS data and evaluated using seismic acceleration equations and tables of ASCE 41-13, the site is located in a region of **High Seismicity** with a design short-period spectral response acceleration parameter (S_{DS}) of 1.009g and a design spectral response acceleration parameter at a one second period (S_{D1}) of 0.555g. Per the table shown below, both of these parameters exceed the lower boundaries for high seismicity classification, 0.5g for S_{DS} and 0.2g for S_{D1}.

*Level of Seismicity**	*S_{DS}*	*S_{D1}*
Low	< 0.167g	< 0.067g
Moderate	≥ 0.167g < 0.500g	≥ 0.067g < 0.200g
High	≥ 0.500g	≥ 0.200g

*Where S_{DS} and S_{D1} values fall in different levels of seismicity, the higher level shall be used.

The spectral response parameters S_S and S_1 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 in accordance with ASCE 41 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 and Tier 2 analyses, since values obtained for the BSE-1E hazard level need not exceed the hazard levels for new construction.

The following chart depicts the response spectra for the multiple seismic hazard levels defined by ASCE 41-13: two existing hazard levels and two hazard levels corresponding to code design of new structures (ASCE 7).



<u>Structural System and Materials Description</u>

General

There are three buildings on the subject site: A four-story Courthouse building, a two-story Annex building immediately adjacent to the Courthouse, and a three-level parking garage located approximately 100 feet south of the Annex. These structures are shown on the site map in Figure 1.

Figure 1: Site map

Courthouse: The Courthouse building is a four-story concrete structure with precast concrete façade panels that was designed in 1966. It is approximately 57,300 square feet in total building area, and the ground floor is partially built into the hillside such that it daylights on the north and west faces. The building currently houses courtrooms and administration offices, and floor-to-floor heights are 13'-6" at the basement level, 16'-6" at the plaza level, and 15'-0" at the third and fourth floor levels. The overall building height is approximately 60'-0" to the roof slab. Figure 2 shows a longitudinal section through the building looking north, and Figure 3 shows a transverse section through the building and adjacent Annex looking west.

Annex: The Annex building is a two-story concrete structure with precast concrete façade panels that was designed in 1965. It is approximately 24,200 square feet, and the ground floor is partially underground, daylighting on the west face and on the west portion of the south face. The Annex is joined to the Courthouse at the basement level and is separated from the Courthouse by approximately three inches at the above-grade second level. Floor-to-floor heights are 13'-6" at both levels. Figure 3 shows a section through the Annex and Courthouse buildings.

Parking Structure: The Parking structure is a three-level concrete structure (including parking on the roof level) designed in 1973. It is approximately 34,200 square feet and is located on a sloping grade (sloping down to the west). Figure 4 shows a plan and section view of the Parking Garage.

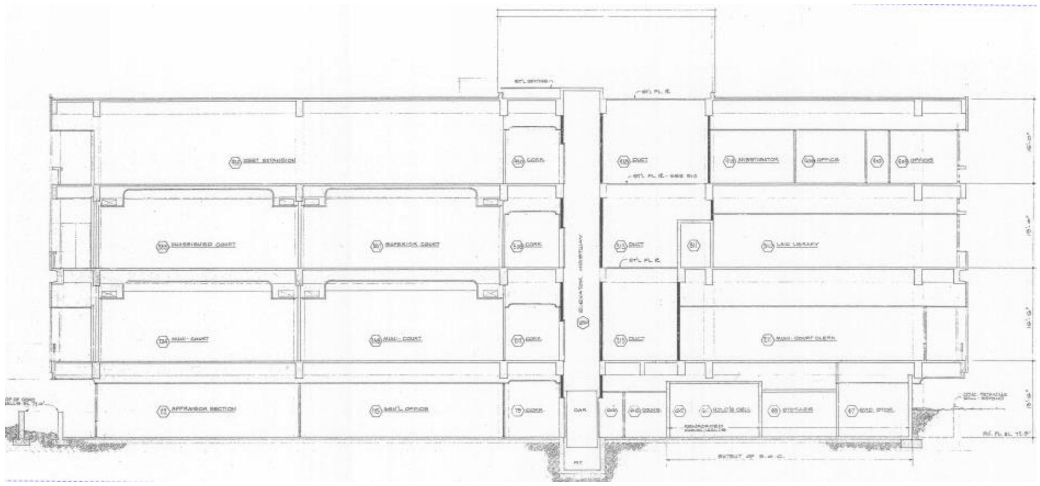


Figure 2: Longitudinal section through Courthouse looking North

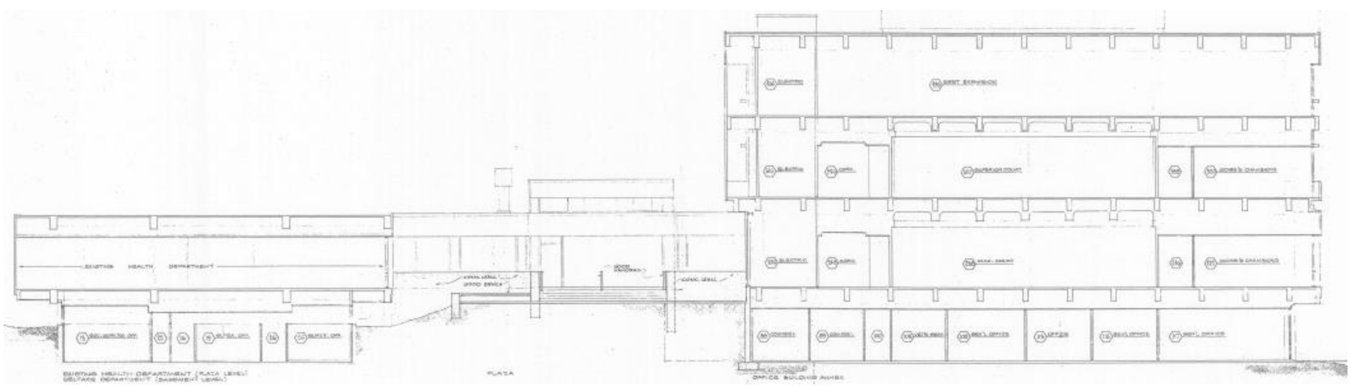


Figure 3: Transverse section through Courthouse (right) and Annex (left) looking West

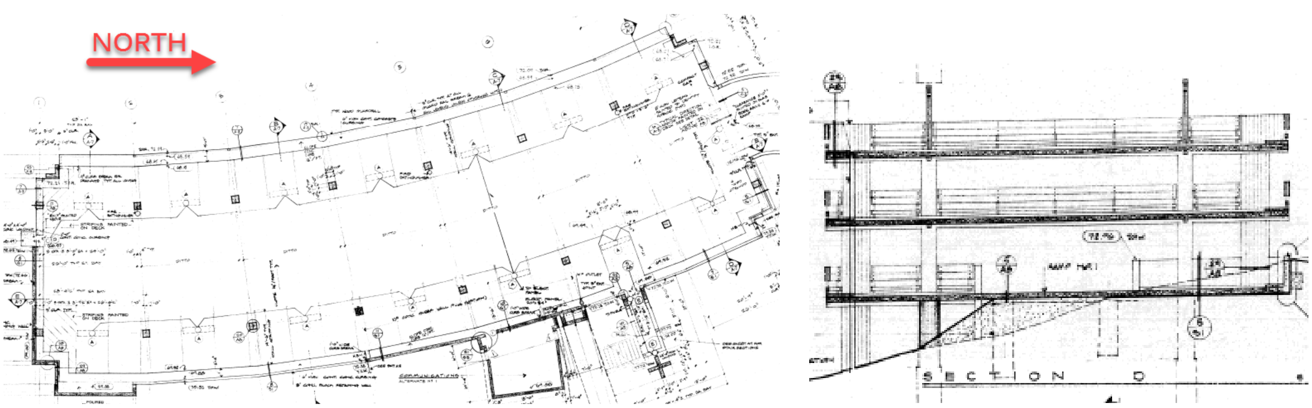


Figure 4: Plan view of Parking Garage (left); Section through Parking Garage looking north

Roof Framing

Courthouse: The roof level of the Courthouse building is approximately 60 feet above the basement level floor. Roof framing consists of one-way concrete slabs over metal deck that span to concrete roof beams at 8’-0” spacing. The roof beams span east-west across 36’-0” bays to concrete girders. The concrete girders span north-south across 24'-0" bays to concrete columns. At the east and west sides of the building, the beams cantilever 8'-0" past the last girder line. At the north and south sides of the building, the girders cantilever 5'-6" past the column line to support a beam at the edge of the slab. Over the framing is a built-up roofing system that drains to a number of interior drains. In the center of the roof slab is a mechanical penthouse that occurs over two framing bays (~36'-0" x 48'-0") and is 16'-0" tall. This penthouse structure is steel-framed with a corrugated steel un-topped roof deck that spans to steel beams at 6'-0" spacing supported by steel girders and steel wide flange columns.

Annex: The structural drawings for the Annex were not available for review. Based on the information in the Courthouse drawings, limited architectural drawings of the Annex, and the construction type observed on site, the framing at the Annex is assumed to be similar to the Courthouse.

Parking Garage: See Floor Framing section.

Floor Framing

Courthouse: The floor levels of the Courthouse building are framed very similarly to each other, and in a similar configuration to the roof. One-way concrete slabs over metal deck span to concrete beams at 8'-0" spacing. Beams span east-west across 36'-0" bays to concrete girders, and girders span north-south across 24'-0" bays to concrete columns. The framing cantilevers on all four sides of the building similar to the roof framing, and the exterior glazing occurs at the edge of the slab typically on the upper two floors. At the lower floors, including the daylighting basement, glazing typically is typically set back from the edge of the slab, occurring along the column lines. A balcony occurs around the perimeter of the plaza level, and at numerous locations on the third and fourth floors. Columns are on a regular grid (24x36 feet) and are typically 18-inch square at the third and fourth floors and 24-inch square at the basement and plaza floors.

Annex: The structural drawings for the Annex were not available for review. Based on the information in the Courthouse drawings, limited architectural drawings of the Annex, and the construction type observed on site, the framing at the Annex is assumed to be similar to the Courthouse.

Parking Garage: Each of the three floors of the parking garage is framed the same using post-tensioned slabs and girders. The slab is post-tensioned in both directions, but only spans one direction (north-south) to girders spaced at approximately 25-foot spacing. The girders are also post-tensioned and are supported by two columns each, oriented with a middle span of 34 feet between the columns and a cantilever of 13.5 feet at each end. Columns are 24-inches square between the first and second level and between the second and third level. Below the first level, the columns increase to 32-inch diameter and vary in height above grade depending on the slope of the hillside.

Walls

Courthouse: All interior and exterior walls at the Courthouse are non-bearing partitions constructed of light-gage metal studs. Most interior walls are one-hour fire rated; shaft walls have a two-hour rating.

Annex: The structural drawings for the Annex were not available for review. The framing at the Annex is assumed to be similar to the Courthouse.

Parking Garage: Only one structural wall occurs at the parking garage, which is a 10-inch-thick concrete shear wall along the east face of the building. At the upper level, the wall occurs in one bay (approximately 25-foot length) and at the lower floor, the wall occurs in two bays (approximately 50-foot length, with openings).

Seismic Force-Resisting Systems

Courthouse: The de-facto lateral system at the Courthouse building is concrete moment frames. However, the reinforcement design and detailing in the beams and columns does not provide sufficient ductility to resist design seismic forces required by modern building codes.

Annex: The structural drawings for the Annex were not available for review. The lateral system at the Annex is assumed to be the same as the Courthouse.

Parking Garage: In the north-south direction, one reinforced concrete shear wall occurs at the east face of the building. As the rest of the system in this direction consists of flat slabs framing to concrete columns, the structure is expected to be extremely torsionally irregular in this direction. The de-facto lateral system in the east-west direction is concrete moment frames, formed by post-tensioned beams and cast-in-place columns. These frames were not designed and detailed to provide sufficient ductility to resist design seismic forces required by modern building codes.

Foundations

Courthouse: The Courthouse is supported by shallow pad footings at interior and exterior column locations. The footings range in size from 9'-4" square to 10'-8" square and are typically 36 inches deep. They are typically well-reinforced with #9 bars at approximately 4-inch spacing in both directions, placed with three inches clear to the bottom of the footing. Concrete columns are concentric to the footings and column reinforcement typically continues into the bottom of the footing. A 6-inch slab on grade is present throughout the basement floor, and retaining walls occur on the south and east sides of the building.

Annex: The structural drawings for the Annex were not available for review. The foundation system at the Annex is assumed to be similar to the Courthouse, though the pad footings are expected to be significantly smaller since the building loads are expected to be approximately half of what they are at the Courthouse.

Parking Garage: Concrete columns that support the post-tensioned beam systems at the Parking Garage continue into the ground and support the building as drilled caissons. Caissons are 36-inches in diameter and extend approximately 26 feet below grade. At the west column line, the columns above the caissons (32-inch diameter) extend above grade three to seven feet up to the first floor.

Field Verification and Condition Assessment

The structures on the campus appear to be in generally good structural condition with minimal structural damage or deterioration apparent, and appear to be constructed in general accordance with the provided structural drawings, although there are a few items of note.

Courthouse: Localized damage to the exterior sidewalk slab and stair on the southeast side of the building was observed. The cracking appears to have been caused by settlement of the slab adjacent to the building, but does not appear to be structurally significant to the building itself. See Photo 6 in Appendix A.

Annex: Spalling of an exterior concrete column was observed at the northeast corner of the Annex. The spalling appears to be isolated to this column and does not appear to be structurally significant. See Photo 7 in Appendix A.

A large crack was observed in a concrete beam and exterior concrete wall at the exterior of the northwest corner of the Annex building adjacent to the Courthouse. The crack may have been caused by differential settlement between the two buildings, although no signs of settlement were readily observed in the rest of the building. Therefore, the crack does not appear to be structurally significant. See Photo 8 in Appendix A.

Parking Garage: Systemic corrosion was observed in nonstructural components throughout the parking garage, particularly in guardrails and associated hardware. Dry rot and or deterioration due to pests was also observed in wood members on the roof. None of these conditions are judged to be structurally significant. See Photos 9 and 10 in Appendix A.

Material Properties

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

Building Type

Per ASCE/SEI 41-13, these building can be classified as **Building Type C1: Concrete Moment Frames**. As described by ASCE/SEI 41-13: *'These buildings consist of a frame assembly of cast-in-place concrete beams and columns. Floor and roof framing consists of cast-in-place concrete slabs, concrete beams, one-way joists, two-way waffle joists, or flat slabs. Seismic forces are resisted by concrete moment frames that develop their stiffness through monolithic beam-column connections. In older construction, or in levels of low seismicity, the moment frames may consist of the column strips of two-way flat slab systems. Modern frames in levels of high seismicity have joint reinforcing, closely spaced ties, and special detailing to provide ductile performance. This detailing is not present in older construction. The foundation system may consist of a variety of elements.'*

Note that the Parking Garage consists of concrete moment frames in the east-west direction and a dual system in the north-south direction, composed of a concrete shear wall and moment frames. For the purpose of this evaluation, this structure was evaluated using the criteria for Building Type C1. However, retrofit solutions included a review of the contributions from the concrete shear wall.

Historical Performance

Modern concrete lateral-resisting systems, including moment frames, are designed to be ductile. When subjected to strong ground motions, earthquake-induced cyclic loading forms "plastic hinges"—deformations that absorb earthquake energy and dissipate forces—before the concrete fails in shear. The design of semi-ductile and ductile concrete systems began in the late 1960s and the early 1970s when prescriptive rules for lateral-resisting elements were implemented to govern design. Structures built prior to this were non-ductile and tended to be relatively weak. Prior to the prescriptive requirements of the late 1960s, when spiral/shear ties were not always included in design, columns could fail in shear before plastic hinges could form. These buildings are known to perform poorly and may collapse when ground motion is strong enough to initiate shear failures in the columns. The semi-ductile and ductile system designs that began in the late 1960s include many of the modern design features and are expected to perform satisfactorily per code in a seismic event, provided there are no vertical or horizontal configuration irregularities that will concentrate demand on individual components.

Benchmark Buildings

In addition to classifying buildings by type of construction, ASCE 41 identifies 'Benchmark Buildings' for each type. The detailing of seismic force-resisting systems in Benchmark Buildings is generally considered to meet the performance requirements of ASCE 41. When a building is determined to meet Benchmark Building requirements through field verification of construction compliant with benchmark code requirements, only the review of foundation and nonstructural elements is required. Even though a building appears to meet the benchmark criteria, a full analysis may still be recommended under certain circumstances.

For building type C1, the 1993 seismic design provisions are referenced as the oldest permitted standard. Since the Courthouse and Annex were designed in 1966 and 1965, respectively, they are assumed to have been designed to the 1964 Uniform Building Code. The Parking Garage documents indicate that the design is in accordance with the 1970 Uniform Building Code. Therefore, none of the subject buildings meet the criteria of a Benchmark Building and a complete Tier 1 analysis is required.

ANALYSIS OVERVIEW

Tier 1 analysis requires only hand-calculations to check existing conditions against "quick check" procedures listed in the Tier 1 checklists. These analyses are detailed in Appendix E of this report and described conceptually in the Findings and Recommendations section below.

The Tier 2 analysis of each building requires a full-building model to be created to analyze each building's response to seismic forces. A full-building, three-dimensional model was constructed in ETABS analysis software for the Courthouse and Parking Garage buildings, as shown in Figures 5 and 6. Modeled elements include concrete roof and floor slabs, concrete beams and girders, concrete columns, and the concrete shear wall in the Parking Garage. The seismic analysis procedure used was a Linear Static Procedure with seismic accelerations in accordance with Seismic Hazard Level BSE-1E in ASCE 41-13.

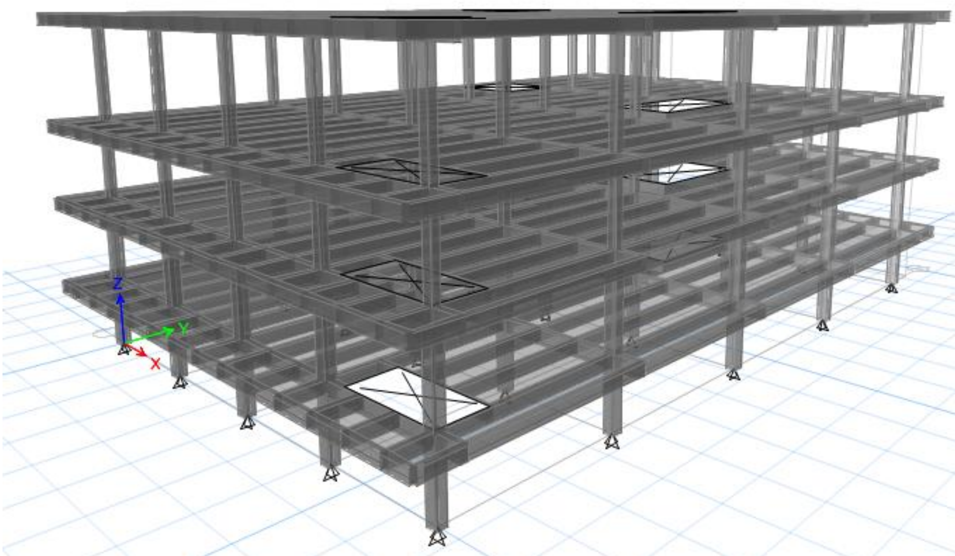


Figure 5: Courthouse model in ETABS software

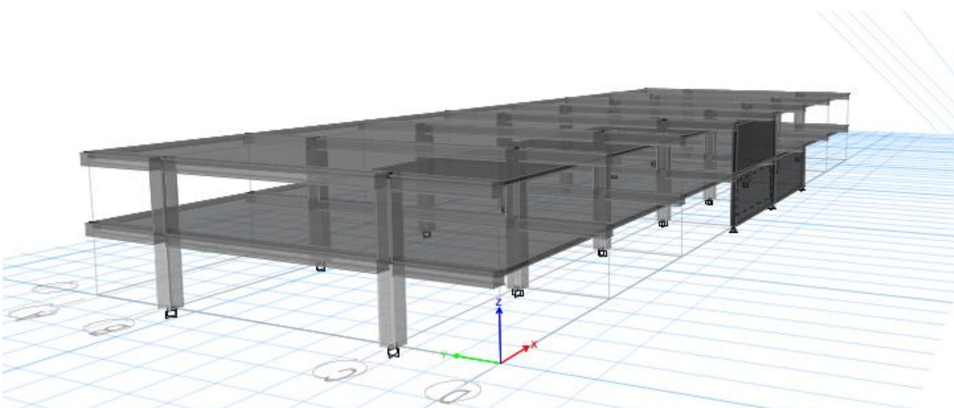


Figure 6: Parking Garage model in ETABS software

FINDINGS AND RECOMMENDATIONS

Structural

The ASCE 41-13 Tier 1 Basic Life Safety and Building Type Specific Checklists indicate multiple noncompliant and unknown areas for Life Safety Performance. For the Courthouse, 11 items were found to be noncompliant with the Tier 1 checks and an additional three items were unknown. The Annex is assumed to have the same deficiencies as the Courthouse. For the Parking Garage, eight items were found to be noncompliant with the Tier 1 checks, and an additional four items were unknown. The noncompliant and unknown deficiencies for each building are listed below, along with the Tier 2 evaluation results (where applicable) and recommended actions.

Courthouse (and Annex)

a. ADJACENT BUILDINGS (ASCE Section 4.3.1.2): *"The clear distance between the building being evaluated and any adjacent building is greater than 4% of the height of the shorter building."* The height of the Annex above grade is approximately 13'-6". Four percent of 13'-6" is approximately 6.5 inches. Because the gap between the Courthouse and Annex is only approximately three inches, this Tier 1 deficiency is applicable.

   RECOMMENDATION: The addition of new shear walls in both buildings will provide additional stiffness and will decrease the anticipated lateral deformations of the buildings during an earthquake. Additional analysis should be performed to estimate horizontal movement in a seismic event with the additional stiffness considered. Minor damage may occur due to pounding between structures during a seismic event; however, damage due to this condition is not anticipated to cause life safety structural concerns within the subject building. Improved anchorage of the precast panel finish at these locations may also be prudent to reduce the probability of the panels being dislodged in a seismic event.

b. COLUMN SHEAR STRESS CHECK: *"The shear stress in the concrete columns, calculated using the Quick Check procedure of Section 4.5.3.2, is less than the greater of 100 lb/in.^{2} or $2\sqrt{f'_c}$."* The shear stress in the columns were found to be 1.5 to 2.7 times higher than the allowable stress in the quick check procedure and were therefore found to be noncompliant with the Tier 1 check.

   The Tier 2 analysis was performed in accordance with ASCE 41-13 Section 5.5.2.1.4. The maximum demands on the structure were extracted from the 3D ETABS model to compare with the capacities listed in Chapter 10 of ASCE 41-13. Calculations were performed to determine if the columns are shear- or flexure-controlled, which is required to determine the specific *m*-factor of the columns. Using the appropriate *m*-factor and knowledge factor, the capacities were found and compared to the demands. Columns have a demand-capacity ratio of 1.5 to 2.5 at each floor level and therefore remain noncompliant in the Tier 2 evaluation.

   RECOMMENDATIONS: Because columns were found to be inadequate to resist the BSE-1E seismic demands at the Life Safety performance level, a retrofit of the building is recommended to improve seismic performance. The recommended retrofit scope includes the addition of concrete shear walls in both directions at all levels. This new lateral system will significantly reduce building drifts and seismic demands on the concrete columns and provide the building with adequate strength and stiffness to meet the Life Safety performance objective at the BSE-1E hazard level. Additionally, we recommend "jacketing" the concrete columns at all floor levels with fiber-reinforced polymer (FRP) to provide adequate deformation compatibility with the new shear wall system.

c. NO SHEAR FAILURES: *"The shear capacity of frame members is able to develop the moment capacity at the ends of the members."* The columns in the basement level and some columns at the plaza level do not have sufficient capacity to develop the flexural strength of the section and therefore will experience a shear failure before a flexural failure. Columns at the third and fourth floor levels, as well as all beams and girder, are able to fully develop the strength in flexure before a shear failure occurs. Because of the deficiency in the basement and plaza level columns, the Tier 1 deficiency is applicable.

The Tier 2 analysis was performed in accordance with ASCE 41-13 Section 5.5.2.3.4. The Tier 2 check of these elements was included in the analysis of the previous item for Column Shear Stress. The columns remain noncompliant in the Tier 2 evaluation for this deficiency.

RECOMMENDATIONS: See the recommendations for Column Shear Stress above.

d. STRONG COLUMN-WEAK BEAM: *"The sum of the moment capacity of the columns is 20% greater than that of the beams at frame joints."* None of the interior girder-to-column connections meet the "strong column-weak beam" ratio required; therefore, the building was found to be noncompliant with the Tier 1 check.

The Tier 2 analysis was performed in accordance with ASCE 41-13 Section 5.5.2.1.5 and evaluated the interior girder-to-column connections. Similar calculations to Column Shear Stress Check were performed for this deficiency. The maximum shear and moment demands were extracted from the ETABS model and compared to the calculated capacities. All columns were found to be deficient in both shear and flexure, hence this deficiency remains noncompliant in the Tier 2 evaluation.

RECOMMENDATIONS: See the recommendations for Column Shear Stress above.

e. BEAM BARS: *"At least two longitudinal top and two longitudinal bottom bars extend continuously throughout the length of each frame beam. At least 25% of the longitudinal bars provided at the joints for either positive or negative moment are continuous throughout the length of the members."* The as-built structural drawings (Sheet S4) show that the beams and girders all have discontinuous reinforcement at certain locations; therefore, the structure is noncompliant with this Tier 1 check.

The Tier 2 analysis was performed in accordance with ASCE 41-13 Section 5.5.2.3.5 whereby flexural demands of noncompliant beams and girders were analyzed at the middle and ends of their span using an *m*-factor of 1.0. Maximum demands for the beams were extracted from the ETABS model to be compared to the capacity. All beams and girders had adequate capacity at the middle of their span, but none of the beams had adequate capacity at their ends. The only girders that had adequate capacity at their ends were at the roof level; all other girders did not have adequate capacity at their ends. Therefore, the beams and girders remain noncompliant in the Tier 2 evaluation for this deficiency.

RECOMMENDATION: See the recommendations for Column Shear Stress above. These recommendations will similarly relieve stresses on the beams as they do for the columns.

f. COLUMN-BAR SPLICES: *"All column-bar lap splice lengths are greater than $35d_b$ and are enclosed by ties spaced at or less than $8d_b$. Alternatively, column bars are spliced with mechanical couplers with a capacity of at least 1.25 times the nominal yield strength of the spliced bar."* Most of the columns in the building do not meet this requirement based on the reinforcement detailed in the as-built drawings. Therefore, the structure is noncompliant with this Tier 1 check.

The Tier 2 analysis was performed in accordance with ASCE 41-13 Section 5.5.2.3.6 whereby the flexural demands at noncompliant column splices were calculated. These occur at the plaza, third, and fourth floor levels. None of the Tier 1 noncompliant columns passed the Tier 2 check, and therefore the columns remain noncompliant for the Tier 2 evaluation of this deficiency.

RECOMMENDATIONS: See the recommendations for Column Shear Stress above.

g. BEAM-BAR SPLICES: *"The lap splices or mechanical couplers for longitudinal beam reinforcing are not located within $l_b/4$ of the joints and are not located in the vicinity of potential plastic hinge locations."* The lap splices in the beams are nearly always located in the regions noted based on the reinforcement detailed in the as-built drawings. Therefore, the structure is noncompliant with this Tier 1 check.

   The Tier 2 analysis was performed in accordance with ASCE 41-13 Section 5.5.2.3.6. The beams were analyzed in the same manner as the check for Beam Bars, but the *m*-factor was increased to 1.5 in accordance with Table 10-13 in ASCE 41-13. Even considering this increased *m*-factor value, the beams were still found to be noncompliant, and therefore the beams remain noncompliant for the Tier 2 evaluation of this deficiency.

   RECOMMENDATIONS: See the recommendations for Column Shear Stress above. These recommendations will similarly decrease stresses in the beams as they do for the columns.

h. COLUMN-TIE SPACING: *"Frame columns have ties spaced at or less than d/4 throughout their length and at or less than $8d_b$ at all potential plastic hinge locations."* Both 18-inch square and 24-inch square columns have ties spaced at 18 inches typical throughout their height and at 9 inches in the hinge zones. These spacings do not meet the requirements of this check and therefore the structure is noncompliant with this Tier 1 check.

   The Tier 2 analysis was performed in accordance with ASCE 41-13 Section 5.5.2.3.7. Column shear and flexural demands were checked in the Column Shear Stress and Column-Bar Splices checks and were found to be noncompliant. Therefore, the columns remain noncompliant for the Tier 2 evaluation of this deficiency.

   RECOMMENDATIONS: See the recommendations for Column Shear Stress above.

i. STIRRUP SPACING: *"All beams have stirrups spaced at or less than d/2 throughout their length. At potential plastic hinge locations, stirrups are spaced at or less than the minimum of $8d_b$ or d/4."* Typical girder stirrup spacing is 14 inches (including in hinge zones), and typical beam stirrup spacing is 18 inches (including hinge zones). For girders, stirrup spacing outside the hinge zone is compliant, but at all other locations (girders in the hinge zone, beams in all locations), the stirrup spacing is noncompliant. Therefore, the structure is noncompliant with this Tier 1 check.

   The Tier 2 analysis was performed in accordance with ASCE 41-13 Section 5.5.2.3.7 whereby seismic shear demands from ETABS were checked against shear capacities for typical beams and girders at each floor (at mid-span and over columns). All beams were noncompliant, with demand-to-capacity ratios ranging from 1.0 (just barely compliant) to 4.2. Roof girders were found to be compliant, but all other girders were noncompliant, with demand-to-capacity ratios ranging from 1.2 to 2.5. Therefore, the beams remain noncompliant for the Tier 2 evaluation of this deficiency.

RECOMMENDATIONS: See the recommendations for Column Shear Stress above. These recommendations will similarly decrease stresses in the beams as they do for the columns.

j. JOINT TRANSVERSE REINFORCING: *"Beam-column joints have ties spaced at or less than* $8d_b$*."* Based on the two checks above, this item is noncompliant with this Tier 1 check.

The Tier 2 analysis was performed in accordance with ASCE 41-13 Section 5.5.2.3.8. At all conditions, the shear demands from the ETABS model were found to be less than the capacities calculated in accordance with ASCE 41-13 Section 10.4.2.3.2. Therefore, this deficiency is waived in the Tier 2 analysis.

RECOMMENDATIONS: None.

k. DEFLECTION COMPATIBILITY: *"Secondary components have the shear capacity to develop the flexural strength of the components."* Precast concrete panel façade connections are nonductile and not designed to resist the forces generated when the structure experiences lateral movement from seismic forces. Therefore, the structure is noncompliant with this Tier 1 check.

The Tier 2 model of the structure did not model the precast concrete façade panel connections, as these are considered a nonstructural component.

RECOMMENDATIONS: Retrofit to the precast panel connections is recommended to achieve a more ductile attachment that can better respond to seismic building movement.

l. UNKNOWN ITEMS: Items present in the Tier 1 checklist that were noted to be "Unknown" due to lack of documentation or geotechnical information include the following:

   i. LIQUEFACTION: *"Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance shall not exist in the foundation soils at depths within 50 feet under the building."*
   ii. SLOPE FAILURE: *"The building site is sufficiently remote from potential earthquake-induced slope failures or rockfalls to be unaffected by such failures or is capable of accommodating any predicted movements without failure."*
   iii. SURFACE FAULT RUPTURE: *"Surface fault rupture and surface displacement at the building site are not anticipated."*

RECOMMENDATIONS: It is recommended that a Geotechnical Engineer is consulted and engaged to provide a report at the subject site that addresses the above "unknown" items.

Parking Garage

a. 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."* A concrete shear wall is present only on one side of the building, a condition that moves the center of rigidity to the east side of the building when considering seismic forces in the north-south direction. Therefore, by inspection, the building is noncompliant for this Tier 1 check.

The Tier 2 analysis was performed in accordance with ASCE 41-13 Section 5.3.2.6 whereby the torsional response was modeled in ETABS and a retrofit solution was investigated to mitigate the irregularity.

RECOMMENDATIONS: The addition of another concrete shear all on the west side of the building is recommended to remove the torsional irregularity.

b. COLUMN SHEAR STRESS CHECK: *"The shear stress in the concrete columns, calculated using the Quick Check procedure of Section 4.5.3.2, is less than the greater of 100 lb/in.^{2} or $2\sqrt{f'_c}$."* The shear stresses in the columns were found to be up to 2.0 times higher than the allowable stress in the quick check procedure and were therefore found to be noncompliant with the Tier 1 check.

The Tier 2 analysis was performed in accordance with ASCE 41-13 Section 5.5.2.1.4 whereby the maximum seismic force demands were determined from the ETABS model for the columns at each floor and compared to capacities calculated with the appropriate *m*-factor for each element. The columns below the first floor were found to be just under 100% stressed, but columns at the first and second floors were overstressed by 38% and 28% respectively. Therefore, the columns remain noncompliant in the Tier 2 evaluation for this deficiency.

RECOMMENDATIONS: The retrofit recommendation to remedy this item is twofold. First, in the north-south direction, another concrete shear wall is recommended to be added along the west face of the building. This will significantly reduce the seismic forces that are resisted by the columns and remove the torsional irregularity. Second, to accommodate seismic demands in the east-west direction, the columns on both floors are recommended to be "jacketed" in fiber-reinforced polymer (FRP) to increase shear capacity and ductility and improve lateral deflections (drift).

c. FLAT SLAB FRAMES: *"The seismic-force-resisting system is not a frame consisting of columns and a flat slab or plate without beams."* The configuration of the structure in the north-south direction does not meet this requirement, and therefore the building is noncompliant for this Tier 1 check.

The Tier 2 analysis was performed in accordance with ASCE 41-13 Section 5.5.2.3.1 whereby the flexural capacities of the flat slab were checked for seismic demands derived from the ETABS model. The slab was found to be adequate in flexure. Punching shear is not a concern with the configuration present because transverse beams are present between the flat slab and columns. Therefore, this deficiency is waived in the Tier 2 analysis.

RECOMMENDATIONS: None.

d. NO SHEAR FAILURES: *"The shear capacity of frame members is able to develop the moment capacity at the ends of the members."* The columns at the first and second floors do not have sufficient capacity to develop the flexural strength and therefore will experience a shear failure before a flexural failure. Columns at the ground floor (below the first floor) and beams at all floors are able to fully develop the strength in flexure before a shear failure occurs. Because of the deficiency in the first and second floor columns, the building is noncompliant for this Tier 1 check.

The Tier 2 analysis was performed in accordance with ASCE 41-13 Section 5.5.2.3.4. Because only the columns at the first and second floors were noncompliant in the Tier 1 check, the Tier 2 check of these elements was included in the analysis of the previous item for Column Shear Stress. The columns remain noncompliant in the Tier 2 evaluation for this deficiency.

RECOMMENDATIONS: See the recommendations for Column Shear Stress above.

e. STRONG COLUMN-WEAK BEAM: *"The sum of the moment capacity of the columns is 20% greater than that of the beams at frame joints."* The beam-to-column connections in the transverse direction (beam-column) at the second floor and roof do not meet the "strong column-weak beam" ratio required, therefore the building was found to be noncompliant with the Tier 1 check.

   The Tier 2 analysis was performed in accordance with ASCE 41-13 Section 5.5.2.1.5. The seismic demands were derived from the ETABS model and compared to the interior girder-to-column capacities using an *m*-factor of 2.0. Using this check, all connections were found to be compliant, with a maximum demand-to-capacity ratio of 0.88 occurring at the roof level in the transverse direction. Therefore, this deficiency is waived in the Tier 2 analysis.

   RECOMMENDATIONS: None.

f. COLUMN-TIE SPACING: *"Frame columns have ties spaced at or less than d/4 throughout their length and at or less than $8d_b$ at all potential plastic hinge locations."* The 32-inch diameter columns that occur below the first floor have spirals spaced at 2.5 inches, which meets the requirements for this check in both hinge and non-hinge zones. The 24-inch square columns at the first and second floors have ties at 12 inch spacing typical, which does not meet the requirement of this check. The tie spacing in the hinge zones for the 24-inch diameter columns are 6 inches and 2.25 inches at the roof and second floor respectively; the roof level spacing is noncompliant. Therefore, the structure is noncompliant with this Tier 1 check.

   The Tier 2 analysis was performed in accordance with ASCE 41-13 Section 5.5.2.3.7 whereby the force demand in noncompliant columns was calculated and the adequacy of the elements was evaluated. Column shear demands were checked in the Column Shear Stress check and were found to be noncompliant. Therefore, considering that no increases to the capacity (*m*-factors) are allowed, the columns remain noncompliant for the Tier 2 evaluation of this deficiency.

   RECOMMENDATIONS: See the recommendations for Column Shear Stress above.

g. STIRRUP SPACING: *"All beams have stirrups spaced at or less than d/2 throughout their length. At potential plastic hinge locations, stirrups are spaced at or less than the minimum of $8d_b$ or d/4."* Typical beam stirrup spacing is 18 inches and hinge zone stirrup spacing is 4 inches. Therefore, stirrup spacing outside the hinge zone is noncompliant, but hinge zone stirrup spacing is compliant. Therefore, the structure is noncompliant with this Tier 1 check.

   The Tier 2 analysis was performed in accordance with ASCE 41-13 Section 5.5.2.3.7 whereby seismic shear demands derived from the ETABS model were checked against shear capacities for typical beams (at mid-span and over columns). At all beam locations the shear demands were found to be less than the capacities. Therefore, this deficiency is waived in the Tier 2 analysis.

   RECOMMENDATIONS: None.

h. JOINT TRANSVERSE REINFORCING: *"Beam-column joints have ties spaced at or less than $8d_b$."* Based on the two checks above, this item is noncompliant with this Tier 1 check.

The Tier 2 analysis was performed in accordance with ASCE 41-13 Section 5.5.2.3.8. At all conditions, the shear demands from the ETABS model were found to be less than the capacities calculated in accordance with ASCE 41-13 Section 10.4.2.3.2. Therefore, this deficiency is waived in the Tier 2 analysis.

RECOMMENDATIONS: None.

i. UNKNOWN ITEMS: Items present in the Tier 1 checklist that were noted to be “Unknown” due to lack of documentation, geotechnical information, or access to view existing conditions include the following:

  iv. LIQUEFACTION: *“Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building’s seismic performance shall not exist in the foundation soils at depths within 50 feet under the building.”*
  v. SLOPE FAILURE: *“The building site is sufficiently remote from potential earthquake-induced slope failures or rockfalls to be unaffected by such failures or is capable of accommodating any predicted movements without failure.”*
  vi. SURFACE FAULT RUPTURE: *“Surface fault rupture and surface displacement at the building site are not anticipated.”*
  vii. 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.”* Ties are not present, but the Site Class is unknown due to a lack of geotechnical information.

RECOMMENDATIONS: It is recommended that a Geotechnical Engineer is consulted and engaged to provide a report at the subject site that addresses the above “unknown” items.

<u>Nonstructural</u>

The buildings were not evaluated for nonstructural elements. However, one nonstructural element of note at the Courthouse/Annex buildings was the precast panel façade. Based on the building type and precast panel connections documented in the as-built drawings, we estimate that the precast panel connections to the structure are not adequate to satisfy deflection compatibility requirements to accommodate expected seismic movements (drifts) of the building. Therefore, the precast panel façade elements are expected to present a falling hazard when the structure is subjected to significant ground motions. Retrofit scope will be required to address this concern to mitigate the hazard posed by these panels.

SCHEMATIC RETROFIT SCOPE

Courthouse

The Courthouse building has eight deficiencies in accordance with the Tier 2 procedures of ASCE 41-13. The deficiency-only retrofit recommendations provide remediation of these deficiencies to meet a Life Safety performance level in response to a BSE-1E seismic hazard level.

Concrete Shear Walls

The main retrofit scope for this structure is the addition of two 12-inch-thick, 25-foot-long concrete shear walls in each direction (4 walls total). The walls will be continuous from the basement to the roof of the structure. These walls are indicated on Figure 7 with an “A” symbol and are located where there appear to be the fewest conflicts with the existing architectural layout (the locations are flexible as long as they can be installed continuously from basement to roof). Door openings and mechanical penetrations through the walls are acceptable and can be detailed in the retrofit.

An approximate estimate of wall concrete and reinforcement quantities based on the retrofit solution analyzed in the Tier 2 evaluation is provided for each level below. This is based on a linear static, fixed-base analysis model.

Floor	**Volume of Concrete in New Shear Walls (cubic yards)**	**Weight of Reinforcing Steel (pounds per cubic yard of concrete)**
Basement	48	270
Plaza	60	185
Third	56	130
Fourth	56	90

Reinforcing steel will be doweled through each existing concrete slab and beams and lap spliced to the wall reinforcing above and below such that the reinforcing is continuous over the height of the wall. Assume two layers of dowels at 18-inch spacing at each floor.

Concrete Columns

The addition of concrete shear walls will greatly reduce the seismic forces in the existing columns. However, the columns will still require deformation compatibility with the expected seismic drift of the structure. Given the lack of shear reinforcing in the columns, we propose to apply fiber-reinforced polymer (FRP) to the columns, which will increase their ductility and accommodate the imposed lateral deflections.

The FRP would wrap completely around each column to provide confinement. We anticipate two to four layers of FRP at each column, and column corners will need to be chamfered to install the FRP. This may not be economically viable for the exterior columns, as the windows and waterproofing would have to be altered. Another option for the exterior columns would be to provide a secondary gravity system, such as a tube steel columns, directly inside of each concrete column. This would provide redundancy in case of a large seismic event that could cause shear failures in the concrete columns and potentially affect the column’s vertical load carrying capability. HSS column sizes are provided as an alternate in the table below. All columns will have a steel base and top plate with four epoxy anchors to the beam above and below.

Floor	**HSS Column Size**
Basement & Plaza	HSS9x9x5/8

Third & Fourth	HSS7x7x1/2

Exterior column locations are typically noted with a “B” symbol on Figures 7 and 8 (16 per floor); interior columns are noted with a “C” symbol (9 per floor). Assume that one HSS column will be placed at each exterior concrete column on each floor in this alternate, and that FRP will still be used at interior columns.

Foundation

New spread footings will be required below the new shear walls, as shown on the foundation plan in Figure 8. Connections from new to existing footings will be required at all new-to-existing interfaces.

An approximate estimate of foundation concrete and reinforcement quantities based on the retrofit solution analyzed in the Tier 2 evaluation is provided for each level below. This is based on a linear static, fixed base analysis model.

Volume of Concrete in New Footings	130 yd^3
Weight of Reinforcing Steel in New Footings	175 lbs/yd^3
Epoxy Dowel Quantity	#5 @ 12”oc top and bottom at all interfaces

Precast Connections

Precast panel elements form the façade of the building and are typically noted with a “D” symbol on Figure 7. The precast panel connections are nonductile attachments that have the potential to fail in a seismic event. Two options are available to mitigate this concern:

1. Replace each connection with a ductile connection. This connection will include two post-installed anchors to the panel and two to the structure with a ductile steel connector in between. Approximately 1,000 precast panel connections occur in the existing structure and need to be replaced.
2. Replace the precast panels with a façade that is compatible with the expected seismic deflections of the structure. This façade will likely be more lightweight than the existing, and will reduce the seismic mass of the structure and decrease the overall retrofit cost as well.

Collector Strengthening

Collector strengthening is required at the new concrete shear wall north of Gridline 6 at all levels. Collector strengthening may consist of either a steel angle (approximate weight = 50 plf) with epoxy anchors at 16-inch spacing to the existing concrete beam or unidirectional FRP applied to the top of the slab above the new shear wall. The steel angle or FRP strengthening would extend approximately 30 feet beyond the new shear wall.

Base Isolation

As an alternate, base isolation is a possible retrofit solution at this structure. This would consist of shoring the structure and providing base-isolating devices at all columns in the basement floor. Shear walls and column jacketing retrofit recommendations discussed above would largely not be required as base isolation reduces the seismic forces on the elements in a building. This option appears to be a cost-competitive alternate to the retrofit scope discussed above, and we recommend that this option be considered in further detail in subsequent design phases. ZFA has consulted with a base isolation manufacturer for a rough cost estimate for the isolation equipment (excluding shoring and installation costs). They estimate a cost of $200,000 for the isolation equipment.

Additional Cost Items

If a seismic retrofit is performed on the building, destructive testing of structural components will likely be required to verify the existing material strengths. The extent of the testing will be dependent on the jurisdiction's requirements, but for estimating purposes, we anticipate approximately six concrete cores and three reinforcing steel coupons will be required to be removed from the existing structural components and tested.

Further Recommendations

In order to investigate the "unknown" items on the Tier 1 checklist, we recommend the development of geotechnical report including a geological hazards assessment.

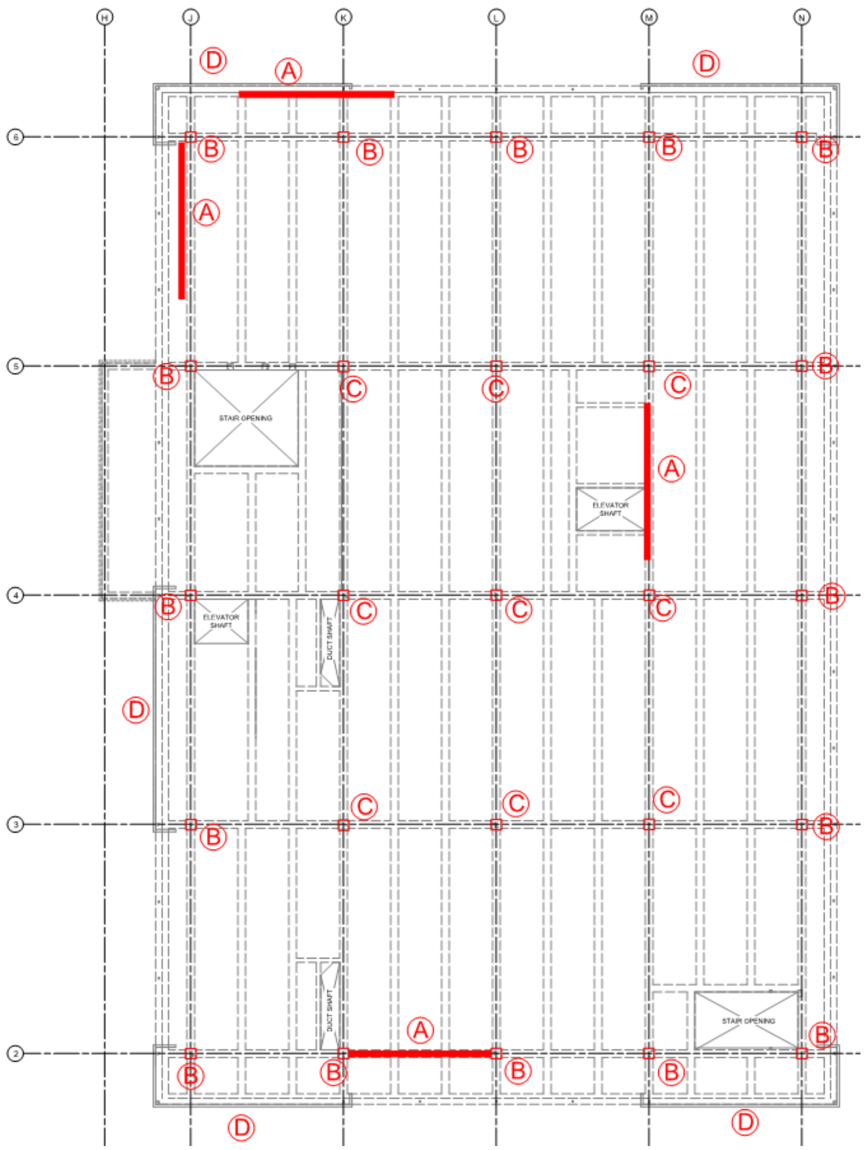


Figure 7: Typical floor plan at Courthouse indicating: A=proposed locations of retrofitted shear walls; B=exterior concrete columns; C=interior concrete columns; D=exterior precast concrete façade panels

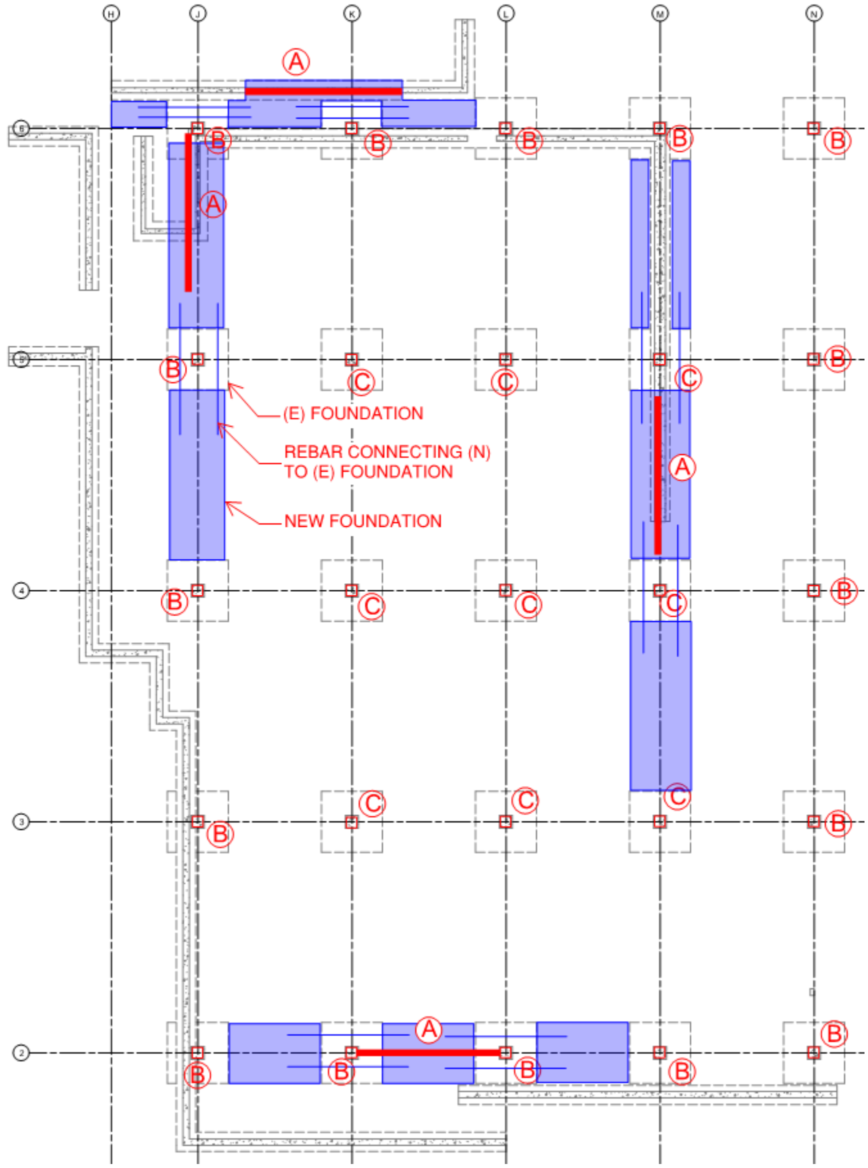


Figure 8: Foundation plan at Courthouse indicating: A=proposed locations of retrofitted shear walls; B=exterior concrete columns; C=interior concrete columns; purple boxes=new footings

<u>Annex</u>

Structural drawings were not available for the Annex building. Based on the limited architectural drawings available and on-site observations, it appears that the Annex building is similar to the Courthouse in materials and construction type. Therefore, the deficiencies present in the Tier 2 evaluation for the Courthouse are assumed to be present at the Annex, and the recommended retrofit is provided to remediate those assumed deficiencies.

Concrete Shear Walls

The main retrofit scope is the addition of 12-inch thick, approximately 12-foot long concrete shear walls in the north-south and east-west directions (5 total). The walls will be continuous from the ground floor to the roof of the structure. These walls are indicated on Figure 9 with an “A” symbol and are located where there appear to be the fewest conflicts with the existing architectural layout (the locations are flexible as long as they can be installed continuously on both floors). Door openings and mechanical penetrations through the walls are acceptable and can be detailed in the retrofit. New spread footings will be required below the new shear walls, similar to the configuration at the Courthouse.

Concrete Columns

The addition of concrete shear walls will greatly reduce the seismic forces in the existing columns. However, the columns will still require deformation compatibility with the expected seismic drift of the structure. Given the assumed lack of shear reinforcing in the columns, we propose to apply fiber-reinforced polymer (FRP) to the columns, which will greatly increase their ductility and accommodate the imposed lateral deflections.

The FRP would wrap completely around each column to provide confinement. We anticipate two to four layers of FRP at each column, and column corners will need to be chamfered to install the FRP. This may not be economically viable for the exterior columns, as the windows and waterproofing would have to be altered. Another option for the exterior columns would be to provide a secondary gravity system, such as tube steel columns, directly inside of each concrete column. This would provide redundancy in case of a large seismic event that could cause shear failures in the concrete columns and potentially affect the column's vertical load carrying capability.

Column locations are typically noted with a "B" symbol on Figure 9.

Precast Connections

See the information presented in the Courthouse section above.

Collector Strengthening

Collector strengthening, similar to that for the Courthouse, may also be required in the Annex.

Additional Cost Items

If a seismic retrofit is performed on the building, destructive testing of structural components will likely be required to verify the existing material strengths. The extent of the testing will be dependent on the jurisdiction's requirements as well as the availability of original construction drawings, but for estimating purposes, we anticipate approximately three concrete cores and two reinforcing steel coupons will be required to be removed from the existing structural components and tested.

Further Recommendations

The seismic retrofit scope for the Annex is based on the findings for the Courthouse. Original construction documents would be helpful in verifying the proposed scope for the Annex. If existing documentation is not available, extensive destructive and nondestructive testing may be required to determine the existing conditions.

In order to investigate the "unknown" items on the Tier 1 checklist, we recommend the development of geotechnical report including a geological hazards assessment.

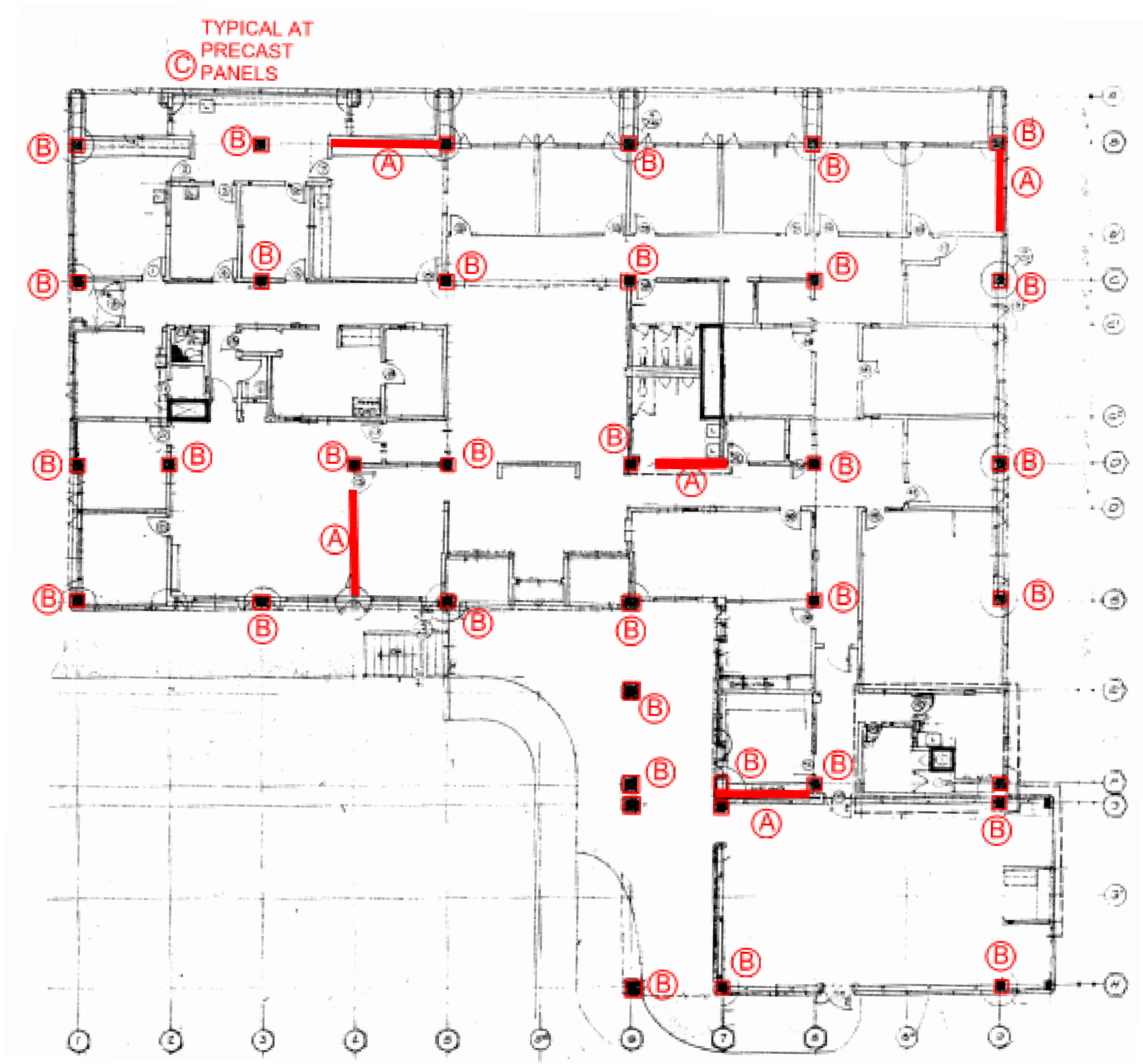


Figure 9: Typical floor plan at Annex indicating: A=proposed locations of retrofitted shear walls; B=concrete columns; C= exterior precast concrete façade panels

<u>Parking Garage</u>

The Parking Garage building has four deficiencies in accordance with the Tier 2 procedures of ASCE 41-13. The deficiency-only retrofit recommendations provide remediation of these deficiencies to meet a Life Safety performance level in response to a BSE-1E hazard.

Concrete Shear Walls

The main retrofit scope in the north-south direction is the addition of one 10-inch-thick concrete wall with two mats of #5 rebar at 18-inch spacing each way. The wall will be continuous from the grade to the roof of the structure (3 levels) and is placed such that no parking spaces will be affected. The wall is indicated in Figure 10 with an “A” symbol and is located across from the existing concrete shear wall on the uphill (east) side of the structure. Reinforcing from the wall above and below each floor will need to be doweled through the existing slabs at each level, as shown in Figure 11. The existing slab surface will be roughened to ¼” amplitude.

An approximate estimate of concrete and reinforcement quantities based on the retrofit solution analyzed in the Tier 2 evaluation is provided for each level below. This is based on a linear static, fixed-base analysis model.

Floor	**Volume of Concrete in New Shear Walls (cubic yards)**	**Weight of Reinforcing Steel (pounds per cubic yard of concrete)**
Ground	70	95
First	70	95
Second	70	95

Concrete Columns

Given the lack of shear reinforcing in the columns, we propose to apply two layers of fiber-reinforced polymer (FRP) to the second level columns and three layers to the first level columns, which will greatly increase their shear capacity, ductility, and accommodate the imposed lateral deflections. The FRP will wrap completely around the columns to provide confinement. The square columns will require all four corners to be chamfered prior to FRP application and the surface will need to be cleaned of corrosion.

Foundation

The new concrete shear wall will require a foundation consisting of three to six helical piles at each end of the wall (six to 12 helical piles total). A minimum 6-foot square pile cap with 36" depth will be required. Piers will be battered to resist lateral loads. The pile caps will be tied together with a minimum 36-inch-wide by 24-inch-deep concrete grade beam. Foundations will be designed per the recommendations of a geotechnical investigation and report.

An approximate estimate of foundation concrete and reinforcement quantities based on the retrofit solution analyzed in the Tier 2 evaluation is provided for each level below. This is based on a linear static, fixed base analysis model.

Volume of Concrete in New Footings	15 yd^3
Weight of Reinforcing Steel in New Footings	175 lbs/ yd^3
Helical Pile Quantity (Total)	6 to 12

Additional Cost Items

If a seismic retrofit is performed on the building, destructive testing of structural components will likely be required to verify the existing material strengths. The extent of the testing will be dependent on the jurisdiction requirements as well as the availability of original construction drawings, but for estimating purposes, we anticipate approximately six concrete cores and three reinforcing steel coupons will be required to be removed from the existing structural components and tested.

Because the structure is comprised of post-tensioned slabs and beams, all areas that require doweling or anchorage through the existing structure will required scanning to identify the location of the post-tensioning strands to avoid damage during construction.

Further Recommendations

In order to investigate the "unknown" items on the Tier 1 checklist and to determine the foundation design below the new shear wall, we recommend the development of geotechnical report including a geological hazards assessment.

Figure 10: Typical floor plan at Parking Garage indicating: A=proposed location of retrofitted shear wall; B=concrete columns; C=new footing (also shown as purple boxes)

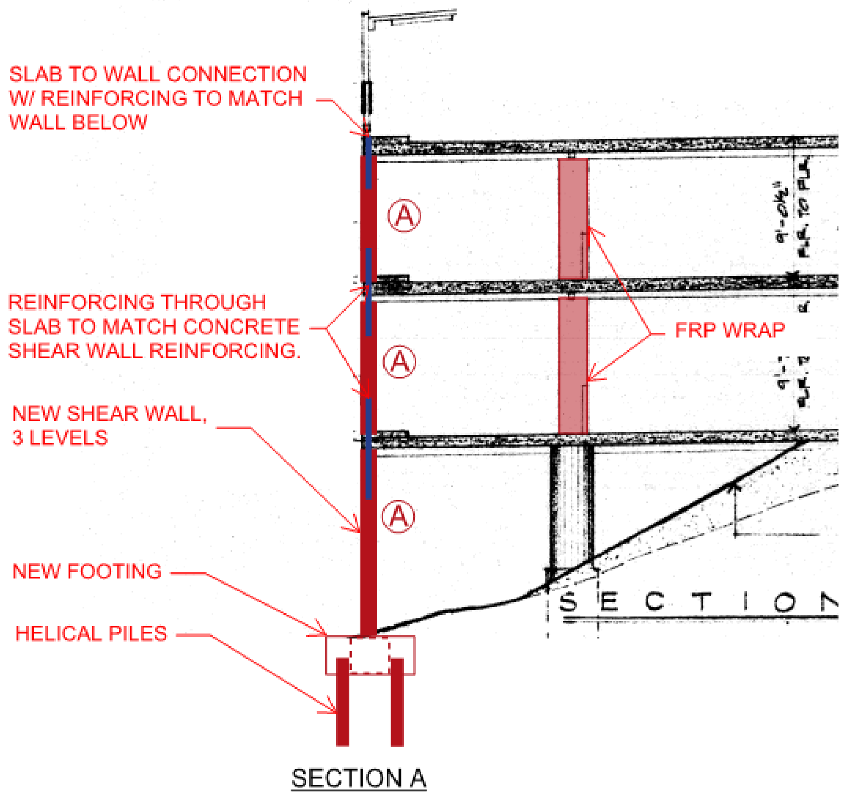


Figure 11: Section A through proposed new shear wall at the Parking Garage

RELIABILITY OF SEISMIC EVALUATIONS

In general, structural engineers do not have the ability to 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 occur at 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 life safety. This effort is based on science and technology as well as on observations made from actual seismic events. Building design and evaluation codes are constantly evolving to better meet performance targets based on this information. 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.

CLOSING

The seismic review and analyses associated with this evaluation were based on available original structural drawings, and the site reviews were based on that which was plainly visible. No attempt was made to uncover hidden conditions or perform any destructive or non-destructive testing. The items discussed in this report are subject to revision should more information become available.

This report is general in nature 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 building 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.

Sincerely,

Chelsea Drenick, SE
Senior Engineer

Ryan Bogart, SE
Senior Associate

Mark A. Moore, SE
Executive Principal

APPENDIX A – PHOTOGRAPHS

Photo 1: Courthouse Building Entrance

Photo 2: Courthouse Building East Elevation

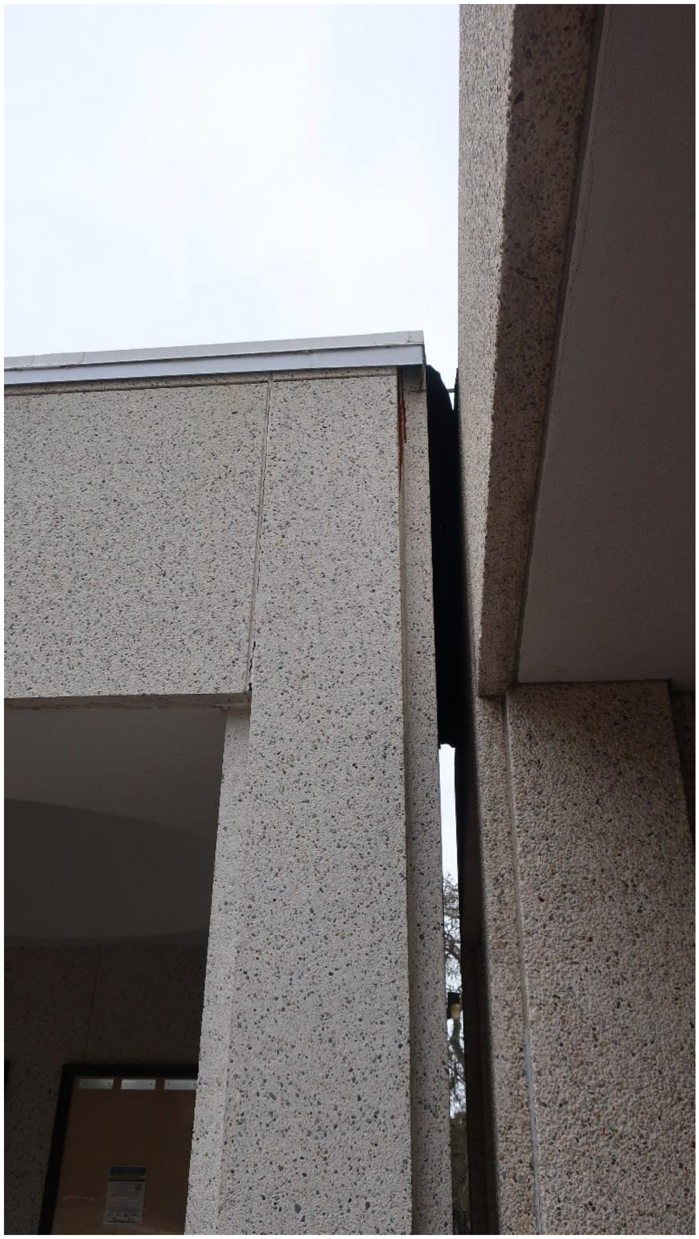


Photo 3: Seismic Gap Between Courthouse and Annex

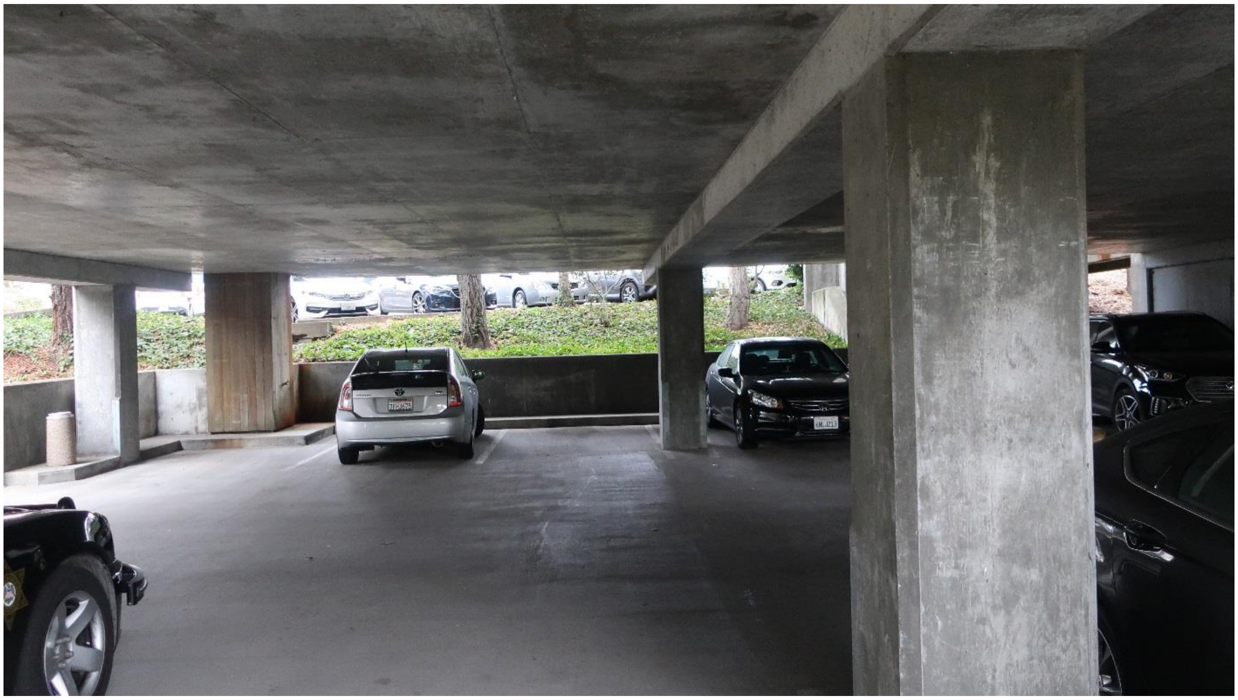


Photo 4: Parking Garage Interior

Photo 5: Parking Garage Foundation Below First Floor

Photo 6: Cracked Exterior Sidewalk Slab at the Courthouse

Photo 7: Spalled Concrete at Existing Column at the Annex

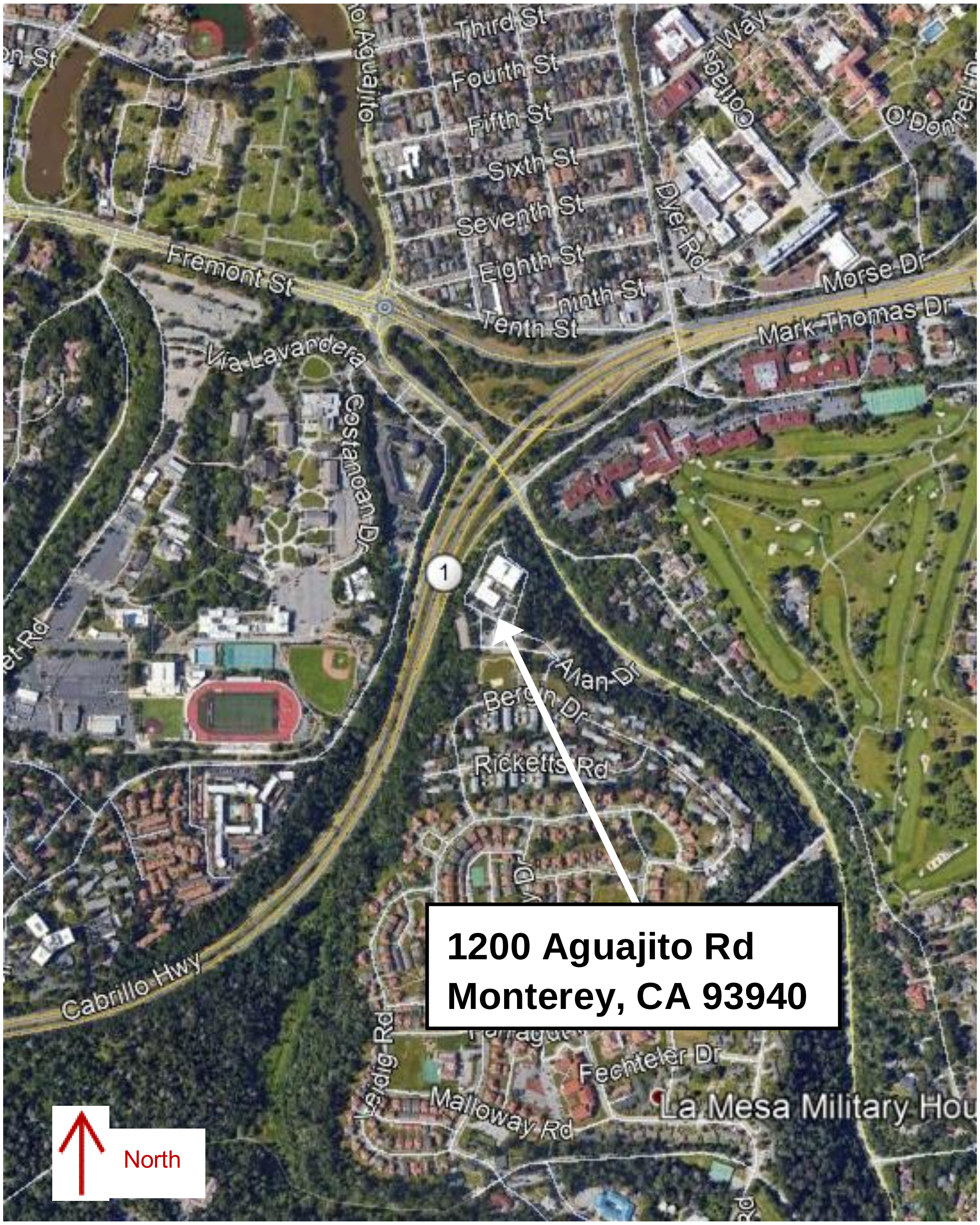
Photo 8: Cracked Concrete Beam and Wall at Annex Exterior

Photo 9: Corrosion at Parking Garage Guardrail

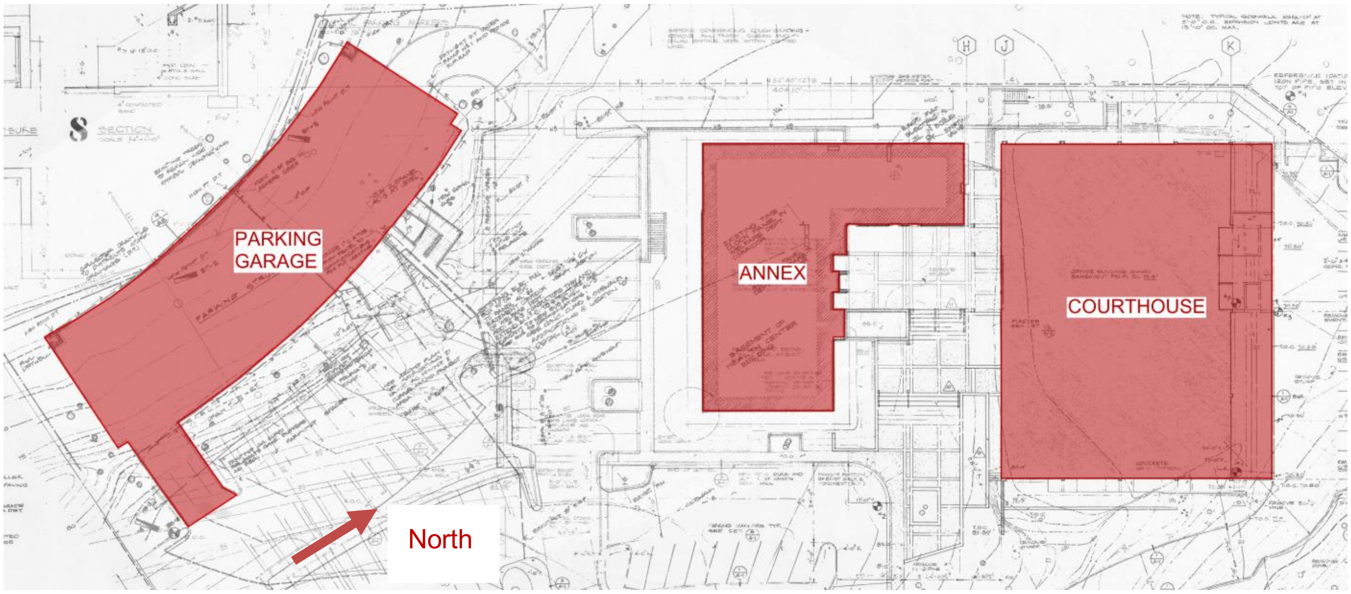
Photo 10: Deteriorated Wood at Parking Garage

APPENDIX B – MAPS

Location Map



Site Map



APPENDIX C – SUMMARY DATA SHEET

Summary Data Sheet - Courthouse

BUILDING DATA

Building Name:	Monterey County Courthouse	Date:	11/29/1966
Building Address:	1200 Aguajito Road, Monterey, CA 93940		
Latitude: 36.590	Longitude: -121.880	By:	Wallace Holm

Year Built:	1966	Year(s) Remodeled:	Unknown	Original Design Code:	Unknown (1964 UBC assumed)
Area (sf):	68000	Length (ft):	160 feet	Width (ft):	104 feet
No. of Stories:	4	Story Height:	~15 feet	Total Height:	60 feet

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

CONSTRUCTION DATA

Gravity Load Structural System:	Concrete columns, beams, slab		
Exterior Transverse Walls:	N/A	Openings?	
Exterior Longitudinal Walls:	N/A	Openings?	
Roof Materials/Framing:	Concrete beams and slab		
Intermediate Floors/Framing:	Concrete beams and slab		
Ground Floor:	Slab-on-grade		
Columns:	Concrete	Foundation:	Spread footings
General Condition of Structure:	Good		
Levels Below Grade?	Partial story below grade		

LATERAL-FORCE-RESISTING SYSTEM

	Longitudinal	Transverse
System:	Concrete moment frames	Concrete moment frames
Vertical Elements:	Concrete columns	Concrete columns
Diaphragms:	Concrete slab	Concrete slab

EVALUATION DATA

BSE-1N Spectral Response Accelerations:	S_{XS}=	1.009 g	S_{X1}=	0.555 g
Soil Factors:	Class=	D (assumed)		
BSE-1E Spectral Response Accelerations:	S_{XS}=	0.739 g	S_{X1}=	0.391 g
Level of Seismicity:		High	Performance Level:	Life Safety
Building Period:	T=	0.717 sec		
Spectral Acceleration:	S_a=	0.545 g		
Modification Factor:	$C_mC_1C_2$=	0.88	Building Weight: W=	10,660 kips
Pseudo Lateral Force:	$V=C_mC_1C_2S_aW$=	0.479W		

BUILDING CLASSIFICATION:	**C1 – Concrete Moment Frames**

COMPLETED TIER 1 CHECKLISTS	Yes	No
Basic Configuration Checklist	☑	☐
Building Type C1 Structural Checklist	☑	☐
Nonstructural Component Checklist	☐	☑

Material Properties

To account for uncertainty in the as-built data, a knowledge factor, κ, is determined according to ASCE 41 Table 6-1.

			Default Value per ASCE 41, 4.2.3?	*Alternate Value Source?*
Concrete			Table (4-2)	
Beams, Columns and Slabs:	f'_c=	4,000 psi	☐	Construction drawings
Slab-on-grade:	f'_c=	3,000 psi	☐	Construction drawings
Reinforcing Steel			Table (4-3)	
All Bars:	f_y=	40 or 50 ksi	☐	Construction drawings

Summary Data Sheet – Courthouse Parking Garage

BUILDING DATA

Building Name:	Monterey County Courthouse Parking Garage				
Building Address:	1200 Aguajito Road, Monterey, CA 93940				
Latitude:	36.590	Longitude:	-121.880		
Year Built:	1966	Year(s) Remodeled:	Unknown	Original Design Code:	Unknown (1964 UBC assumed)
Area (sf):	33000	Length (ft):	175 feet	Width (ft):	61 feet
No. of Stories:	3	Story Height:	~9.5 feet	Total Height:	28 feet

USE ☐ Industrial ☐ Office ☐ Warehouse ☐ Hospital ☐ Residential ☐ Educational ☑ Other: Parking Garage

CONSTRUCTION DATA

Gravity Load Structural System:	Concrete post-tensioned beams and slab; CIP columns		
Exterior Transverse Walls:	N/A	Openings?	
Exterior Longitudinal Walls:	N/A	Openings?	
Roof Materials/Framing:	Concrete post-tensioned beams and slab		
Intermediate Floors/Framing:	Concrete post-tensioned beams and slab		
Ground Floor:	Concrete post-tensioned beams and slab		
Columns:	CIP concrete	Foundation:	Concrete piers
General Condition of Structure:	Good		
Levels Below Grade?	Partial story below grade		

LATERAL-FORCE-RESISTING SYSTEM

	Longitudinal	Transverse
System:	Concrete slab/column moment frames/concrete shear wall	Concrete moment frames
Vertical Elements:	Concrete columns	Concrete columns
Diaphragms:	Post-tensioned concrete slab	Post-tensioned concrete slab

EVALUATION DATA

BSE-1N Spectral Response Accelerations:	S_{XS}=	1.009 g	S_{X1}=	0.555 g
Soil Factors:	Class=	D (assumed)		
BSE-1E Spectral Response Accelerations:	S_{XS}=	0.739 g	S_{X1}=	0.391 g
Level of Seismicity:		High	Performance Level:	Life Safety
Building Period:	T=	0.361 sec		
Spectral Acceleration:	S_a=	0.74 g		
Modification Factor:	$C_mC_1C_2$=	0.99	Building Weight: W=	3,829 kips
Pseudo Lateral Force:	$V=C_mC_1C_2S_aW$=	0.73W		

BUILDING CLASSIFICATION:	**C1 – Concrete Moment Frames**

COMPLETED TIER 1 CHECKLISTS	Yes	No
Basic Configuration Checklist	☑	☐
Building Type C1 Structural Checklist	☑	☐
Nonstructural Component Checklist	☐	☑

Material Properties

To account for uncertainty in the as-built data, a knowledge factor, κ, is determined according to ASCE 41 Table 6-1.

			Default Value per ASCE 41, 4.2.3?	*Alternate Value Source?*
Concrete			Table (4-2)	
Columns, Post-tensioned Beams and Slab:	f'_c=	4,000 psi	☐	Construction drawings
Lower story columns:	f'_c=	5,000 psi	☐	Construction drawings
All other concrete:	f'_c=	3,000 psi	☐	Construction drawings
Reinforcing Steel			Table (4-3)	
#5 and smaller in ramps and walls:	f_y=	40 ksi	☐	Construction drawings
All other bars:	f_y=	60 ksi	☐	Construction drawings
Post-tensioned tendons				
	f_u=	270 ksi		Construction drawings

APPENDIX D – TIER 1 CHECKLISTS

16.1.2LS Life Safety Basic Configuration Checklist

This Basic Configuration Checklist shall be completed for all building types, except buildings in very low seismicity, being evaluated to the Life Safety Performance Level. Once this checklist has been completed, complete the appropriate building type checklist for the desired seismic performance level as shown in Table 4-7. Tier 1 evaluation shall include on-site investigation and condition assessment as required by Section 4.2.1.

Each of the evaluation statements on this checklist shall be marked Compliant (C), Non-compliant (NC), Unknown (U), or Not Applicable (N/A) for a Tier 1 Evaluation. Compliant statements identify issues that are acceptable according to the criteria of this standard, while non-compliant and unknown statements identify issues that require further investigation. Certain statements may not apply to the buildings being evaluated. For non-compliant and unknown evaluation statements, the design professional may choose to conduct further investigation using the corresponding Tier 2 Evaluation procedure; corresponding section numbers are in parentheses following each evaluation statement.

C16.1.2LS Life Safety Basic Configuration Checklist

For buildings in low, moderate, and high seismicity the following evaluation statements represent general configuration issues applicable for most building based on observed earthquake structural damage during actual earthquakes. This checklist should be completed for all buildings in low, moderate, and high seismicity for Life Safety Performance Level.

The section numbers in parentheses following each evaluation statement refer to the commentary in Appendix A regarding the statement's purpose and the corresponding Tier 2 Evaluation procedures. If additional information on the evaluation statement is required, refer to the commentary in the Tier 2 procedure for that evaluation statement.

Low Seismicity

Building System

General

C	NC	N/A	U	
☑	☐	☐	☐	LOAD PATH: The structure shall contain a complete, well defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation. (Commentary: Sec. A.2.1.1. Tier 2: Sec. 5.4.1.1)
☐	☑	☐	☐	ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building is greater than 4% of the height of the shorter building. This statement shall not apply for the following building types: W1, W1A, and W2. (Commentary: Sec. A.2.1.2. Tier 2: Sec. 5.4.1.2) *The height of the annex is approximately 14', 4% of the height = 6.75". The gap between the two is approximately 2-3 inches*
☐	☐	☑	☐	MEZZANINES: Interior mezzanine levels are braced independently from the main structure or are anchored to the seismic-force-resisting elements of the main structure. (Commentary: Sec. A.2.1.3. Tier 2: Sec. 5.4.1.3) *There are no mezzanines shown on the existing drawings.*

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. (Commentary: Sec. A2.2.2. Tier 2: Sec. 5.4.2.1) *See calculations*
☑	☐	☐	☐	SOFT STORY: The stiffness of the seismic-force-resisting system in any story is not less than 70% of the seismic-force-resisting system stiffness in an adjacent story above or less than 80% of the average seismic-force-resisting system stiffness of the three stories above. (Commentary: Sec. A.2.2.3. Tier 2: Sec. 5.4.2.2) *See calculations*
☑	☐	☐	☐	VERTICAL IRREGULARITIES: All vertical elements in the seismic-force-resisting system are continuous to the foundation. (Commentary: Sec. A.2.2.4. Tier 2: Sec. 5.4.2.3) *All columns are continuous from footings up to the roof.*
☑	☐	☐	☐	GEOMETRY: There are no changes in the net horizontal dimension of the seismic-force-resisting system of more than 30% in a story relative to adjacent stories, excluding one-story penthouses and mezzanines. (Commentary: Sec. A.2.2.5. Tier 2: Sec. 5.4.2.4) *Building plan is regular at all stories.*
☑	☐	☐	☐	MASS: There is no change in effective mass more than 50% from one story to the next. Light roofs, penthouses, and mezzanines need not be considered. (Commentary: Sec. A.2.2.6. Tier 2: Sec. 5.4.2.5) *Building plans are similar in size and mass at all stories.*
☑	☐	☐	☐	TORSION: The estimated distance between the story center of mass and the story center of rigidity is less than 20% of the building width in either plan dimension. (Commentary: Sec. A.2.2.7. Tier 2: Sec. 5.4.2.6) *By inspection the building plans are regular and will have similar center of mass to center of rigidity.*

Medium 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 shall not exist in the foundation soils at depths within 50 ft under the building. (Commentary: Sec. A.6.1.1. Tier 2: 5.4.3.1) *Geotechnical information on the existing architectural drawings does not address liquefaction susceptibility. Review of the soil borings indicates the site is unlikely to experience liquefaction. However more information, or review by a geotechnical engineer is required.*
☐	☐	☐	☑	SLOPE FAILURE: The building site is sufficiently remote from potential earthquake-induced slope failures or rockfalls to be unaffected by such failures or is capable of accommodating any predicted movements without failure. (Commentary: Sec. A.6.1.2. Tier 2: 5.4.3.1) *No geotechnical information available*
☐	☐	☐	☑	SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site are not anticipated. (Commentary: Sec. A.6.1.3. Tier 2: 5.4.3.1) *No geotechnical information available*

High Seismicity: Complete the Following Items in Addition to the Items for Low and 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 $0.6S_a$. (Commentary: Sec. A.6.2.1. Tier 2: Sec. 5.4.3.3) *See calculations.*
☑	☐	☐	☐	TIES BETWEEN FOUNDATION ELEMENTS: The foundation has ties adequate to resist seismic forces where footings, piles, and piers are not restrained by beams, slabs, or soils classified as Site Class A, B, or C. (Commentary: Sec. A.6.2.2. Tier 2: Sec. 5.4.3.4) *Ties are not present, but site class is unknown. Given geotechnical borings showing the soil as weathered shale it is possible it will be site class C and will restrain the footings.*

16.9LS Building Type C1

Life Safety Structural Checklist For Building Type C1: Concrete Moment Frames

This Life Safety Structural Checklist shall be completed where required by Table 4-7 and where the building configuration complies with the description of C1 building type defined in Table 3-1. Tier 1 evaluation shall include on-site investigation and condition assessment as required by Section 4.2.1.

Each of the evaluation statements on this checklist shall be marked Compliant (C), Non-compliant (NC), Not Applicable (N/A), or Unknown (U) for a Tier 1 Evaluation. Compliant statements identify issues that are acceptable according to the criteria of this standard, while non-compliant and unknown statements identify issues that require further investigation. Certain statements may not apply to the buildings being evaluated. For non-compliant and unknown evaluation statements, the design professional may choose to conduct further investigation using the corresponding Tier 2 Evaluation procedure; corresponding section numbers are in parentheses following each evaluation statement.

C16. 9LS Life Safety Structural Checklist for Building Type C1

Building Type C1

These buildings consist of a frame assembly of cast-in-place concrete beams and columns. Floor and roof framing consists of cast-in-place concrete slabs, concrete beams, one-way joists, two-way waffle joists, or flat slabs. Seismic forces are resisted by concrete moment frames that develop their stiffness through monolithic beam-column connections. In older construction, or in levels of low seismicity, the moment frames may consist of the column strips of two-way flat slab systems. Modern frames in levels of high seismicity have joint reinforcing, closely spaced ties, and special detailing to provide ductile performance. This detailing is not present in older construction. Foundations consist of concrete spread footings, mat foundations, or deep foundations.

Refer to Sections A.3.1 and A.3.1.4 for additional commentary related to concrete moment frames.

Low Seismicity

Seismic-Force-Resisting System

C	NC	N/A	U	
☑	☐	☐	☐	REDUNDANCY: The number of lines of moment frames in each principal direction is greater than or equal to 2. The number of bays of moment frames in each line is greater than or equal to 2. (Commentary: Sec. A.3.1.1.1. Tier 2: Sec. 5.5.1.1) *There are 5 existing frame lines in each direction*
☑	☐	☐	☐	COLUMN AXIAL STRESS CHECK: The axial stress caused by unfactored gravity loads in columns subjected to overturning forces because of seismic demands is less than $0.20f'_c$. Alternatively, the axial stress caused by overturning forces alone, calculated using the Quick Check procedure of Section 4.5.3.6, is less than $0.30f'_c$. (Commentary: Sec. A.3.1.4.2. Tier 2: Sec. 5.5.2.1.3) *See supplemental calculations, maximum axial stress is 310 psi and allowable is 900 psi*

Connections

C	NC	N/A	U

☑ ☐ ☐ ☐ CONCRETE COLUMNS: All concrete columns are doweled into the foundation with a minimum of 4 bars. (Commentary: Sec. A.5.3.2. Tier 2: Sec. 5.7.3.1)
The existing drawings show (12) #11 bar dowels at each column

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

Seismic-Force-Resisting System

C NC N/A U

☐ ☐ ☑ ☐ INTERFERING WALLS: All concrete and masonry infill walls placed in moment frames are isolated from structural elements. (Commentary: Sec. A.3.1.2.1. Tier 2: Sec. 5.5.2.1.1)
No infill concrete or masonry walls appear to be present based on existing drawings

☐ ☑ ☐ ☐ COLUMN SHEAR STRESS CHECK: The shear stress in the concrete columns, calculated using the Quick Check procedure of Section 4.5.3.2, is less than the greater of 100 lb/in.^{2} or $2\sqrt{f'_c}$. (Commentary: Sec. A.3.1.4.1. Tier 2: Sec. 5.5.2.1.4)
See calculations.

☑ ☐ ☐ ☐ FLAT SLAB FRAMES: The seismic-force-resisting system is not a frame consisting of columns and a flat slab or plate without beams. (Commentary: Sec. A.3.1.4.3. Tier 2: Sec. 5.5.2.3.1)
Building system has girders and beams.

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

☐ ☐ ☑ ☐ PRESTRESSED FRAME ELEMENTS: The seismic-force-resisting frames do not include any prestressed or posttensioned elements where the average prestress exceeds the lesser of 700 lb/in.^{2} or $f'_c/6$ at potential hinge locations. The average prestress is calculated in accordance with the Quick Check procedure of Section 4.5.3.8. (Commentary: Sec. A.3.1.4.4. Tier 2: Sec. 5.5.2.3.2)

☑ ☐ ☐ ☐ CAPTIVE COLUMNS: There are no columns at a level with height/depth ratios less than 50% of the nominal height/depth ratio of the typical columns at that level. (Commentary: Sec. A.3.1.4.5. Tier 2: Sec. 5.5.2.3.3)
No captive columns based on existing drawings.

☐ ☑ ☐ ☐ NO SHEAR FAILURES: The shear capacity of frame members is able to develop the moment capacity at the ends of the members. (Commentary: Sec. A.3.1.4.6. Tier 2: Sec. 5.5.2.3.4)
See calculations.

☐ ☑ ☐ ☐ STRONG COLUMN—WEAK BEAM: The sum of the moment capacity of the columns is 20% greater than that of the beams at frame joints. (Commentary: Sec. A.3.1.4.7. Tier 2: Sec. 5.5.2.1.5)
See calculations.

☐ ☑ ☐ ☐ BEAM BARS: At least two longitudinal top and two longitudinal bottom bars extend continuously throughout the length of each frame beam. At least 25% of the longitudinal bars provided at the joints for either positive or negative moment are continuous throughout the length of the members. (Commentary: A.3.1.4.8. Tier 2: Sec. 5.5.2.3.5)
See calculations.

☐ ☑ ☐ ☐ COLUMN-BAR SPLICES: All column-bar lap splice lengths are greater than $35d_b$ and are enclosed by ties spaced at or less than $8d_b$. Alternatively, column bars are spliced with mechanical couplers with a capacity of at least 1.25 times the nominal yield strength of the spliced bar. (Commentary: Sec. A.3.1.4.9. Tier 2: Sec. 5.5.2.3.6)

See calculations.

☐ ☑ ☐ ☐ BEAM-BAR SPLICES: The lap splices or mechanical couplers for longitudinal beam reinforcing are not located within $l_b/4$ of the joints and are not located in the vicinity of potential plastic hinge locations. (Commentary: Sec. A.3.1.4.10. Tier 2: Sec. 5.5.2.3.6)

See calculations.

☐ ☑ ☐ ☐ COLUMN-TIE SPACING: Frame columns have ties spaced at or less than d/4 throughout their length and at or less than $8d_b$ at all potential plastic hinge locations. (Commentary: Sec. A.3.1.4.11. Tier 2: Sec. 5.5.2.3.7)

See calculations.

☐ ☑ ☐ ☐ STIRRUP SPACING: All beams have stirrups spaced at or less than d/2 throughout their length. At potential plastic hinge locations, stirrups are spaced at or less than the minimum of $8d_b$ or d/4. (Commentary: Sec. A.3.1.4.12. Tier 2: Sec. 5.5.2.3.7)

See calculations.

☐ ☑ ☐ ☐ JOINT TRANSVERSE REINFORCING: Beam–column joints have ties spaced at or less than $8d_b$. (Commentary: Sec. A.3.1.4.13. Tier 2: Sec. 5.5.2.3.8)

See calculations.

☐ ☑ ☐ ☐ DEFLECTION COMPATIBILITY: Secondary components have the shear capacity to develop the flexural strength of the components. (Commentary: Sec. A.3.1.6.2. Tier 2: Sec. 5.5.2.5.2)

Precast concrete panel façade connections are nonductile and not designed to resist the forces generated when the structure experiences lateral movement from seismic forces.

☐ ☐ ☑ ☐ FLAT SLABS: Flat slabs or plates not part of the seismic-force-resisting system have continuous bottom steel through the column joints. (Commentary: Sec. A.3.1.6.3. Tier 2: Sec. 5.5.2.5.3)

The floor throughout the building is concrete over metal deck with only one layer of reinforcement.

Diaphragms

C	NC	N/A	U	
☑	☐	☐	☐	DIAPHRAGM CONTINUITY: The diaphragms are not composed of split-level floors and do not have expansion joints. (Commentary: Sec. A.4.1.1. Tier 2: Sec. 5.6.1.1)

Connections

C	NC	N/A	U	
☐	☐	☑	☐	UPLIFT AT PILE CAPS: Pile caps have top reinforcement, and piles are anchored to the pile caps. (Commentary: Sec. A.5.3.8. Tier 2: Sec. 5.7.3.5)

16.1.2LS Life Safety Basic Configuration Checklist

This Basic Configuration Checklist shall be completed for all building types, except buildings in very low seismicity, being evaluated to the Life Safety Performance Level. Once this checklist has been completed, complete the appropriate building type checklist for the desired seismic performance level as shown in Table 4-7. Tier 1 evaluation shall include on-site investigation and condition assessment as required by Section 4.2.1.

Each of the evaluation statements on this checklist shall be marked Compliant (C), Non-compliant (NC), Unknown (U), or Not Applicable (N/A) for a Tier 1 Evaluation. Compliant statements identify issues that are acceptable according to the criteria of this standard, while non-compliant and unknown statements identify issues that require further investigation. Certain statements may not apply to the 34buildings being evaluated. For non-compliant and unknown evaluation statements, the design professional may choose to conduct further investigation using the corresponding Tier 2 Evaluation procedure; corresponding section numbers are in parentheses following each evaluation statement.

C16.1.2LS Life Safety Basic Configuration Checklist

For buildings in low, moderate, and high seismicity the following evaluation statements represent general configuration issues applicable for most building based on observed earthquake structural damage during actual earthquakes. This checklist should be completed for all buildings in low, moderate, and high seismicity for Life Safety Performance Level.

The section numbers in parentheses following each evaluation statement refer to the commentary in Appendix A regarding the statement's purpose and the corresponding Tier 2 Evaluation procedures. If additional information on the evaluation statement is required, refer to the commentary in the Tier 2 procedure for that evaluation statement.

Low Seismicity

Building System

General

C	NC	N/A	U	
☑	☐	☐	☐	LOAD PATH: The structure shall contain a complete, well defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation. (Commentary: Sec. A.2.1.1. Tier 2: Sec. 5.4.1.1)
☑	☐	☐	☐	ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building is greater than 4% of the height of the shorter building. This statement shall not apply for the following building types: W1, W1A, and W2. (Commentary: Sec. A.2.1.2. Tier 2: Sec. 5.4.1.2)
☐	☐	☑	☐	MEZZANINES: Interior mezzanine levels are braced independently from the main structure or are anchored to the seismic-force-resisting elements of the main structure. (Commentary: Sec. A.2.1.3. Tier 2: Sec. 5.4.1.3)

Building Configuration

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. (Commentary: Sec. A2.2.2. Tier 2: Sec. 5.4.2.1) *By inspection of plans and elevations, a weak story condition does not exist.*
☑	☐	☐	☐	SOFT STORY: The stiffness of the seismic-force-resisting system in any story is not less than 70% of the seismic-force-resisting system stiffness in an adjacent story above or less than 80% of the average seismic-force-resisting system stiffness of the three stories above. (Commentary: Sec. A.2.2.3. Tier 2: Sec. 5.4.2.2) *By inspection of plans and elevations, a soft story condition does not exist.*
☑	☐	☐	☐	VERTICAL IRREGULARITIES: All vertical elements in the seismic-force-resisting system are continuous to the foundation. (Commentary: Sec. A.2.2.4. Tier 2: Sec. 5.4.2.3)
☑	☐	☐	☐	GEOMETRY: There are no changes in the net horizontal dimension of the seismic-force-resisting system of more than 30% in a story relative to adjacent stories, excluding one-story penthouses and mezzanines. (Commentary: Sec. A.2.2.5. Tier 2: Sec. 5.4.2.4)
☑	☐	☐	☐	MASS: There is no change in effective mass more than 50% from one story to the next. Light roofs, penthouses, and mezzanines need not be considered. (Commentary: Sec. A.2.2.6. Tier 2: Sec. 5.4.2.5)
☐	☑	☐	☐	TORSION: The estimated distance between the story center of mass and the story center of rigidity is less than 20% of the building width in either plan dimension. (Commentary: Sec. A.2.2.7. Tier 2: Sec. 5.4.2.6) *The parking structure has a single concrete shear wall in the east-west direction along the south end of the structure. The building is torsionally irregular.*

Medium 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 shall not exist in the foundation soils at depths within 50 ft under the building. (Commentary: Sec. A.6.1.1. Tier 2: 5.4.3.1) *Geotechnical information on the existing architectural drawings does not address liquefaction susceptibility. Review of the soil borings indicates the site is unlikely to experience liquefaction. However more information, or review by a geotechnical engineer is required.*
☐	☐	☐	☑	SLOPE FAILURE: The building site is sufficiently remote from potential earthquake-induced slope failures or rockfalls to be unaffected by such failures or is capable of accommodating any predicted movements without failure. (Commentary: Sec. A.6.1.2. Tier 2: 5.4.3.1) *Geotechnical information on the existing architectural drawings does not address slope failure, however the site is on a significant slope so this may be a concern.*
☐	☐	☐	☑	SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site are not anticipated. (Commentary: Sec. A.6.1.3. Tier 2: 5.4.3.1) *No geotechnical information available*

High Seismicity: Complete the Following Items in Addition to the Items for Low and 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 $0.6S_a$. (Commentary: Sec. A.6.2.1. Tier 2: Sec. 5.4.3.3)

See calculations

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

Ties are not present, but site class is unknown. Geotechnical information is required.

16.9LS Building Type C1

Life Safety Structural Checklist For Building Type C1: Concrete Moment Frames

This Life Safety Structural Checklist shall be completed where required by Table 4-7 and where the building configuration complies with the description of C1 building type defined in Table 3-1. Tier 1 evaluation shall include on-site investigation and condition assessment as required by Section 4.2.1.

Each of the evaluation statements on this checklist shall be marked Compliant (C), Non-compliant (NC), Not Applicable (N/A), or Unknown (U) for a Tier 1 Evaluation. Compliant statements identify issues that are acceptable according to the criteria of this standard, while non-compliant and unknown statements identify issues that require further investigation. Certain statements may not apply to the buildings being evaluated. For non-compliant and unknown evaluation statements, the design professional may choose to conduct further investigation using the corresponding Tier 2 Evaluation procedure; corresponding section numbers are in parentheses following each evaluation statement.

C16. 9LS Life Safety Structural Checklist for Building Type C1

Building Type C1

These buildings consist of a frame assembly of cast-in-place concrete beams and columns. Floor and roof framing consists of cast-in-place concrete slabs, concrete beams, one-way joists, two-way waffle joists, or flat slabs. Seismic forces are resisted by concrete moment frames that develop their stiffness through monolithic beam-column connections. In older construction, or in levels of low seismicity, the moment frames may consist of the column strips of two-way flat slab systems. Modern frames in levels of high seismicity have joint reinforcing, closely spaced ties, and special detailing to provide ductile performance. This detailing is not present in older construction. Foundations consist of concrete spread footings, mat foundations, or deep foundations.

Refer to Sections A.3.1 and A.3.1.4 for additional commentary related to concrete moment frames.

Low Seismicity

Seismic-Force-Resisting System

C	NC	N/A	U	
☑	☐	☐	☐	REDUNDANCY: The number of lines of moment frames in each principal direction is greater than or equal to 2. The number of bays of moment frames in each line is greater than or equal to 2. (Commentary: Sec. A.3.1.1.1. Tier 2: Sec. 5.5.1.1) *There are 2 lines in the E-W direction and 8 lines in the N-S direction.*
☑	☐	☐	☐	COLUMN AXIAL STRESS CHECK: The axial stress caused by unfactored gravity loads in columns subjected to overturning forces because of seismic demands is less than 0.20f'_c. Alternatively, the axial stress caused by overturning forces alone, calculated using the Quick Check procedure of Section 4.5.3.6, is less than 0.30f'_c. (Commentary: Sec. A.3.1.4.2. Tier 2: Sec. 5.5.2.1.3) *See supplemental calculations, check is compliant.*

Connections

C NC N/A U

☑ ☐ ☐ ☐ CONCRETE COLUMNS: All concrete columns are doweled into the foundation with a minimum of 4 bars. (Commentary: Sec. A.5.3.2. Tier 2: Sec. 5.7.3.1)

See S1, (4) dowels from column to foundation.

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

Seismic-Force-Resisting System

C	NC	N/A	U	
☐	☐	☑	☐	INTERFERING WALLS: All concrete and masonry infill walls placed in moment frames are isolated from structural elements. (Commentary: Sec. A.3.1.2.1. Tier 2: Sec. 5.5.2.1.1) *More field investigation is required to determine if infill walls are isolated from the column and beam system.*
☐	☑	☐	☐	COLUMN SHEAR STRESS CHECK: The shear stress in the concrete columns, calculated using the Quick Check procedure of Section 4.5.3.2, is less than the greater of 100 lb/in.^{2} or $2\sqrt{f'_c}$. (Commentary: Sec. A.3.1.4.1. Tier 2: Sec. 5.5.2.1.4) *See calculations.*
☐	☑	☐	☐	FLAT SLAB FRAMES: The seismic-force-resisting system is not a frame consisting of columns and a flat slab or plate without beams. (Commentary: Sec. A.3.1.4.3. Tier 2: Sec. 5.5.2.3.1) *There are beams in one direction, but in the other direction it is prestressed slab without beams.*

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	
☑	☐	☐	☐	PRESTRESSED FRAME ELEMENTS: The seismic-force-resisting frames do not include any prestressed or posttensioned elements where the average prestress exceeds the lesser of 700 lb/in.^{2} or $f'_c/6$ at potential hinge locations. The average prestress is calculated in accordance with the Quick Check procedure of Section 4.5.3.8. (Commentary: Sec. A.3.1.4.4. Tier 2: Sec. 5.5.2.3.2) *See calculations*
☑	☐	☐	☐	CAPTIVE COLUMNS: There are no columns at a level with height/depth ratios less than 50% of the nominal height/depth ratio of the typical columns at that level. (Commentary: Sec. A.3.1.4.5. Tier 2: Sec. 5.5.2.3.3) *All ramps are exterior to the structure. There are columns supporting the ramps that are less than 50% of nominal height of typical columns, but the ramps are separated by expansion joints from the structure and therefore are not analyzed as part of the structure.*
☐	☑	☐	☐	NO SHEAR FAILURES: The shear capacity of frame members is able to develop the moment capacity at the ends of the members. (Commentary: Sec. A.3.1.4.6. Tier 2: Sec. 5.5.2.3.4) *See calculations.*
☐	☑	☐	☐	STRONG COLUMN—WEAK BEAM: The sum of the moment capacity of the columns is 20% greater than that of the beams at frame joints. (Commentary: Sec. A.3.1.4.7. Tier 2: Sec. 5.5.2.1.5) *See calculations.*
☑	☐	☐	☐	BEAM BARS: At least two longitudinal top and two longitudinal bottom bars extend continuously throughout the length of each frame beam. At least 25% of the longitudinal bars provided at the joints for either positive or negative moment are continuous throughout the length of the members. (Commentary: A.3.1.4.8. Tier 2: Sec. 5.5.2.3.5) *See beam section on existing drawing sheet S1, (2)#8 continuous top and bottom.*

☑ ☐ ☐ ☐ COLUMN-BAR SPLICES: All column-bar lap splice lengths are greater than $35d_b$ and are enclosed by ties spaced at or less than $8d_b$. Alternatively, column bars are spliced with mechanical couplers with a capacity of at least 1.25 times the nominal yield strength of the spliced bar. (Commentary: Sec. A.3.1.4.9. Tier 2: Sec. 5.5.2.3.6)

Column splices at the 1st level are compliant. No other column splices occur per sheet S1.

☑ ☐ ☐ ☐ BEAM-BAR SPLICES: The lap splices or mechanical couplers for longitudinal beam reinforcing are not located within $l_b/4$ of the joints and are not located in the vicinity of potential plastic hinge locations. (Commentary: Sec. A.3.1.4.10. Tier 2: Sec. 5.5.2.3.6)

Lap splices are located lb/4 from center of joint, see sheet S1.

☐ ☑ ☐ ☐ COLUMN-TIE SPACING: Frame columns have ties spaced at or less than d/4 throughout their length and at or less than $8d_b$ at all potential plastic hinge locations. (Commentary: Sec. A.3.1.4.11. Tier 2: Sec. 5.5.2.3.7)

See calculations, some locations are compliant, but others fail. Will need further investigation in tier 2.

☐ ☑ ☐ ☐ STIRRUP SPACING: All beams have stirrups spaced at or less than d/2 throughout their length. At potential plastic hinge locations, stirrups are spaced at or less than the minimum of $8d_b$ or d/4. (Commentary: Sec. A.3.1.4.12. Tier 2: Sec. 5.5.2.3.7)

Hinge zone is compliant but the typical is noncompliant.

☐ ☑ ☐ ☐ JOINT TRANSVERSE REINFORCING: Beam–column joints have ties spaced at or less than $8d_b$. (Commentary: Sec. A.3.1.4.13. Tier 2: Sec. 5.5.2.3.8)

See calculations.

☐ ☐ ☑ ☐ DEFLECTION COMPATIBILITY: Secondary components have the shear capacity to develop the flexural strength of the components. (Commentary: Sec. A.3.1.6.2. Tier 2: Sec. 5.5.2.5.2)

☐ ☐ ☑ ☐ FLAT SLABS: Flat slabs or plates not part of the seismic-force-resisting system have continuous bottom steel through the column joints. (Commentary: Sec. A.3.1.6.3. Tier 2: Sec. 5.5.2.5.3)

The post-tensioned slab is inherently part of the seismic force resisting system. There are no flat slabs or plates not part of the system.

Diaphragms

C NC N/A U

☑ ☐ ☐ ☐ DIAPHRAGM CONTINUITY: The diaphragms are not composed of split-level floors and do not have expansion joints. (Commentary: Sec. A.4.1.1. Tier 2: Sec. 5.6.1.1)

The diaphragms are continuous at each floor level. The expansion joints occur at the ramps that are not attached to the structure.

Connections

C NC N/A U

☐ ☐ ☑ ☐ UPLIFT AT PILE CAPS: Pile caps have top reinforcement, and piles are anchored to the pile caps. (Commentary: Sec. A.5.3.8. Tier 2: Sec. 5.7.3.5)

APPENDIX E – STRUCTURAL CALCULATIONS

County of Monterey Courthouse Structural Calculations

Monterey, California

ZFA Project Number: 17661

Final Report

June 30, 2018

Prepared For:

RIM Architects

San Francisco, California

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STRUCTURAL NARRATIVE

The following calculations support the ASCE 41-13 Tier 1 and Tier 2 checks for the County of Monterey County Courthouse and parking garage. Deficiencies remain after the Tier 2 evaluation. A Tier 2 deficiency based retrofit is then designed. Concrete shear walls are added to reduce seismic load to the existing structure and mitigate deficiencies to achieve compliance with Life Safety at BSE-1E.

DETAILED DESIGN CRITERIA

BUILDING CODE

Governing Code:	2016 California Building Code
Authority Having Jurisdiction:	City of Monterey

BUILDING SYSTEM DESCRIPTION

Date of Construction:	1966
No. Stories:	4
Footprint:	17000 ft^2
Floor Area:	68000 ft^2
Roof Area:	17000 ft^2
Mean Building Height:	60 ft
Roof Pitch:	0.00 :12

Building Use:	Courthouse
Gravity System:	Concrete beams and columns
Diaphragm System:	Concrete slab over metal deck
Foundation System:	Concrete Footings

SEISMIC DESIGN PARAMETERS

ASCE 41-13 Reference UNO:

Latitude:	36.590 deg	Longitude:	-121.880 deg	
Soil Site Class =	D	Per Geotech Report, Site Class D otherwise		*USGS*
Risk Category:	II			*Section 2.4.1.6.1*
Diaphragm=	Rigid Diaphragm	Concrete slab over metal deck		
Building System, N-S:	C1	Concrete Moment Frame		*Table 3-1*
Building System, E-W:	C1	Concrete Moment Frame		*Table 3-1*
$C_{t,\ N\text{-}S}$ =	0.02	Approximate Period Parameter, Ct, N-S		*Section 4.5.2.4*
$C_{t,\ E\text{-}W}$ =	0.02	Approximate Period Parameter, Ct, E-W		*Section 4.5.2.4*
$\beta_{N\text{-}S}$ =	0.90	Approximate Period Parameter, β, N-S		*Section 4.5.2.4*
$\beta_{E\text{-}W}$ =	0.90	Approximate Period Parameter, β, E-W		*Section 4.5.2.4*
$T_{a,\ N\text{-}S}$=	0.717 sec	Approximate Fundamental Period, N-S		*Section 4.5.2.4*
$T_{a,\ E\text{-}W}$ =	0.717 sec	Approximate Fundamental Period, E-W		*Section 4.5.2.4*

TIER 1 SEISMIC EVALUATION PARAMETERS

Performance Objective:	LS	Life Safety	*Table 2-1*
Seismic Hazard Level:	BSE-1E	20%/50 years	*Table 2-1*
S_S =	0.541 g	Mapped spectral response acceleration parameter	*USGS*
S_1 =	0.193 g	Mapped spectral response acceleration parameter	*USGS*
S_{XS} =	0.739 g	Mapped spectral response acceleration parameter	*USGS*
S_{X1} =	0.391 g	Mapped spectral response acceleration parameter	*ASCE 7 Table 1.5-1*
Seismicity:	High		*Table 2-5*
$S_{a,\ N\text{-}S}$ =	0.545 g	Spectral Response Acceleration, N-S	*Section 4.5.2.3*
$S_{a,\ E\text{-}W}$ =	0.545 g	Spectral Response Acceleration, E-W	*Section 4.5.2.3*
$C_{N\text{-}S}$ =	1.000	Modification Factor	*Table 4-8*
$C_{E\text{-}W}$ =	1.000	Modification Factor	*Table 4-8*
$V_{N\text{-}S}$ =	0.545 *W	Pseudo-Seismic Base Shear, N-S	*Section 4.5.2.1*
$V_{E\text{-}W}$ =	0.545 *W	Pseudo-Seismic Base Shear, N-S	*Section 4.5.2.1*

TIER 2 / 3 SEISMIC EVALUATION PARAMETERS

Performance Objective:	LS	Life Safety	*Table 2-1*
Seismic Hazard Level:	BSE-1E	20%/50 years	*Table 2-1*
S_S =	0.541 g	Mapped spectral response acceleration parameter	*USGS*
S_1 =	0.193 g	Mapped spectral response acceleration parameter	*USGS*
S_{XS} =	0.739 g	Mapped spectral response acceleration parameter	*USGS*
S_{X1} =	0.391 g	Mapped spectral response acceleration parameter	*USGS*
Seismicity:	High		*Table 2-5*
$C_{1,\ N\text{-}S}$ =	1.000	Inelastic-to-elastic displacement factor	*Equation 7-22*
$C_{2,\ N\text{-}S}$ =	1.000	Hysteresis shape factor	*Equation 7-23*
Alternate $(C_1C_2)_{N\text{-}S}$ =	1.100	2 ≤ mmax < 6	*Table 7-3*
Use Alternate $(C_1C_2)_{N\text{-}S}$?	Yes		
$(C_1C_2)_{N\text{-}S}$ =	1.100		
$C_{m,\ N\text{-}S}$ =	0.800	Effective mass factor	*Table 7-4*
$C_{1,\ E\text{-}W}$ =	1.000	Inelastic-to-elastic displacement factor	*Equation 7-22*
$C_{2,\ E\text{-}W}$ =	1.000	Hysteresis shape factor	*Equation 7-23*
Alternate $(C_1C_2)_{E\text{-}W}$ =	1.100	2 ≤ mmax < 6	*Table 7-3*
Use Alternate $(C_1C_2)_{E\text{-}W}$?	Yes		
$(C_1C_2)_{E\text{-}W}$ =	1.100		
$C_{m,\ E\text{-}W}$ =	0.800	Effective mass factor	*Table 7-4*
$S_{a,N\text{-}S}$ =	0.544	Spectral Response Acceleration	*Equation 2-9*
$S_{a,E\text{-}W}$ =	0.544	Spectral Response Acceleration	*Equation 2-9*
$V_{N\text{-}S}$ (ULT) =	0.479 *W	Pseudo-Seismic Base Shear, N-S	*Equation 7-21*
$V_{E\text{-}W}$ (ULT) =	0.479 *W	Pseudo-Seismic Base Shear, E-W	*Equation 7-21*

MATERIAL STRENGTH AND SPECIFICATIONS

ASCE 41-13 Reference UNO:

CONCRETE:

Knowledge Factor, κ	0.75	Concrete Knowledge Factor	*Table 6-1*
Foundations, f'_c =	3000 psi	Default Lower Bound: 2500 psi - 3000 psi	*Table 10-2*
Foundations, f'_{ce} =	4500 psi		*Table 10-1*
Slab on grade, f'_c =	3000 psi	Default Lower Bound: 3000 psi - 4000 psi	*Table 10-2*
Slab on grade, f'_{ce} =	4500 psi		*Table 10-1*
Structural walls, f'_c =	3000 psi	Default Lower Bound: 2500 psi - 4000 psi	*Table 10-2*
Structural walls, f'_{ce} =	4500 psi		*Table 10-1*
Beams, f'_c =	4000 psi	Default Lower Bound: 3000 psi - 4000 psi	*Table 10-2*
Beams, f'_{ce} =	6000 psi		*Table 10-1*
Columns, f'_c =	4000 psi	Default Lower Bound: 3000 psi - 6000 psi	*Table 10-2*
Columns, f'_{ce} =	6000 psi		*Table 10-1*
Fill over metal deck, f'_c =	4000 psi	Default Lower Bound: 3000 psi - 4000 psi	*Table 10-2*
Fill over metal deck, f'_{ce} =	6000 psi		*Table 10-1*
Elevated slabs, f'_c =	4000 psi	Default Lower Bound: 3000 psi - 4000 psi	*Table 10-2*
Elevated slabs, f'_{ce} =	6000 psi		*Table 10-1*
Weight of Concrete =	150 pcf		
Weight of Light Wt. Concrete =	110 pcf		

CONCRETE REINFORCING:

Knowledge Factor, κ	0.75	Reinforcing Knowledge Factor	*Table 6-1*
Reinforcing Steel, f_y =	40 ksi	Default Lower-Bound: 33, 40, 50, 60, 65, 70 ksi	*Table 10-3*
Reinforcing Steel, f_{ye} =	50 ksi		*Table 10-1*
Reinforcing Steel, f_y =	70 ksi	Default Lower-Bound: 55, 70, 80, 90, 75, 80, 100 ksi	*Table 10-3*
Reinforcing Steel, f_{ye} =	87.5 ksi		*Table 10-1*
Reinforcing Steel ties, f_y =	40 ksi	Default Lower-Bound: 33, 40, 50, 60, 65, 70 ksi	*Table 10-3*
Reinforcing Steel ties, f_{ye} =	50 ksi		*Table 10-1*
Reinforcing Steel ties, f_y =	70 ksi	Default Lower-Bound: 55, 70, 80, 90, 75, 80, 100 ksi	*Table 10-3*
Reinforcing Steel ties, f_{ye} =	87.5 ksi		*Table 10-1*

GRAVITY / SEISMIC FLAT WEIGHT TAKEOFF (PSF)

Typical Floor

CBC Live Load Category: 22. Office: offices [Table 1607.1]

Material	Sloped	Deck	Joists	Girders	Columns	Model	Seismic	Model Seismic
Finish		0.0	0.0	0.0	0.0	0.0	0.0	0.0
Solar / Other		0.0	0.0	0.0	0.0	0.0	0.0	0.0
4.5" conc over metal deck		43.0	43.0	43.0	43.0	43.0	43.0	43.0
		0.0	0.0	0.0	0.0	0.0	0.0	0.0
Insulation		0.0	0.0	0.0	0.0	0.0	0.0	0.0
M.E.P.		2.0	2.0	2.0	2.0	2.0	2.0	2.0
Ceiling - 7/8" plaster			7.0	7.0	7.0	7.0	7.0	7.0
Sprinklers			1.5	1.5	1.5	1.5	1.5	1.5
Joists			52.1	52.1	52.1		52.1	
Girders				15.8	15.8		15.8	
Columns					9.5		9.5	
Misc.		15.0	15.0	15.0	15.0	15.0	15.0	15.0
Dead Load		60.0	120.6	136.4	145.9	68.5	145.9	68.5
Dead Load - Horiz Projection		60.0	120.6	136.4	145.9	68.5	145.9	68.5
Partitions		0.0	0.0	0.0	0.0	0.0	0.0	0.0
Live Load		50.0	50.0	50.0	50.0	50.0	0.0	0.0
Live Load - Reduced R_2 = 1.00		50.0	50.0	50.0	50.0	50.0	0.0	0.0
Total Load		110.0	170.6	186.4	195.9	118.5	145.9	68.5

Roof

CBC Live Load Category: 26. Roof: ordinary [Table 1607.1]
Slope: 0.00:12

Material	Sloped	Deck	Joists	Girders	Columns	Model	Seismic	Model Seismic
Finish		0.0	0.0	0.0	0.0	0.0	0.0	0.0
Solar / Other		0.0	0.0	0.0	0.0	0.0	0.0	0.0
Topping		43.0	43.0	43.0	43.0	43.0	43.0	43.0
Sheathing / Decking		0.0	0.0	0.0	0.0	0.0	0.0	0.0
Insulation		0.0	0.0	0.0	0.0	0.0	0.0	0.0
M.E.P.		2.0	2.0	2.0	2.0	2.0	2.0	2.0
Ceiling			7.0	7.0	7.0	7.0	7.0	7.0
Sprinklers			1.5	1.5	1.5	1.5	1.5	1.5
Joists			52.1	52.1	52.1		52.1	
Girders				15.8	15.8		15.8	
Columns					4.9		4.9	
Misc.		15.0	15.0	15.0	15.0	15.0	15.0	15.0
Dead Load		60.0	120.6	136.4	141.3	68.5	141.3	68.5
Dead Load - Horiz Projection		60.0	120.6	136.4	141.3	68.5	141.3	68.5
Partitions		0.0	0.0	0.0	0.0	0.0	0.0	0.0
Live Load		20.0	20.0	20.0	20.0	20.0	0.0	0.0
Live Load - Reduced R_2 = 1.00		20.0	20.0	20.0	20.0	20.0	0.0	0.0
Total Load		80.0	140.6	156.4	161.3	88.5	141.3	68.5

GRAVITY / SEISMIC WALL WEIGHT TAKEOFF (PSF)

Precast 4" panel

Material	Weight
Precast 4" panel	50.0
TOTAL	50.0

12" thick shear walls

Material	Weight
18" concrete	150.0
TOTAL	150.0

Diaphragm Weight Information:

Level	Area (sq ft)	Diaphragm Unit Weight (psf)	Diaphragm Weight (kips)	Wall Unit Weight (psf)	Wall Trib Height (ft)	Wall Length (ft)	Shear Wall weight (psf)	Shear Trib Height (ft)	Shear Wall Length (ft)	Wall Weight (kips)	Level Weight (kips)
ROOF	17120	141	2,419	50	4.0	534.0	150.0	0.0	0.0	107	**2,526**
3rd	17120	146	2,498	50	8.0	534.0	150.0	0.0	0.0	214	**2,711**
2nd	17120	146	2,498	50	8.0	534.0	150.0	0.0	0.0	214	**2,711**
1st	17120	146	2,498	50	8.0	534.0	150.0	0.0	0.0	214	**2,711**
Σ			9,912						0	748	**10,660**

Retrofit Diaphragm Weight Information:

Level	Area (sq ft)	Diaphragm Unit Weight (psf)	Diaphragm Weight (kips)	Wall Unit Weight (psf)	Wall Trib Height (ft)	Wall Length (ft)	Shear Wall weight (psf)	Shear Trib Height (ft)	Shear Wall Length (ft)	Wall Weight (kips)	Level Weight (kips)
ROOF	17120	141	2,419	50	4.0	534.0	150.0	7.5	100.0	219	**2,639**
3rd	17120	146	2,498	50	8.0	534.0	150.0	15.0	100.0	439	**2,936**
2nd	17120	146	2,498	50	8.0	534.0	150.0	15.8	100.0	450	**2,948**
1st	17120	146	2,498	50	8.0	534.0	150.0	15.0	100.0	439	**2,936**
Σ			9,912							1,546	**11,459**

Vertical Distribution (LSP) - TIER 1

Vertical Distribution of Seismic Forces (North-South)

Y Direction — *ASCE 41-13 §7.4.1.3.2*

$V_{N/S}$ = 5,812 *kips* Total Base Shear
k = 1.1 For T = 0.72 sec

	w_x (kips)	h_x (ft)	$w_x h^k_x$	C_{vx}	F_x (kips)	V_x (kips)
Roof	2,526	60.00	236,404	0.40	2,340	2,340
4th Level	2,711	45.00	184,453	0.31	1,826	4,166
3rd Level	2,711	30.00	117,672	0.20	1,165	5,331
Plaza Level	2,711	13.50	48,555	0.08	481	5,812
Total	*10,660*		*587,084*	*1.00*	*5,812*	

Vertical Distribution of Seismic Forces (East-West)

X Direction — *ASCE 41-13 §7.4.1.3.2*

$V_{E/W}$ = 5,812 *kips* Total Base Shear
k = 1.1 For T = 0.72 sec

	w_x (kips)	h_x (ft)	$w_x h^k_x$	C_{vx}	F_x (kips)	V_x (kips)
Roof	2,526	60.00	236,404	0.40	2,340	2,340
4th Level	2,711	45.00	184,453	0.31	1,826	4,166
3rd Level	2,711	30.00	117,672	0.20	1,165	5,331
Plaza Level	2,711	13.50	48,555	0.08	481	5,812
Total	*10,660*		*587,084*	*1.00*	*5,812*	

SEISMIC HAZARD ANALYSIS (TIER 1) — *ASCE 41-13 §2.4*

Site Coordinates

Latitude =	36.5901	deg	County of Monterey Courthouse
Longitude =	-121.8804	deg	Monterey, California

Spectral Response Acceleration Parameters — *ASCE 41-13 §2.4.1.4*

Site Class =	**D**		Site Soil Classification
S_S =	0.541	g	Mapped Short-period Spectral Response Acceleration
S_1 =	0.193	g	Mapped 1-sec period Spectral Response Acceleration
S_{XS} =	0.739	g	Short-period Spectral Response Acceleration at BSE-1E
S_{X1} =	0.391	g	1-sec period Spectral Response Acceleration at BSE-1E

SEISMIC FORCE — *ASCE 41-13 §4.5.2*

Building Properties

Type N/S =	C1		Building Type in North-South Direction	*ASCE 41-13 Table 3-1*
Type E/W =	C1		Building Type in East-West Direction	*ASCE 41-13 Table 3-1*
Height, h_n =	60.00	ft	Height above base to roof level	
Stories =	4		Number of stories	
Weight N/S =	10659.9	k	Seismic Weight of Building in North-South Direction	
Weight E/W =	10659.9	k	Seismic Weight of Building in East-West Direction	

Building Period — *ASCE 41-13 §4.5.2.4*

North-South Direction:

C_t =	0.02		Period Adjustment Factor	
β =	0.90		Empirical Fundamental Period Adjustment Factor	
T =	0.717	sec	Fundamental Period	$=C_t*h_n^{\beta}$

East-West Direction:

C_t =	0.02		Period Adjustment Factor	
β =	0.90		Empirical Fundamental Period Adjustment Factor	
T =	0.717	sec	Fundamental Period	$=C_t*h_n^{\beta}$

Pseudo-Seismic Force — *ASCE 41-13 §4.5.2.1*

North-South Direction:

S_a =	0.55	g	Spectral Response Acceleration	$=S_{x1}/T < S_{xs}$
C =	1.00		Modification Factor	*Table 4-8*
V =	0.55	*W	Pseudo-Seismic Force in Terms of Weight	=C*Sa*W
V =	**5812.0**	k	Pseudo-Seismic Force	

East-West Direction:

S_a =	0.55	g	Spectral Response Acceleration	$=S_{x1}/T < S_{xs}$
C =	1.00		Modification Factor	*Table 4-8*
V =	0.55	*W	Pseudo-Seismic Force in Terms of Weight	=C*Sa*W
V =	**5812.0**	k	Pseudo-Seismic Force	

Column Axial Stress Caused by Overturning

f'c=	4000	psi
0.3f'c=	1200	psi

Eqn (4-12)

$$p = \frac{1}{M_s}\left(\frac{2}{3}\right)\left(\frac{V h_n}{L n_f}\right)\left(\frac{1}{A_{col}}\right)$$

N-S Direction

nf =	5	
V =	5811.96098	kips
hn =	60	ft
L =	144	ft
Ms =	2	
Acol =	4	ft2
p=	40.4	k/ft2
p=	280.3	psi
DCR=	0.23	OK

E-W Direction

nf =	5	
V =	5812	kips
hn =	60	ft
L =	96	ft
Ms =	2	
Acol =	4	ft2
p=	60.5	k/ft2
p=	420	psi
DCR=	0.35	OK

Compliant

Column Shear Stress Check

$$v = \frac{1}{M_s}\left(\frac{n_c}{n_c - n_f}\right)\left(\frac{V h_n}{L n_f}\right)$$

stress check < 126.5 psi

N-S Direction & E-W Direction

nc =	25
nf =	5
Ms =	2

	Ac	Vj	vj	vj	DCR	
	ft2	kips	k/ft2	psi		
Roof	56.25	2,340	26.0	180.6	1.4	Non Compliant
4th	56.25	4,166	46.3	321.5	2.5	Non Compliant
3rd	100	5,331	33.3	231.4	1.8	Non Compliant
Plaza	100	5,812	36.3	252.3	2.0	Non Compliant

No floors pass the column shear stress test, check is non-compliant

No Shear Failures

Columns

f'c= 4000 psi

	b (in)	d (in)	area (in2)	stl area (in2)	ρl	Av	s (in)	fyl (ksi)	fyv (ksi)
Roof	18	18	324	10.16	0.03136	0.44	18	40	40
4th	18	18	324	10.16	0.03136	0.44	18	40	40
3rd (1)	24	24	576	8	0.01389	0.44	18	50	40
3rd (2)	24	24	576	12.48	0.02167	0.44	18	50	40
Plaza	24	24	576	16.92	0.02938	0.44	18	50	40

	Vo Vc+Vs	P demand	Mp Spcol	L	V_p = 2M/L	Failure Mechanism
Roof	57.1	66.1	276	12.5	44.2	FLEXURE
4th	57.1	134.4	300	12.5	48.0	FLEXURE
3rd (1)	94.9	202.7	490	14	70.0	FLEXURE
3rd (2)	94.9	271.0	700	14	100.0	SHEAR
Plaza	94.9	339.2	815	11	148.2	SHEAR

First floor columns have shear failures and so do some of the second floor columns
Check is non-compliant

Girders

	V	Mp	L	V_p = 2M/L	Failure Mechanism
Roof	106.7	948.9	23	82.5	FLEXURE
4th	106.7	1106.8	23	96.2	FLEXURE
3rd	106.7	1176.1	23	102.3	FLEXURE
Plaza	106.7	794.5	23	69.1	FLEXURE

All girders are flexure controlled, check is compliant

Beams

There are multiple beam reinforcing details, typical chosen for ea flr

	V	Mp	L	V_p = 2M/L	Failure Mechanism
Roof	65.4	340.2	35	19.4	FLEXURE
4th	65.4	624.5	35	35.7	FLEXURE
3rd	65.4	522.8	35	29.9	FLEXURE
Plaza	65.4	624.5	35	35.7	FLEXURE

All beams are flexure controlled, check is compliant

Strong Column - Weak Beam

	Mcol	1.2(Mbm)	
Roof	276	1139	Non Compliant
4th	600	1822	Non Compliant
3rd (1)	980	2441	Non Compliant
3rd (2)	1400	2441	Non Compliant
Plaza	1630	1907	Non Compliant

All levels are non-compliant for strong column - weak beam check. Note this is checking strongest beam which is the girders. Other direction may pass and further checks will be done in tier 2

Beam Bars

Based on existing drawing sheet S4 all beams have locations of noncontinuous top and bottom longitudinal rebar
Check is non-compliant

Column Bar Splices

	rebar	35db (inches)	per S1 (inches)	
4th	#10	44	42	Non Compliant
3rd (1)	#11	49	45	Non Compliant
3rd (2)	#9	39	48	OK
Plaza	#11	49	48	Non Compliant
Base	#11	49	60	OK

	rebar	8db	spacing	
4th	#10	10	9	Non Compliant
3rd (1)	#11	11	9	Non Compliant
3rd (2)	#9	9	9	OK
Plaza	#11	11	9	Non Compliant
Base	#11	11	9	Non Compliant

Column Splices are non-compliant but are close to passing. In the tier 2 check m-factors will be reduced slightly and checked for this reduction in ductility.

Beam Bar Splices

lap splices are located directly adjacent to columns. Non-Compliant

Column Tie Spacing

	d/4 (inches)	8db (inches)	spacing (inches)	
24" TYP	6.0	-	18	Non Compliant
18" TYP	4.5	-	18	Non Compliant
hinge	-	3.0	9	Non Compliant

Check is non-compliant

Stirrup Spacing

	d/2 (inches)	min(8db,d/4) (inches)	spacing (inches)	
Girder TY	17.3	-	14	OK
Girder hir	-	8.6	14	Non Compliant
Beam TY	14.3	-	18	Non Compliant
Beam hin	-	7.1	18	Non Compliant

Check is non-compliant

Joint Transverse Reinforcing
Non-compliant based on two checks above

Weak/Soft Story
precast concrete panels will add additional stiffness at the third and fourth levels

panel strength

f'c=	3000	psi
Vc=	5258	plf

North-South Direction

	#col	Vcol kips	Lwall ft	Vwall kips	Vtot
Roof	25	1428	120	631	2059
4th	25	1428	120	631	2059
3rd	25	2371	0	0	2371
Plaza	25	2371	0	0	2371

East-West Direction

	#col	Vcol kips	Lwall ft	Vwall kips	Vtot
Roof	25	1428	59	312	1740
4th	25	1428	59	312	1740
3rd	25	2371	0	0	2371
Plaza	25	2371	0	0	2371

Compliant, there are no weak or soft stories.

Overturning

›ast horizontal dimension / building height =	1.6	
0.6Sa =	0.3	OK

Compliant

Beam: Girder Plaza, M- & M+

Properties			Flexural Reinf.				Shear Reinf.		
f'_c =	4000	psi	5	#10	bars			2	Legs
f_y =	50	ksi	A_s =	6.35	in^2	OK	#4	@ 14	"oc
f_{yt} =	50	ksi	$A_{s,min}$ =	0.91	in^2				
			a =	6.67	in		V_c =	59.1	k
Beam Dimensions			β_1 =	0.85			V_s =	47.7	k
Width =	14.0	in	c =	7.85	in		V_n =	106.7	k
Depth =	36.0	in	ε_s =	0.010					
Cover =	1.5	in							
d =	33.4	in	M_n =	794.5	k-ft				

Beam: Girder 3rd, M-

Properties			Flexural Reinf.					Shear Reinf.		
f'_c =	4000	psi	3	#10	bars	+ 4 #11 bars			2	Legs
f_y =	50	ksi	A_s =	10.05	in^2	OK		#4	@ 14	"oc
f_{yt} =	50	ksi	$A_{s,min}$ =	0.91	in^2					
			a =	10.56	in			V_c =	59.1	k
Beam Dimensions			β_1 =	0.85				V_s =	47.7	k
Width =	14.0	in	c =	12.42	in			V_n =	106.7	k
Depth =	36.0	in	ε_s =	0.005						
Cover =	1.5	in								
d =	33.4	in	M_n =	1176.1	k-ft					

Beam: Girder 3rd, M+

Properties			Flexural Reinf.					Shear Reinf.		
f'_c =	4000	psi	3	#10	bars	+ 2 #11 bars			2	Legs
f_y =	50	ksi	A_s =	6.93	in^2	OK		#4	@ 14	"oc
f_{yt} =	50	ksi	$A_{s,min}$ =	0.91	in^2					
			a =	7.28	in			V_c =	59.1	k
Beam Dimensions			β_1 =	0.85				V_s =	47.7	k
Width =	14.0	in	c =	8.56	in			V_n =	106.7	k
Depth =	36.0	in	ε_s =	0.009						
Cover =	1.5	in								
d =	33.4	in	M_n =	858.3	k-ft					

Beam: Girder 4th, M-

Properties			Flexural Reinf.				Shear Reinf.		
f'_c =	4000	psi	6	#11	bars			2	Legs
f_y =	50	ksi	A_s =	9.36	in^2	OK	#4	@ 14	"oc
f_{yt} =	50	ksi	$A_{s,min}$ =	0.91	in^2				
			a =	9.83	in		V_c =	59.0	k
Beam Dimensions			β_1 =	0.85			V_s =	47.6	k
Width =	14.0	in	c =	11.57	in		V_n =	106.5	k
Depth =	36.0	in	ε_s =	0.006					
Cover =	1.5	in							
d =	33.3	in	M_n =	1106.8	k-ft				

Beam: Girder 4th, M+

Properties			Flexural Reinf.				Shear Reinf.		
f'_c =	4000	psi	2	#11	bars			2	Legs
f_y =	50	ksi	A_s =	3.12	in^2	OK	#4	@ 14	"oc
f_{yt} =	50	ksi	$A_{s,min}$ =	0.91	in^2				
			a =	3.28	in		V_c =	59.0	k
Beam Dimensions			β_1 =	0.85			V_s =	47.6	k
Width =	14.0	in	c =	3.86	in		V_n =	106.5	k
Depth =	36.0	in	ε_s =	0.023					
Cover =	1.5	in							
d =	33.3	in	M_n =	411.5	k-ft				

Beam: Girder Roof, M- & M+

Properties			Flexural Reinf.				Shear Reinf.		
f'_c =	4000	psi	5	#11	bars			2	Legs
f_y =	50	ksi	A_s =	7.80	in^2	OK	#4	@ 14	"oc
f_{yt} =	50	ksi	$A_{s,min}$ =	0.91	in^2				
			a =	8.19	in		V_c =	59.0	k
Beam Dimensions			β_1 =	0.85			V_s =	47.6	k
Width =	14.0	in	c =	9.64	in		V_n =	106.5	k
Depth =	36.0	in	ε_s =	0.007					
Cover =	1.5	in							
d =	33.3	in	M_n =	948.9	k-ft				

Beam: Beam plaza level, M+ center & 4th level

Properties			Flexural Reinf.			Shear Reinf.	
f'_c =	4000 psi		2	#9 bars + 1 #10 bars			2 Legs
f_y =	50 ksi		A_s =	3.27 in^2	OK	#3	@ 18 "oc
f_{yt} =	50 ksi		$A_{s,min}$ =	0.76 in^2			
			a =	3.43 in		V_c =	48.6 k
Beam Dimensions			β_1 =	0.85		V_s =	16.8 k
Width =	14.0 in		c =	4.04 in		V_n =	65.4 k
Depth =	30.0 in		ε_s =	0.017			
Cover =	1.5 in						
d =	27.4 in		M_n =	350.4 k-ft			

Beam: Beam plaza level, M- end & 4th level

Properties			Flexural Reinf.			Shear Reinf.	
f'_c =	4000 psi		4	#11 bars			2 Legs
f_y =	50 ksi		A_s =	6.24 in^2	OK	#3	@ 18 "oc
f_{yt} =	50 ksi		$A_{s,min}$ =	0.76 in^2			
			a =	6.55 in		V_c =	48.3 k
Beam Dimensions			β_1 =	0.85		V_s =	16.7 k
Width =	14.0 in		c =	7.71 in		V_n =	65.0 k
Depth =	30.0 in		ε_s =	0.008			
Cover =	1.5 in						
d =	27.3 in		M_n =	624.5 k-ft			

Beam: Beam 3rd level, M+ center

Properties			Flexural Reinf.			Shear Reinf.	
f'_c =	4000 psi		3	#10 bars			2 Legs
f_y =	50 ksi		A_s =	3.81 in^2	OK	#3	@ 18 "oc
f_{yt} =	50 ksi		$A_{s,min}$ =	0.76 in^2			
			a =	4.00 in		V_c =	48.5 k
Beam Dimensions			β_1 =	0.85		V_s =	16.7 k
Width =	14.0 in		c =	4.71 in		V_n =	65.2 k
Depth =	30.0 in		ε_s =	0.014			
Cover =	1.5 in						
d =	27.4 in		M_n =	402.7 k-ft			

Beam: Beam 3rd level, M- end

Properties		Flexural Reinf.			Shear Reinf.	
f'_c =	4000 psi	4	#10 bars		2 Legs	
f_y =	50 ksi	A_s =	5.08 in^2	OK	#3	@ 18 "oc
f_{yt} =	50 ksi	$A_{s,min}$ =	0.76 in^2			
		a =	5.34 in		V_c =	48.5 k
Beam Dimensions		β_1 =	0.85		V_s =	16.7 k
Width =	14.0 in	c =	6.28 in		V_n =	65.2 k
Depth =	30.0 in	ε_s =	0.010			
Cover =	1.5 in					
d =	27.4 in	M_n =	522.8 k-ft			

Beam: Beam roof, M+ center

Properties		Flexural Reinf.			Shear Reinf.	
f'_c =	4000 psi	3	#8 bars		2 Legs	
f_y =	50 ksi	A_s =	2.37 in^2	OK	#3	@ 18 "oc
f_{yt} =	50 ksi	$A_{s,min}$ =	0.76 in^2			
		a =	2.49 in		V_c =	48.7 k
Beam Dimensions		β_1 =	0.85		V_s =	16.8 k
Width =	14.0 in	c =	2.93 in		V_n =	65.5 k
Depth =	30.0 in	ε_s =	0.025			
Cover =	1.5 in					
d =	27.5 in	M_n =	259.3 k-ft			

Beam: Beam roof, M- at end

Properties		Flexural Reinf.			Shear Reinf.	
f'_c =	4000 psi	4	#8 bars		2 Legs	
f_y =	50 ksi	A_s =	3.16 in^2	OK	#3	@ 18 "oc
f_{yt} =	50 ksi	$A_{s,min}$ =	0.76 in^2			
		a =	3.32 in		V_c =	48.7 k
Beam Dimensions		β_1 =	0.85		V_s =	16.8 k
Width =	14.0 in	c =	3.91 in		V_n =	65.5 k
Depth =	30.0 in	ε_s =	0.018			
Cover =	1.5 in					
d =	27.5 in	M_n =	340.2 k-ft			

SEISMIC HAZARD ANALYSIS (TIER 2)

ASCE 41-13 §2.4.1

Site Coordinates

Latitude =	36.5901	deg	County of Monterey Courthouse
Longitude =	-121.880	deg	Monterey, California

Spectral Response Acceleration Parameters

ASCE 41-13 §2.4.1.4

Site Class =	D		Site Soil Classification
S_S =	0.541	g	Mapped Short-period Spectral Response Acceleration
S_1 =	0.193	g	Mapped 1-sec period Spectral Response Acceleration
S_{XS} =	0.739	g	Short-period Spectral Response Acceleration at BSE-1E
S_{X1} =	0.391	g	1-sec period Spectral Response Acceleration at BSE-1E

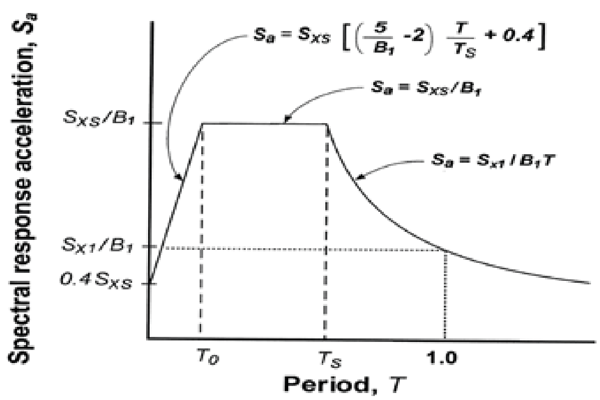
General Horizontal Response Spectrum

ASCE 41-13 §2.4.1.7

β =	5%		Effective Viscous Damping Ratio	
B_1 =	1.00		Damping Coefficient	*= 4 / (5.6 - ln(100β))*
T_S =	0.53	sec	Period at Constant Velocity Region	*= S_{X1} / S_{XS}*
T_0 =	0.11	sec	Period at Constant Acceleration Region	*= $0.2\ T_S$*
$0.4S_{XS}$=	0.30	g	Peak Ground Acceleration	
S_{XS}/B_1 =	0.74	g	Short period Spectral Response Acceleration	
S_{X1}/B_1 =	0.39	g	1-sec period Design Spectral Response Acceleration	

Spectral Acceleartion at Building Period

ASCE 41-13 §2.4



ASCE 31-13 Figure 2-1: General Horizontal Response Spectrum

Existing Conditions

North-South Direction

T=	0.717	sec	Building period in N-S direction	*Per LSP calcs*
S_a=	**0.544**		Spectral Response Acceleration at Building Period	*Eq. 2-10*

East-West Direction

T=	0.717	sec	Building period in N-S direction	*Per LSP calcs*
S_a=	**0.544**		Spectral Response Acceleration at Building Period	*Eq. 2-10*

Retrofit Structure

North-South Direction

T=	0.431	sec	Building period in N-S direction	*Per LSP calcs*
S_a=	**0.737**		Spectral Response Acceleration at Building Period	*Eq. 2-10*

East-West Direction

T=	0.431	sec	Building period in N-S direction	*Per LSP calcs*
S_a=	**0.737**		Spectral Response Acceleration at Building Period	*Eq. 2-10*

LINEAR STATIC PROCEDURE (LSP) - Tier 2

ASCE 41-13 §7.4.1

# Stories =	4		Number of stories in building	

Period Determination for LSP - Method 2 - Empirical

ASCE 41-13 §7.4.1.2

North-South Direction

Building Type =	C1			
C_t =	0.018		Factor for adjustment of period	*ASCE 41-13 §7.4.1.2.2*
β =	0.90		Factor for adjustment of period	
h_n =	60.0	ft	Roof Height	
T =	**0.717**	sec	Building period in N-S direction	*ASCE 41-13 Eq. 7-18*

East-West Direction

Building Type =	C1			
C_t =	0.018		Factor for adjustment of period	
β =	0.90		Factor for adjustment of period	
h_n =	60.0	ft	Roof Height	
T =	**0.717**	sec	Building period in E-W direction	*ASCE 41-13 Eq. 7-18*

Pseudo-Seismic Force for LSP (North-South)

ASCE 41-13 §7.4.1.3.1

$V = C_1 C_2 C_m S_a W$			Pseudo-Lateral Force	*ASCE 41-13 Eq. 7-21*
C_1 =	1.0		Modification Factor, Inelastic Displacements	*ASCE 41-13 Eq. 7-22*
C_2 =	1.0		Modification Factor, Cyclic Behavior	*ASCE 41-13 Eq. 7-23*
$C_1 C_2$ =	1.1		Alternative Value for Modification Factors	*ASCE 41-13 Table 7-3*
Use alternate C_1C_2?	Yes			
C_m =	0.9		Effective Mass Factor	*ASCE 41-13 Table 7-4*
$S_a(T)$ =	**0.544**	g	Spectral Response Acceleration	*for T = 0.72sec*
W =	10659.9	kips	Effective Seismic Weight	
$V_{N/S}$ =	**0.538**	*W	Pseudo-Lateral Force	
$V_{N/S}$ =	**5,740.3**	kips	Pseudo-Lateral Force	

Pseudo-Seismic Force for LSP (East-West)

ASCE 41-13 §7.4.1.3.1

$V = C_1 C_2 C_m S_a W$			Pseudo-Lateral Force	*ASCE 41-13 Eq. 7-21*
C_1 =	1.0		Modification Factor, Inelastic Displacements	*ASCE 41-13 Eq. 7-22*
C_2 =	1.0		Modification Factor, Cyclic Behavior	*ASCE 41-13 Eq. 7-23*
$C_1 C_2$ =	1.1		Alternative Value for Modification Factors	*ASCE 41-13 Table 7-3*
Use alternate C_1C_2?	Yes			
C_m =	0.9		Effective Mass Factor	*ASCE 41-13 Table 7-4*
$S_a(T)$ =	**0.544**	g	Spectral Response Acceleration	*for T = 0.72sec*
W =	10659.9	kips	Effective Seismic Weight	
$V_{E/W}$ =	**0.538**	*W	Pseudo-Lateral Force	
$V_{E/W}$ =	**5,740.3**	kips	Pseudo-Lateral Force	

Column Shear Stress Check

Tier 2: Section 5.5.2.1.4

k= 0.9 per table 6-1

f'ce= 6000 psi

	b (in)	d (in)	area (in2)	stl area (in2)	ρl	Av	s (in)	fyl (ksi)	fyv (ksi)	ρv
Roof	18	18	324	10.16	0.03136	0.44	18	50	50	0.0014
4th	18	18	324	10.16	0.03136	0.44	18	50	50	0.0014
3rd (1)	24	24	576	8	0.01389	0.44	18	62.5	50	0.0010
3rd (2)	24	24	576	12.48	0.02167	0.44	18	62.5	50	0.0010
Plaza	24	24	576	16.92	0.02938	0.44	18	62.5	50	0.0010

	Vo Vc+Vs	P demand	Mp Spcol	L	V_p = 2M/L	Vp/Vo		Vn kips	P/AF'c	V/bdfc	m
Roof	70.4	58	325	12.5	52.0	0.74	ii	52.0	0.03	4.77	1.16
4th	70.4	114	360	12.5	57.6	0.82	ii	57.6	0.06	8.32	1.16
3rd (1)	116.7	174	560	14	80.0	0.69	ii	80.0	0.05	6.71	1.09
3rd (2)	116.7	174	775	14	110.7	0.95	ii	110.7	0.05	6.71	1.09
Plaza	116.7	232	1055	11	191.8	1.64	iii	116.7	0.07	6.30	1

	Max V Eqx (kips)	Max V Eqy (kips)	mkVn	DCR
Roof	109.8	109.8	73.20	1.5
4th	191.5	190.1	73.20	2.6
3rd	280.6	272.3	114.96	2.4
Plaza	263.5	259.3	105.06	2.5

Tier 2 check is non-compliant

No Shear Failures

Tier 2: Section 5.5.2.3.4

All columns are noncompliant, the adequacy of the columns for shear was checked in the previous section. The beams are all controlled by flexural failure and therefore do not need to be check

Strong column - Weak Beam
Tier 2: Section 5.5.2.1.5

Evaluate columns for shear & flexure with m=2. Column shear demands per shear check above

	Max V Eqx (kips)	Max V Eqy (kips)	m	Vn	mkVn	DCR
Roof	109.8	109.8	2	52.0	93.6	1.2
4th	191.5	190.1	2	57.6	103.7	1.8
3rd (1)	280.6	272.3	2	80.0	144.0	1.9
3rd (2)	280.6	272.3	2	110.7	199.3	1.4
Plaza	263.5	259.3	2	116.7	210.1	1.2

all columns fail the shear check with m=2

	Max M Eqx (k-ft)	Max M Eqy (k-ft)	m	Mn	mkMn	DCR
Roof	1075	1054	2	325	585.0	1.8
4th	1400	1365	2	360	648.0	2.2
3rd (1)	2184	2037	2	560	1008.0	2.2
3rd (2)	2184	2037	2	775	1395.0	1.6
Plaza	2865	2820	2	1055	1899.0	1.5

all columns fail the flexure check with m=2

Beam Bars
Tier 2: Section 5.5.2.3.5

flexural demand of noncompliant beams shall be checked at ends and middle with an m-factor = 1

Beams

	Max Mu (k-ft)	m	Mn	mkMn	DCR
roof end	600	1	259.3	233.3	2.3
roof middle	213	1	259.3	233.3	0.8
4th end	1345	1	624.5	562.0	2.2
4th middle	230	1	350.4	315.4	0.7
3rd end	2044	1	522.8	470.5	3.9
3rd middle	275	1	402.7	362.4	0.7
plaza end	2848	1	624.5	562.0	4.6
plaza middle	350	1	350.4	315.4	1.0

Girders

	Max Mu (k-ft)	m	Mn	mkMn	DCR
roof end	895	1	949	854.1	0.9
roof middle	310	1	949	854.1	0.3
4th end	1980	1	1107	996.1	1.8
4th middle	320	1	412	370.4	0.8
3rd end	3000	1	1176	1058.5	2.6
3rd middle	390	1	858	772.5	0.5
plaza end	4500	1	795	715.1	5.7
plaza middle	290	1	795	715.1	0.4

Most beams fail induced end moments. All of the beams pass the demand at the beam middle

Column-Bar Splices
Tier 2: Section 5.5.2.3.6

The flexural demands at noncompliant column splices shall be calculated and the adequacy shall be evaluated.

There are non-conforming column splices at the 3rd, 4th and roof levels. The m is taken as 1 for non-conforming lap lengths

	Max M Eqx (k-ft)	Max M Eqy (k-ft)	m	Mn	mkMn	DCR
Roof	1075	1054	1.00	325.0	292.5	3.7
4th	1400	1365	1.00	360.0	324.0	4.3
3rd (1)	2184	2037	1.00	560.0	504.0	4.3
3rd (2)	2184	2037	1.00	775.0	697.5	3.1

All columns fail the flexural check

Beam-Bar Splices
Tier 2: Section 5.5.2.3.6

This check is similar to the Beam Bar check above, except instead of an m=1 it is allowed to use an m=1.5 per Table 10-13. However, upon inspection of the results from m=1, even with an increased m, beams will still fail.

Column-Tie Spacing
Tier 2: Section 5.5.2.3.7

The force demand in in noncompliant columns shall be calculated and the adequacy of the elements shall be evaluated

The column shear and flexural demand were checked with the reduced m factors above in the Column Shear Stress check and the Column- Bar splices. All column checks are non compliant.

Stirrup Spacing
Tier 2: Section 5.5.2.3.7

f'ce= 6000 psi

Beams

b= 14 inch d= 28.5 inch

	Mn	L	Vp=2M/L	Vo (kips)		Max Vu (kips)	V/ bdf'c	m	Vn	mkVn	DCR
roof, M+	259.3	35	14.8	65.5	Flexure	58	1.88	3.0	14.8	40.0	1.4
roof, M-	340.2	35	19.4	65.5	Flexure	58	1.88	3.0	19.4	52.5	1.1
4th, M+	350.4	35	20.0	65.0	Flexure	94	3.04	3.0	20.0	54.1	1.7
4th, M-	624.5	35	35.7	65.0	Flexure	94	3.04	3.0	35.7	96.3	1.0
3rd, M+	402.7	35	23.0	65.2	Flexure	124	4.01	3.0	23.0	62.0	2.0
3rd, M-	522.8	35	29.9	65.2	Flexure	124	4.01	3.0	29.9	80.5	1.5
plaza, M+	350.4	35	20.0	65.2	Flexure	170	5.5	2.2	20.0	40.5	4.2
plaza, M-	624.5	35	35.7	65.2	Flexure	170	5.5	2.2	35.7	72.3	2.4

Girders

b= 14 inch d= 34.5 inch

	Mn	L	Vp=2M/L	Vo (kips)		Max Vu (kips)	V/ bdf'c	m	Vn	mkVn	DCR
roof, M+	948.9	23	82.5	106.5	Flexure	115	3.07	3.0	82.5	222.8	0.5
roof, M-	948.9	23	82.5	106.5	Flexure	115	3.07	3.0	82.5	222.8	0.5
4th, M+	411.5	23	35.8	106.5	Flexure	180	4.81	2.6	35.8	83.6	2.2
4th, M-	1106.8	23	96.2	106.5	Flexure	180	4.81	2.6	96.2	224.7	0.8
3rd, M+	858.3	23	74.6	106.7	Flexure	220	5.88	2.1	74.6	138.4	1.6
3rd, M-	1176.1	23	102.3	106.7	Flexure	220	5.88	2.1	102.3	189.6	1.2
plaza, M+	794.5	23	69.1	106.7	Flexure	305	8.15	2.0	69.1	124.4	2.5
plaza, M-	794.5	23	69.1	106.7	Flexure	305	8.15	2.0	69.1	124.4	2.5

Most beams and girders checked do not have sufficient strength for the seismic shear demands.
Check is noncompliant

Joint Transverse Reinforcing
Tier 2: Section 5.5.2.3.8

f'_{ce}= 6000 psi

Beams (4th and roof level)

col width =	18 inch		
beam width =	14 inch	beam width + joint depth =	32 inch
joint depth =	18 inch	beam width + 2x =	18 inch
x from bm to col =	2 inch		

Beams (plaza and 3rd level)

col width =	24 inch		
beam width =	14 inch	beam width + joint depth =	38 inch
joint depth =	24 inch	beam width + 2x =	24 inch
x from bm to col =	5 inch		

	A_j (in^2)	γ	V_n (Kips)	m	mkVn	Vu	DCR	Compliance
4th	324.0	8.0	200.8	1.0	180.7	58.0	0.29	OK
3rd	324.0	8.0	200.8	1.0	180.7	94.0	0.47	OK
2nd	576.0	8.0	356.9	1.0	321.2	124.0	0.35	OK
1st	576.0	8.0	356.9	1.0	321.2	170.0	0.48	OK

Girders (4th and roof level)

col width =	18 inch		
beam width =	14 inch	beam width + joint depth =	32 inch
joint depth =	18 inch	beam width + 2x =	18 inch
x from bm to col =	2 inch		

Girders (plaza and 3rd level)

col width =	24 inch		
beam width =	14 inch	beam width + joint depth =	38 inch
joint depth =	24 inch	beam width + 2x =	24 inch
x from bm to col =	5 inch		

	A_j (in^2)	γ	V_n (Kips)	m	mkVn	Vu	DCR	Compliance
4th	324.0	8.0	200.8	1.0	180.7	115.0	0.57	OK
3rd	324.0	8.0	200.8	1.0	180.7	180.0	0.90	OK
2nd	576.0	8.0	356.9	1.0	321.2	220.0	0.62	OK
1st	576.0	8.0	356.9	1.0	321.2	305.0	0.85	OK

Adjacent Buildings
Tier 2: Section 5.4.1.2

The courthouse and annex are only a few inches apart directly over the entrance to the courthouse
This condition should be mitigated to prevent pounding of the structures

LINEAR STATIC PROCEDURE (LSP) - Tier 2 - RETROFIT STRUCTURE

ASCE 41-13 §7.4.1

# Stories =	4		Number of stories in building	

Period Determination for LSP - Method 2 - Empirical

ASCE 41-13 §7.4.1.2

North-South Direction

Building Type =	C2			
C_t =	0.020		Factor for adjustment of period	*ASCE 41-13 §7.4.1.2.2*
β =	0.75		Factor for adjustment of period	
h_n =	60.0	ft	Roof Height	
T =	**0.431**	sec	Building period in N-S direction	*ASCE 41-13 Eq. 7-18*

East-West Direction

Building Type =	C2			
C_t =	0.020		Factor for adjustment of period	
β =	0.75		Factor for adjustment of period	
h_n =	60.0	ft	Roof Height	
T =	**0.431**	sec	Building period in E-W direction	*ASCE 41-13 Eq. 7-18*

Pseudo-Seismic Force for LSP (North-South)

ASCE 41-13 §7.4.1.3.1

$V = C_1 C_2 C_m S_a W$			Pseudo-Lateral Force	*ASCE 41-13 Eq. 7-21*
C_1 =	1.0		Modification Factor, Inelastic Displacements	*ASCE 41-13 Eq. 7-22*
C_2 =	1.0		Modification Factor, Cyclic Behavior	*ASCE 41-13 Eq. 7-23*
$C_1 C_2$ =	1.1		Alternative Value for Modification Factors	*ASCE 41-13 Table 7-3*
Use alternate C_1C_2?	Yes			
C_m =	0.8		Effective Mass Factor	*ASCE 41-13 Table 7-4*
$S_a(T)$ =	**0.737**	g	Spectral Response Acceleration	*for T = 0.43sec*
W =	11458.7	kips	Effective Seismic Weight	
$V_{N/S}$ =	**0.649**	*W	Pseudo-Lateral Force	
$V_{N/S}$ =	**7,434.2**	kips	Pseudo-Lateral Force	

Pseudo-Seismic Force for LSP (East-West)

ASCE 41-13 §7.4.1.3.1

$V = C_1 C_2 C_m S_a W$			Pseudo-Lateral Force	*ASCE 41-13 Eq. 7-21*
C_1 =	1.0		Modification Factor, Inelastic Displacements	*ASCE 41-13 Eq. 7-22*
C_2 =	1.0		Modification Factor, Cyclic Behavior	*ASCE 41-13 Eq. 7-23*
$C_1 C_2$ =	1.1		Alternative Value for Modification Factors	*ASCE 41-13 Table 7-3*
Use alternate C_1C_2?	Yes			
C_m =	0.8		Effective Mass Factor	*ASCE 41-13 Table 7-4*
$S_a(T)$ =	**0.737**	g	Spectral Response Acceleration	*for T = 0.43sec*
W =	11458.7	kips	Effective Seismic Weight	
$V_{E/W}$ =	**0.649**	*W	Pseudo-Lateral Force	
$V_{E/W}$ =	**7,434.2**	kips	Pseudo-Lateral Force	

Vertical Distribution (LSP) - Tier 2 Retrofit

Vertical Distribution of Seismic Forces (North-South) *Y Direction* *ASCE 41-13 §7.4.1.3.2*

$V_{N/S}$ = 7,434 *kips* Total Base Shear
k = 1.0 For T = 0.43 sec

	w_x (kips)	h_x (ft)	$w_x h^k_x$	C_{vx}	F_x (kips)	V_x (kips)	F_{px} (kips)
Roof	2,526	60.00	151,563	0.39	2,878	2,878	2,878
4th Level	2,711	45.00	122,008	0.31	2,317	5,195	2,689
3rd Level	2,711	30.00	81,339	0.21	1,544	6,739	2,299
Plaza Level	2,711	13.50	36,602	0.09	695	7,434	1,891
Total	*10,660*		*391,513*	*1.00*	*7,434*		

Vertical Distribution of Seismic Forces (East-West) *X Direction* *ASCE 41-13 §7.4.1.3.2*

$V_{E/W}$ = 7,434 *kips* Total Base Shear
k = 1.0 For T = 0.43 sec

	w_x (kips)	h_x (ft)	$w_x h^k_x$	C_{vx}	F_x (kips)	V_x (kips)	F_{px} (kips)
Roof	2,526	60.00	151,563	0.39	2,878	2,878	2,878
4th Level	2,711	45.00	122,008	0.31	2,317	5,195	2,689
3rd Level	2,711	30.00	81,339	0.21	1,544	6,739	2,299
Plaza Level	2,711	13.50	36,602	0.09	695	7,434	1,891
Total	*10,660*		*391,513*	*1.00*	*7,434*		

Deficiency Based Retrofit

The deficiences that remain after the Tier 2 evaluation will now be mitigated with a retrofit. The scope of the Tier 2 deficiency-based retrofit need not expand beyond that necessary to modify the building to comply with a Tier 1 screening or a Tier 2 Evaluation.

k=	0.9 per table 6-1		f'ce=	6000 psi (existing concrete)
fye=	75 ksi		f'ce=	5200 psi (new walls)

New concrete Shear Walls

twall = 12 inch   Lwall = 24 ft   k= 1

max axial (kips) from ETABS

	X	X	Y	Y	
	Pier 2	Pier 7	Pier J	Pier M	MIN
roof	243	110	155	140	110
Story4	345	188	238	230	188
Story3	480	270	340	310	270
Story2	530	320	390	360	320

max shear (kips) from ETABS

	X	X	Y	Y	
	Pier 2	Pier 7	Pier J	Pier M	MAX
roof	1608	1315	1290	1630	1630
Story4	2800	2355	2540	2610	2800
Story3	3500	2980	3252	3210	3500
Story2	3900	3550	3640	3820	3900

max moment (kip-ft) from ETABS

	X	X	Y	Y	
	Pier 2	Pier 7	Pier J	Pier M	MAX
roof	25700	23200	21250	23200	25700
Story4	54600	55200	51000	52000	55200
Story3	81500	101500	85900	94400	101500
Story2	109000	147000	119000	139100	147000

New Shear Wall Design

	Acv (in2)	α	ρt	Vn	m	mkVn	DCR
roof	3456	2	0.00287	1242.3	2.5	3105.8	0.52
Story4	3456	2	0.00287	1242.3	2.5	3105.8	0.90
Story3	3456	2	0.00407	1553.4	2.5	3883.4	0.90
Story2	3456	2	0.00407	1553.4	2.5	3883.4	1.00

	Mn (kip-ft)	ρl	mkMn	DCR		steel lbs/yd3
roof	15500	0.00390	38750	0.66	#6@18"oc	90
Story4	25200	0.00690	63000	0.88	#6@18"oc, (4)#11 EE	129
Story3	42000	0.00990	105000	0.97	#7@18"oc, (6)#11 EE	185
Story2	55000	0.01630	137500	1.07	#7@9"oc, (8)#11 EE	269

All demands from ETABS model are output with walls in the model to determine reduced loads to existing structure.

Column Shear Stress Check
Tier 2: Section 5.5.2.1.4

	Max V Eqx (kips)	Max V Eqy (kips)	m	mkVn	DCR
Roof	13.0	23.2	1.16	73.20	0.32
4th	17.6	33.3	1.16	73.20	0.45
3rd	26.6	57.2	1.09	114.96	0.50
Plaza	7.1	12.4	1.00	105.06	0.12

Tier 2 check is compliant with new loads

No Shear Failures
Tier 2: Section 5.5.2.3.4

Per check above, the shear controlled elements (columns) are no longer over-stressed. Check is compliant

Strong column - Weak Beam
Tier 2: Section 5.5.2.1.5

Evaluate columns for shear & flexure with m=2. Column shear demands per shear check above

	Max V Eqx (kips)	Max V Eqy (kips)	m	Vn	mkVn	DCR
Roof	13.0	23.2	2	52.0	93.6	0.25
4th	17.6	33.3	2	57.6	103.7	0.32
3rd (1)	26.6	57.2	2	80.0	144.0	0.40
3rd (2)	26.6	57.2	2	110.7	199.3	0.29
Plaza	7.1	12.4	2	116.7	210.1	0.06

all columns pass the shear check with m=2

	Max M Eqx (k-ft)	Max M Eqy (k-ft)	m	Mn	mkMn	DCR
Roof	128	227	2	325	585.0	0.39
4th	125	241	2	360	648.0	0.37
3rd (1)	216	446	2	560	1008.0	0.44
3rd (2)	216	446	2	775	1395.0	0.32
Plaza	80	120	2	1055	1899.0	0.06

all columns pass the flexure check with m=2

Beam Bars
Tier 2: Section 5.5.2.3.5

flexural demand of noncompliant beams shall be checked at ends and middle with an m-factor = 1

Beams

	Max Mu (k-ft)	m	Mn	mkMn	DCR
roof end	112	1	259.3	233.3	0.5
roof middle	96	1	259.3	233.3	0.4
4th end	132	1	624.5	562.0	0.2
4th middle	90	1	350.4	315.4	0.3
3rd end	140	1	522.8	470.5	0.3
3rd middle	85	1	402.7	362.4	0.2
plaza end	124	1	624.5	562.0	0.2
plaza middle	82	1	350.4	315.4	0.3

Girders

	Max Mu (k-ft)	m	Mn	mkMn	DCR
roof end	180	1	949	854.1	0.2
roof middle	180	1	949	854.1	0.2
4th end	200	1	1107	996.1	0.2
4th middle	155	1	412	370.4	0.4
3rd end	230	1	1176	1058.5	0.2
3rd middle	160	1	858	772.5	0.2
plaza end	205	1	795	715.1	0.3
plaza middle	155	1	795	715.1	0.2

All beams pass the retrofit load check, check is compliant
The above demand does not include beams directly adjacent to shear walls.
Beams at shear walls will likely need FRP for induced forces

Column-Bar Splices
Tier 2: Section 5.5.2.3.6

The flexural demands at noncompliant column splices shall be calculated and the adequacy shall be evaluated.

There are non-conforming column splices at the 3rd, 4th and roof levels. The m is taken as 1 for non-conforming lap lengths

	Max M Eqx (k-ft)	Max M Eqy (k-ft)	m	Mn	mkMn	DCR
Roof	128	227	1.00	325.0	292.5	0.78
4th	125	241	1.00	360.0	324.0	0.74
3rd (1)	216	446	1.00	560.0	504.0	0.88
3rd (2)	216	446	1.00	775.0	697.5	0.64

All columns pass the flexure check

Beam-Bar Splices
Tier 2: Section 5.5.2.3.6

This check is similar to the Beam Bar check above, except instead of an m=1 it is allowed to use an m=1.5 per Table 10-13. All beams pass with m=1 so therefore with an increased m, will still be compliant. Check is compliant

Column-Tie Spacing
Tier 2: Section 5.5.2.3.7

The force demand in in noncompliant columns shall be calculated and the adequacy of the elements shall be evaluated

The column shear and flexural demand were checked with the reduced m factors above in the Column Shear Stress check and the Column- Bar splices. All column checks are compliant.

Stirrup Spacing
Tier 2: Section 5.5.2.3.7

f'ce= 6000 psi

Beams

b= 14 inch d= 28.5 inch

	Mn	L	Vp=2M/L	Vo (kips)		Max Vu (kips)	V/ bdf'c	m	Vn	mkVn	DCR
roof, M+	259.3	35	14.8	65.5	Flexure	6	0.19	3.0	14.8	40.0	0.1
roof, M-	340.2	35	19.4	65.5	Flexure	26	0.84	3.0	19.4	52.5	0.5
4th, M+	350.4	35	20.0	65.0	Flexure	6	0.19	3.0	20.0	54.1	0.1
4th, M-	624.5	35	35.7	65.0	Flexure	27	0.87	3.0	35.7	96.3	0.3
3rd, M+	402.7	35	23.0	65.2	Flexure	6	0.19	3.0	23.0	62.1	0.1
3rd, M-	522.8	35	29.9	65.2	Flexure	27	0.87	3.0	29.9	80.7	0.3
plaza, M+	350.4	35	20.0	65.2	Flexure	6	0.19	3.0	20.0	54.1	0.1
plaza, M-	624.5	35	35.7	65.2	Flexure	26	0.84	3.0	35.7	96.3	0.3

Girders

b= 14 inch d= 34.5 inch

	Mn	L	Vp=2M/L	Vo (kips)		Max Vu (kips)	V/ bdf'c	m	Vn	mkVn	DCR
roof, M+	948.9	23	82.5	106.5	Flexure	20	0.53	3.0	82.5	222.8	0.1
roof, M-	948.9	23	82.5	106.5	Flexure	68	1.82	3.0	82.5	222.8	0.3
4th, M+	411.5	23	35.8	106.5	Flexure	20	0.53	3.0	35.8	96.6	0.2
4th, M-	1106.8	23	96.2	106.5	Flexure	71	1.90	3.0	96.2	259.9	0.3
3rd, M+	858.3	23	74.6	106.7	Flexure	20	0.53	3.0	74.6	201.5	0.1
3rd, M-	1176.1	23	102.3	106.7	Flexure	69	1.84	3.0	102.3	276.1	0.2
plaza, M+	794.5	23	69.1	106.7	Flexure	13	0.34	3.0	69.1	186.5	0.1
plaza, M-	794.5	23	69.1	106.7	Flexure	60	1.60	3.0	69.1	186.5	0.3

All beams and girders have sufficient strengh for shear demands. Check is compliant

Adjacent Buildings
Tier 2: Section 5.4.1.2

The courthouse and annex are only a few inches apart directly over the entrance to the courthouse
This condition is to be mitigated to prevent pounding of the structures

Additional Retrofit Considerations
Conceptual Foundation Design

Per architectural drawings, geotechnical borings determined soil to be silty, fine grained sand with shale fragments with shale below.
Per CBC table 1806.2 allowable pressures:

Vertical Foundation Pressure:	1500	psf
qc = 3qallow =	4500	psf
m =	3	life safety, ASCE 41-13 8.4.2.3.2.1
q =	13500	psf

Shear Wall Gridline J

new ftg width =	10	ft
new ftg depth =	5	ft
length total =	120	ft
Overturning =	3305.6	kips each end
Deadload from cols =	690	k
Deadload from wall =	230	k
(e) ftg weight =	45	k
new ftg weight =	412.5	k
3x per ASCE 41 =	4132.5	k
DCR =	0.80	

Shear Wall Gridline M

new ftg width =	10	ft
new ftg depth =	5	ft
length total =	108	ft
Overturning =	3863.9	kips each end
Deadload from cols =	690	k
Deadload from wall =	230	k
(e) ftg weight =	45	k
new ftg weight =	367.5	k
3x per ASCE 41 =	3997.5	k
DCR =	0.97	

Shear Wall Gridline 2

new ftg width =	10	ft
new ftg depth =	5	ft
length total =	72	ft
Overturning =	3406.3	kips each end

Deadload from cols =	690	k
Deadload from wall =	230	k
(e) ftg weight =	45	k
new ftg weight =	232.5	k
3x per ASCE 41 =	3592.5	k
DCR =	0.95	

Shear Wall Gridline 7

new ftg width =	10.5	ft
new ftg depth =	5	ft
length total =	63	ft
Overturning =	3062.5	kips each end

Deadload from cols =	690	k
Deadload from wall =	230	k
(e) ftg weight =	45	k
new ftg weight =	208.6875	k
3x per ASCE 41 =	3521.063	k
DCR =	0.87	

Diaphragm and Drag Connection Check

	Fpx/Fx
Roof	1.0
4th	1.2
3rd	1.5
Plaza	2.7

Diaphragm Loads each Wall Line

	X	X	Y	Y
	Pier 2	Pier 7	Pier J	Pier M
roof	1608	1315	1290	1630
Story4	1384	1207	1451	1138
Story3	1042	930	1060	893
Story2	1088	1551	1056	1660

Diaphragm strength

$$Vn = 2 * t * L * \sqrt{f'c} + A_v f_y L/s$$

fye=	62.5	ksi	Av=	0.11	in2
f'ce=	6	ksi	s=	6.5	in
t slab=	4.5	inch			

Vn = 13.39 klf

Each wall length is 25' determine load to be transferred at wall:

m =	2	
mkVn =	24.1	klf
mkVn =	602.5	kips at wall
mkVn =	1205.1	kips at wall (where slab is both sides)

Remaining load to transfer to wall through beam drag

	X	X	Y	Y
	Pier 2	Pier 7	Pier J	Pier M
roof	403	712	85	425
Story4	179	605	246	0
Story3	0	328	0	0
Story2	0	948	0	0

Beam Name

	X	X	Y	Y
	Pier 2	Pier 7	Pier J	Pier M
roof	RG-2	none	RB-J	RB-M
Story4	4G-2	4B-m	4B-J	4B-M
Story3	3G-2	3B-m	3B-J	3B-M
Story2	2G-2	2B-m	2B-J	2B-M

	X	X	Y	Y
	Pier 2	Pier 7	Pier J	Pier M
roof	(4)#8	none	(2)#9+(2)#8	(6)#8
Story4	(4)#9	(2)#6+(2)#8	(4)#9	(4)#9
Story3	(4)#10	(2)#6+(2)#8	(4)#9	(4)#9
Story2	(2)#9+(2)#10	(2)#6+(2)#6	(4)#11	(4)#8

Existing Continuous Longitudinal steel

	X	X	Y	Y
	Pier 2	Pier 7	Pier J	Pier M
roof	3.16	0.00	3.58	4.74
Story4	4.00	2.46	4.00	4.00
Story3	5.08	2.46	4.00	4.00
Story2	4.54	1.76	6.24	3.16

fy= 50

F=fyAb force controlled

each side	X	X	Y	Y
	Pier 2	Pier 7	Pier J	Pier M
roof	316	0	358	474
Story4	400	246	400	400
Story3	508	246	400	400
Story2	454	176	624	316

remaining force to be transferred

	X	X	Y	Y	
	Pier 2	Pier 7	Pier J	Pier M	additional ft of diaphragm required PIER 7
roof	none	712	none	none	30
Story4	none	359	none	none	15
Story3	none	none	none	none	none
Story2	none	772	none	none	32

provide steel or frp drag at gridline 7
fy= 36

	Area of steel required for drag	
roof	14.8	50plf
Story4	7.5	25 plf

TABLE: Material Properties - Concrete								
Name	E	ν	α	G	Unit Weight	Unit Mass	Fc	Lightweight?
	lb/in²		1/F	lb/in²	lb/ft³	lb-s²/ft⁴	lb/in²	
4000Psi	3604996.5	0.2	0.0000055	1502081.88	150	4.662	4000	No

TABLE: Material Properties - Rebar						
Name	E	α	Unit Weight	Unit Mass	Fy	Fu
	lb/in²	1/F	lb/ft³	lb-s²/ft⁴	lb/in²	lb/in²
A615Gr40	29000000	0.0000065	490	15.23	40000	60000
A615Gr50	29000000	0.0000065	490	15.23	50000	60000
A615Gr60	29000000	0.0000065	490	15.23	60000	90000

TABLE: Frame Sections							
Name	Material	Shape	t3	t2	Area	AS2	AS3
			in	in	in²	in²	in²
CONC BEAM - B	4000Psi	Concrete Recta	25.5	14	357	297.5	297.5
CONC BEAM - G	4000Psi	Concrete Recta	31.5	14	441	367.5	367.5
CONC COL 1st	4000Psi	Concrete Recta	24	24	576	480	480
CONC COL 2nd (1)	4000Psi	Concrete Recta	24	24	576	480	480
CONC COL 2nd (2)	4000Psi	Concrete Recta	24	24	576	480	480
CONC COL 3rd	4000Psi	Concrete Recta	18	18	324	270	270
CONC COL 4th	4000Psi	Concrete Recta	18	18	324	270	270
ConcBm	4000Psi	Concrete Recta	24	18	432	360	360
ConcCol	4000Psi	Concrete Recta	18	18	324	270	270

TABLE: Shell Sections - Wall								
Name	Material	Element Type	Thickness	f11 Modifier	f22 Modifier	f12 Modifier	1 Modi	m22 Modifier
			in					
12" Wall	4000Psi	Shell-Thin	12	0.5	1	1	0.01	0.01

TABLE: Modal Periods and Frequencies			
Case	Mode	Period	Frequency
		sec	cyc/sec
Modal	1	0.512	1.952
Modal	2	0.481	2.078
Modal	3	0.342	2.923
Modal	4	0.288	3.468
Modal	5	0.261	3.828
Modal	6	0.249	4.014
Modal	7	0.245	4.087
Modal	8	0.237	4.214

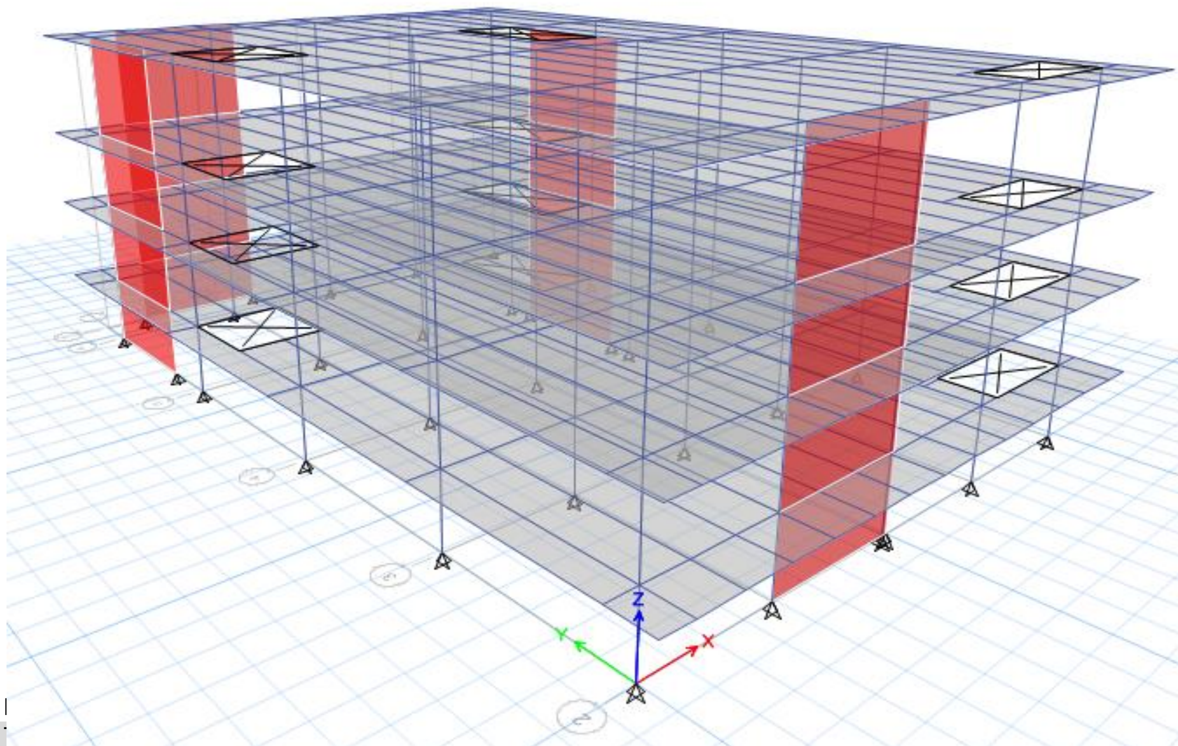
TABLE: Base Reactions

Load Case/Combo	FX	FY	FZ	
	kip	kip	kip	
Dead	0.00	0.00	6473.02	
Live	0.00	0.00	2616.60	
SuperDead	0.00	0.00	5228.53	
EQx	-7462.87	0.00	0.00	<- compare with calculated 7,434. OK
EQy	0.00	-7462.87	0.00	<- compare with calculated 7,434. OK
Env Dead	0.00	0.00	11701.54	<- compare with calculated 11,459. OK

TABLE: Load Combinations

Name	oad Case/Comb	Scale Factor	Type
Env Dead	Dead	1	Linear Add
Env Dead	SuperDead	1	
1.1(D+L)+EQx	Dead	1.1	Linear Add
1.1(D+L)+EQx	SuperDead	1.1	
1.1(D+L)+EQx	Live	1.1	
1.1(D+L)+EQx	EQx	1	
1.1(D+L)+EQy	Dead	1.1	Linear Add
1.1(D+L)+EQy	SuperDead	1.1	
1.1(D+L)+EQy	Live	1.1	
1.1(D+L)+EQy	EQy	1	
0.9D+EQx	Dead	0.9	Linear Add
0.9D+EQx	SuperDead	0.9	
0.9D+EQx	EQx	1	
0.9D+EQy	Dead	0.9	Linear Add
0.9D+EQy	SuperDead	0.9	
0.9D+EQy	EQy	1	
Eqx Combo ENV	1.1(D+L)+EQx	1	Envelope
Eqx Combo ENV	0.9D+EQx	1	
Eqy Combo ENV	1.1(D+L)+EQy	1	Envelope
Eqy Combo ENV	0.9D+EQy	1	

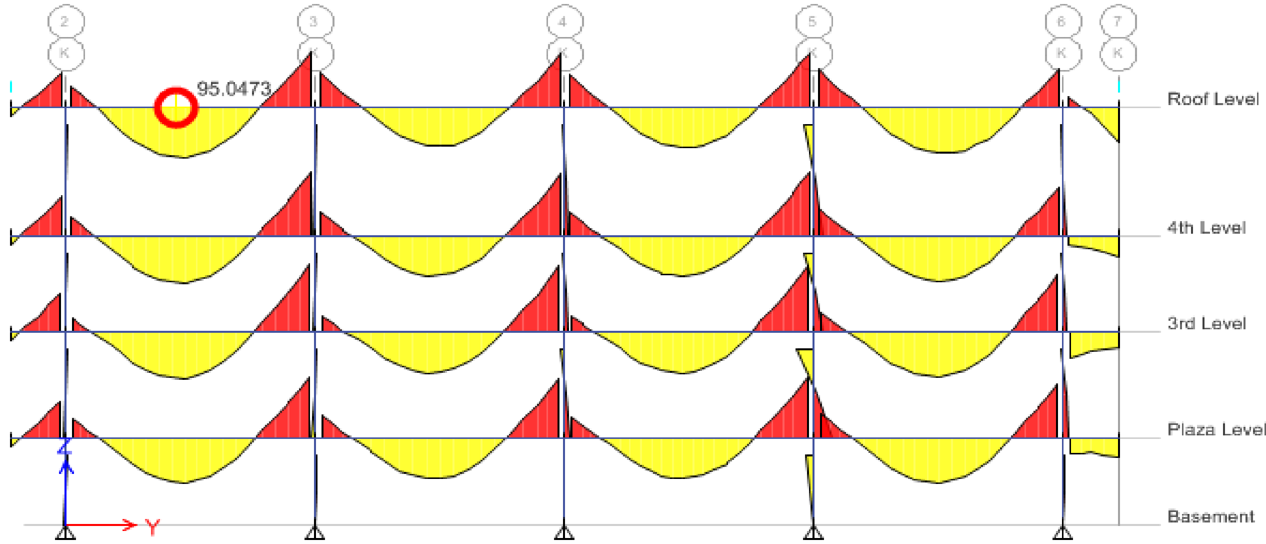
3D ETABS Model



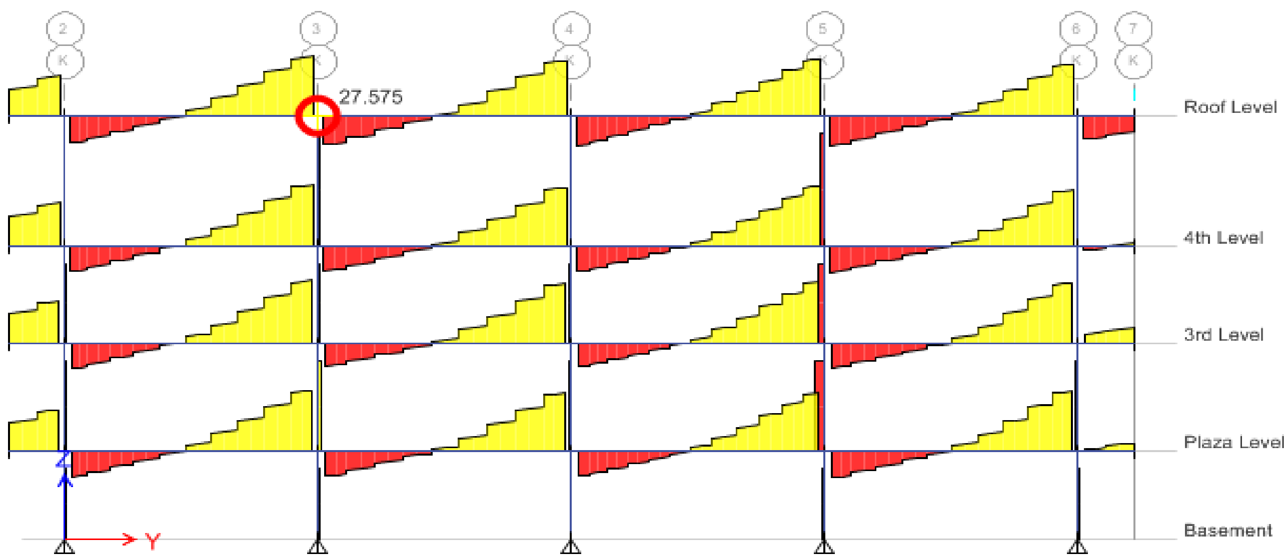
Story	Column	oad Case/Comb	Station ft	P kip	V2 kip	V3 kip	M2 kip-ft	M3 kip-ft
Story4	C1	Eqx Combo EN	0	-51.7	9.984	-2.732	-24.5	95.5123
Story4	C1	Eqx Combo EN	8.6875	-49.061	9.984	-2.732	-0.8	8.7767
Story4	C1	Eqx Combo EN	17.375	-46.422	9.984	-2.732	40.52	-70.9936
Story4	C2	Eqx Combo EN	0	-92.727	5.602	1.068	9.414	54.8928
Story4	C2	Eqx Combo EN	8.6875	-90.088	5.602	1.068	0.137	6.2274
Story4	C2	Eqx Combo EN	17.375	-87.449	5.602	1.068	-5.86	-29.9546
Story4	C3	Eqx Combo EN	0	-88.623	5.776	0.065	0.813	56.9299
Story4	C3	Eqx Combo EN	8.6875	-85.985	5.776	0.065	0.302	6.7519
Story4	C3	Eqx Combo EN	17.375	-83.346	5.776	0.065	-0.07	-32.9745
Story4	C4	Eqx Combo EN	0	-55.162	7.483	2.954	28.58	72.9903
Story4	C4	Eqx Combo EN	8.6875	-52.523	7.483	2.954	2.914	7.9783
Story4	C4	Eqx Combo EN	17.375	-49.884	7.483	2.954	-17.7	-51.542
Story4	C5	Eqx Combo EN	0	-4.964	13.079	0.132	1.101	128.1591
Story4	C5	Eqx Combo EN	8.6875	-2.325	13.079	0.132	-0.04	14.5373
Story4	C5	Eqx Combo EN	17.375	0.314	13.079	0.132	-1.1	-92.9694
Story4	C6	Eqx Combo EN	0	187.841	0.396	-2.646	-22.9	4.7265
Story4	C6	Eqx Combo EN	8.6875	190.48	0.396	-2.646	0.093	1.2867
Story4	C6	Eqx Combo EN	17.375	193.119	0.396	-2.646	43.73	-2.153
Story4	C7	Eqx Combo EN	0	-101.362	7.327	-0.435	-4.96	71.3718
Story4	C7	Eqx Combo EN	8.6875	-98.723	7.327	-0.435	-1.09	7.7184
Story4	C7	Eqx Combo EN	17.375	-96.084	7.327	-0.435	4.964	-53.9108
Story4	C8	Eqx Combo EN	0	-107.592	7.749	0.152	1.537	75.4042
Story4	C8	Eqx Combo EN	8.6875	-104.953	7.749	0.152	0.214	8.2303
Story4	C8	Eqx Combo EN	17.375	-102.314	7.749	0.152	-0.29	-58.8957
Story4	C9	Eqx Combo EN	0	-102.973	7.443	-0.137	-1.28	72.8656
Story4	C9	Eqx Combo EN	8.6875	-100.335	7.443	-0.137	-0.09	8.2002
Story4	C9	Eqx Combo EN	17.375	-97.696	7.443	-0.137	4.97	-52.797
Story4	C10	Eqx Combo EN	0	-110.998	4.059	2.166	20.2	39.2292
Story4	C10	Eqx Combo EN	8.6875	-108.359	4.059	2.166	1.379	3.9679
Story4	C10	Eqx Combo EN	17.375	-105.72	4.059	2.166	-2.46	-30.0651

Story4	C11	Eqx Combo EN	0	-215.181	0.396	-2.039	-17.2	4.6289
Story4	C11	Eqx Combo EN	8.6875	-212.542	0.396	-2.039	0.471	1.1901
Story4	C11	Eqx Combo EN	17.375	-209.903	0.396	-2.039	38.83	-2.2449
Story4	C12	Eqx Combo EN	0	-113.237	6.607	0.677	5.52	64.5796
Story4	C12	Eqx Combo EN	8.6875	-110.598	6.607	0.677	-0.21	7.1825
Story4	C12	Eqx Combo EN	17.375	-107.959	6.607	0.677	-4.53	-49.6683
Story4	C13	Eqx Combo EN	0	-101.498	8.14	0.51	4.868	79.2821
Story4	C13	Eqx Combo EN	8.6875	-98.859	8.14	0.51	0.44	8.5653
Story4	C13	Eqx Combo EN	17.375	-96.221	8.14	0.51	-0.36	-59.403
Story4	C14	Eqx Combo EN	0	-99.67	9.362	-0.757	-7.18	90.947
Story4	C14	Eqx Combo EN	8.6875	-97.032	9.362	-0.757	-0.6	9.6175
Story4	C14	Eqx Combo EN	17.375	-94.393	9.362	-0.757	13.05	-67.5457
Story4	C15	Eqx Combo EN	0	-132.545	11.7	0.104	-0.19	114.1759
Story4	C15	Eqx Combo EN	8.6875	-129.906	11.7	0.104	-1.1	12.6896
Story4	C15	Eqx Combo EN	17.375	-127.268	11.7	0.104	17.55	-88.7967
Story4	C16	Eqx Combo EN	0	-110.124	10.784	-1.268	-9.65	105.2658
Story4	C16	Eqx Combo EN	8.6875	-107.485	10.784	-1.268	1.364	11.5815
Story4	C16	Eqx Combo EN	17.375	-104.847	10.784	-1.268	29.82	-80.6763
Story4	C17	Eqx Combo EN	0	-110.417	7.287	-0.048	-1.16	71.6281
Story4	C17	Eqx Combo EN	8.6875	-107.779	7.287	-0.048	-0.7	8.3234
Story4	C17	Eqx Combo EN	17.375	-105.14	7.287	-0.048	1.368	-53.1027
Story4	C18	Eqx Combo EN	0	-93.364	7.593	2.849	27	74.7843
Story4	C18	Eqx Combo EN	8.6875	-90.725	7.593	2.849	2.245	8.8167
Story4	C18	Eqx Combo EN	17.375	-88.086	7.593	2.849	-8.48	-53.6105
Story4	C19	Eqx Combo EN	0	-53.695	7.435	-4.636	-44.5	73.3383
Story4	C19	Eqx Combo EN	8.6875	-51.056	7.435	-4.636	-4.27	8.7495
Story4	C19	Eqx Combo EN	17.375	-48.417	7.435	-4.636	53.12	-49.5066
Story4	C20	Eqx Combo EN	0	-88.286	8.606	3.523	31.42	84.829
Story4	C20	Eqx Combo EN	8.6875	-85.647	8.606	3.523	0.819	10.0825
Story4	C20	Eqx Combo EN	17.375	-83.009	8.606	3.523	-8.94	-63.0608
Story4	C21	Eqx Combo EN	0	-69.929	4.843	-2.193	-19.2	45.7235
Story4	C21	Eqx Combo EN	8.6875	-67.29	4.843	-2.193	-0.1	3.7879
Story4	C21	Eqx Combo EN	17.375	-64.652	4.843	-2.193	32.18	-36.3322
Story4	C22	Eqx Combo EN	0	-99.255	8.117	-0.382	-4.27	75.436
Story4	C22	Eqx Combo EN	8.6875	-96.616	8.117	-0.382	-0.95	4.9222
Story4	C22	Eqx Combo EN	17.375	-93.977	8.117	-0.382	4.543	-53.3389
Story4	C23	Eqx Combo EN	0	-94.627	8.331	-0.505	-5.14	76.9471
Story4	C23	Eqx Combo EN	8.6875	-91.988	8.331	-0.505	-0.75	4.5736
Story4	C23	Eqx Combo EN	17.375	-89.349	8.331	-0.505	5.457	-55.7802
Story4	C24	Eqx Combo EN	0	-99.272	8.876	-1.786	-16.7	82.2259
Story4	C24	Eqx Combo EN	8.6875	-96.633	8.876	-1.786	-1.2	5.1134
Story4	C24	Eqx Combo EN	17.375	-93.995	8.876	-1.786	19.71	-59.3662
Story4	C25	Eqx Combo EN	0	-81.528	9.632	3.572	31.89	90.9338
Story4	C25	Eqx Combo EN	8.6875	-78.889	9.632	3.572	0.856	7.2538
Story4	C25	Eqx Combo EN	17.375	-76.25	9.632	3.572	-11.8	-67.2357

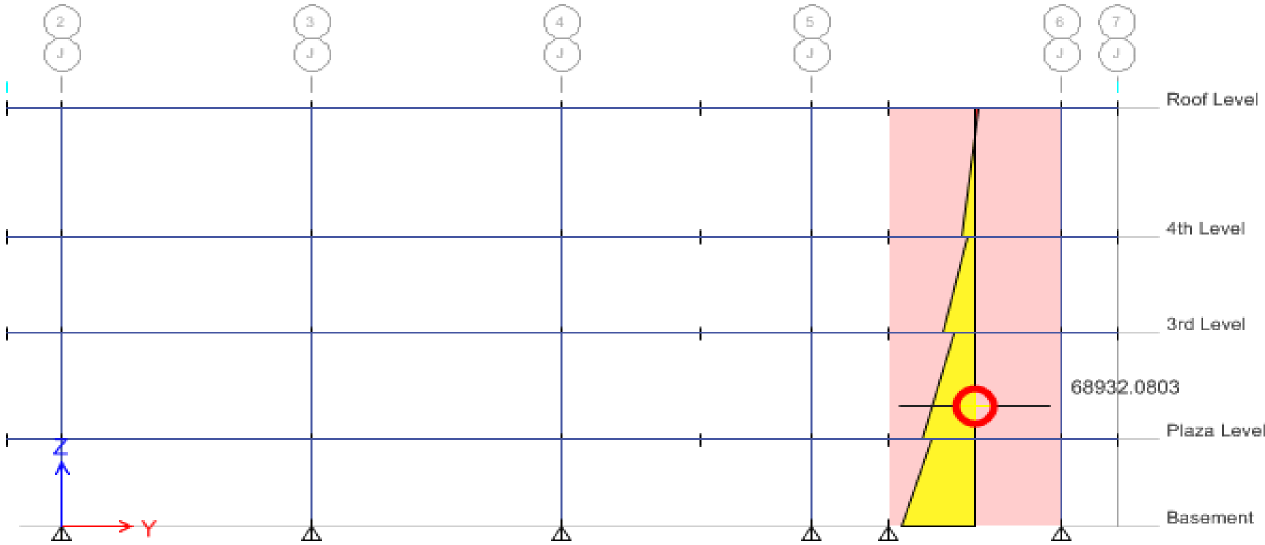
Example graphic of Beam Moment Demands under Max Eqy Combo at Gridline K (k-ft)



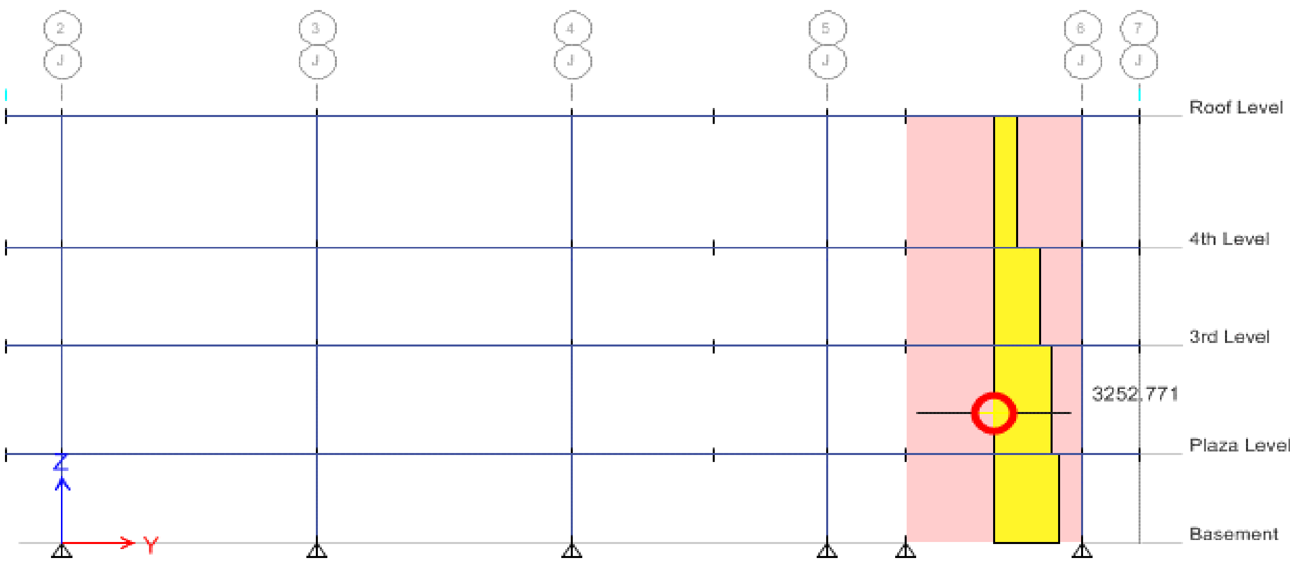
Example graphic of Beam Shear Demands under Max Eqy Combo at Gridline K (kips)



Example graphic of Shear Wall Moments under Max Eqy Combo at Gridline J (k-ft)



Example graphic of Shear Wall Shears under Max Eqy Combo at Gridline J (kips)



ZFA STRUCTURAL ENGINEERS

san francisco
silicon valley
sacramento
santa rosa
napa

County of Monterey Courthouse Parking Garage Structural Calculations

Monterey, California
ZFA Project Number: 17661

Final Report

June 30, 2018

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STRUCTURAL NARRATIVE

The following calculations support the ASCE 41-13 Tier 1 and Tier 2 checks for the County of Monterey Courthouse parking garage. Deficiencies remain after the Tier 2 evaluation. A Tier 2 deficiency base retrofit is then designed. Concrete shear walls are added to reduce seismic load to the existing structure and mitigate deficiencies to achieve compliance with Life Safety BSE-1E.

DETAILED DESIGN CRITERIA

BUILDING CODE

Governing Code:	2016 California Building Code
Authority Having Jurisdiction:	City of Monterey

SEOR STAMP

BUILDING SYSTEM DESCRIPTION

Date of Construction:	1966
No. Stories:	3
Footprint:	11000 ft^2
Floor Area:	33000 ft^2
Roof Area:	17000 ft^2
Mean Building Height:	28 ft
Roof Pitch:	0.00 :12

Building Use:	Parking Garage
Gravity System:	Concrete columns and beam
Diaphragm System:	Post tensioned slab
Foundation System:	Piles

SEISMIC DESIGN PARAMETERS

ASCE 41-13 Reference UNO:

Latitude:	36.590 deg	Longitude:	-121.880 deg	
Soil Site Class =	D	Per Geotech Report, Site Class D otherwise		*USGS*
Risk Category:	II			*Section 2.4.1.6.1*
Diaphragm=	Rigid Diaphragm	Post tensioned slab		
Building System, N-S:	C1	Concrete Moment Frame		*Table 3-1*
Building System, E-W:	C1	Concrete Moment Frame		*Table 3-1*
$C_{t, N-S}$ =	0.02	Approximate Period Parameter, Ct, N-S		*Section 4.5.2.4*
$C_{t, E-W}$ =	0.02	Approximate Period Parameter, Ct, E-W		*Section 4.5.2.4*
β_{N-S} =	0.90	Approximate Period Parameter, β, N-S		*Section 4.5.2.4*
β_{E-W} =	0.90	Approximate Period Parameter, β, E-W		*Section 4.5.2.4*
$T_{a, N-S}$=	0.361 sec	Approximate Fundamental Period, N-S		*Section 4.5.2.4*
$T_{a, E-W}$ =	0.361 sec	Approximate Fundamental Period, E-W		*Section 4.5.2.4*

TIER 1 SEISMIC EVALUATION PARAMETERS

Performance Objective:	LS	Life Safety	*Table 2-1*
Seismic Hazard Level:	BSE-1E	20%/50 years	*Table 2-1*
S_S =	0.541 g	Mapped spectral response acceleration parameter	*USGS*
S_1 =	0.193 g	Mapped spectral response acceleration parameter	*USGS*
S_{XS} =	0.739 g	Mapped spectral response acceleration parameter	*USGS*
S_{X1} =	0.391 g	Mapped spectral response acceleration parameter	*ASCE 7 Table 1.5-1*
Seismicity:	High		*Table 2-5*
$S_{a, N-S}$ =	0.739 g	Spectral Response Acceleration, N-S	*Section 4.5.2.3*
$S_{a, E-W}$ =	0.739 g	Spectral Response Acceleration, E-W	*Section 4.5.2.3*
C_{N-S} =	1.000	Modification Factor	*Table 4-8*
C_{E-W} =	1.000	Modification Factor	*Table 4-8*
V_{N-S} =	0.739 *W	Pseudo-Seismic Base Shear, N-S	*Section 4.5.2.1*
V_{E-W} =	0.739 *W	Pseudo-Seismic Base Shear, N-S	*Section 4.5.2.1*

TIER 2 / 3 SEISMIC EVALUATION PARAMETERS

Performance Objective:	LS	Life Safety	*Table 2-1*
Seismic Hazard Level:	BSE-1E	20%/50 years	*Table 2-1*
S_S =	0.541 g	Mapped spectral response acceleration parameter	*USGS*
S_1 =	0.193 g	Mapped spectral response acceleration parameter	*USGS*
S_{XS} =	0.739 g	Mapped spectral response acceleration parameter	*USGS*
S_{X1} =	0.391 g	Mapped spectral response acceleration parameter	*USGS*
Seismicity:	High		*Table 2-5*
$C_{1,\ N\text{-}S}$ =	1.000	Inelastic-to-elastic displacement factor	*Equation 7-22*
$C_{2,\ N\text{-}S}$ =	1.000	Hysteresis shape factor	*Equation 7-23*
Alternate $(C_1C_2)_{N\text{-}S}$ =	1.100	2 ≤ mmax < 6	*Table 7-3*
Use Alternate $(C_1C_2)_{N\text{-}S}$?	Yes		
$(C_1C_2)_{N\text{-}S}$ =	1.100		
$C_{m,\ N\text{-}S}$ =	0.900	Effective mass factor	*Table 7-4*
$C_{1,\ E\text{-}W}$ =	1.000	Inelastic-to-elastic displacement factor	*Equation 7-22*
$C_{2,\ E\text{-}W}$ =	1.000	Hysteresis shape factor	*Equation 7-23*
Alternate $(C_1C_2)_{E\text{-}W}$ =	1.100	2 ≤ mmax < 6	*Table 7-3*
Use Alternate $(C_1C_2)_{E\text{-}W}$?	Yes		
$(C_1C_2)_{E\text{-}W}$ =	1.100		
$C_{m,\ E\text{-}W}$ =	0.900	Effective mass factor	*Table 7-4*
$S_{a,N\text{-}S}$ =	0.737	Spectral Response Acceleration	*Equation 2-9*
$S_{a,E\text{-}W}$ =	0.737	Spectral Response Acceleration	*Equation 2-9*
$V_{N\text{-}S}$ (ULT) =	0.730 *W	Pseudo-Seismic Base Shear, N-S	*Equation 7-21*
$V_{E\text{-}W}$ (ULT) =	0.730 *W	Pseudo-Seismic Base Shear, E-W	*Equation 7-21*

MATERIAL STRENGTH AND SPECIFICATIONS

ASCE 41-13 Reference UNO:

CONCRETE:

Knowledge Factor, κ	0.75	Concrete Knowledge Factor	*Table 6-1*
Foundations, f'_c =	3000 psi	Default Lower Bound: 2500 psi - 3000 psi	*Table 10-2*
Foundations, f'_{ce} =	4500 psi		*Table 10-1*
Slab on grade, f'_c =	3000 psi	Default Lower Bound: 3000 psi - 4000 psi	*Table 10-2*
Slab on grade, f'_{ce} =	4500 psi		*Table 10-1*
Structural walls, f'_c =	3000 psi	Default Lower Bound: 2500 psi - 4000 psi	*Table 10-2*
Structural walls, f'_{ce} =	4500 psi		*Table 10-1*
Beams, f'_c =	4000 psi	Default Lower Bound: 3000 psi - 4000 psi	*Table 10-2*
Beams, f'_{ce} =	6000 psi		*Table 10-1*
Columns, f'_c =	4000 psi	Default Lower Bound: 3000 psi - 6000 psi	*Table 10-2*
Columns, f'_{ce} =	6000 psi		*Table 10-1*
Fill over metal deck, f'_c =	3000 psi	Default Lower Bound: 3000 psi - 4000 psi	*Table 10-2*
Fill over metal deck, f'_{ce} =	4500 psi		*Table 10-1*
Elevated slabs, f'_c =	4000 psi	Default Lower Bound: 3000 psi - 4000 psi	*Table 10-2*
Elevated slabs, f'_{ce} =	6000 psi		*Table 10-1*
Weight of Concrete =	150 pcf		
Weight of Light Wt. Concrete =	110 pcf		

CONCRETE REINFORCING:

Knowledge Factor, κ	0.75	Reinforcing Knowledge Factor	*Table 6-1*
Reinforcing Steel, f_y =	40 ksi	Default Lower-Bound: 33, 40, 50, 60, 65, 70 ksi	*Table 10-3*
Reinforcing Steel, f_{ye} =	50 ksi		*Table 10-1*
Reinforcing Steel, f_y =	70 ksi	Default Lower-Bound: 55, 70, 80, 90, 75, 80, 100 ksi	*Table 10-3*
Reinforcing Steel, f_{ye} =	87.5 ksi		*Table 10-1*
Reinforcing Steel ties, f_y =	40 ksi	Default Lower-Bound: 33, 40, 50, 60, 65, 70 ksi	*Table 10-3*
Reinforcing Steel ties, f_{ye} =	50 ksi		*Table 10-1*
Reinforcing Steel ties, f_y =	70 ksi	Default Lower-Bound: 55, 70, 80, 90, 75, 80, 100 ksi	*Table 10-3*
Reinforcing Steel ties, f_{ye} =	87.5 ksi		*Table 10-1*

GRAVITY / SEISMIC FLAT WEIGHT TAKEOFF (PSF)

Typical Floor

CBC Live Load Category: 14. Garages: passenger [Table 1607.1]

Material	Sloped	Deck	Joists	Girders	Columns	Model	Seismic	Model Seismic
Finish		0.0	0.0	0.0	0.0	0.0	0.0	0.0
Solar / Other		0.0	0.0	0.0	0.0	0.0	0.0	0.0
7" concrete slab		87.5	87.5	87.5	87.5	87.5	87.5	87.5
		0.0	0.0	0.0	0.0	0.0	0.0	0.0
Insulation		0.0	0.0	0.0	0.0	0.0	0.0	0.0
M.E.P.		2.0	2.0	2.0	2.0	2.0	2.0	2.0
Ceiling - 7/8" plaster			0.0	0.0	0.0	0.0	0.0	0.0
Sprinklers			1.5	1.5	1.5	1.5	1.5	1.5
Joists			0.0	0.0	0.0		0.0	
Girders				13.2	13.2		13.2	
Columns					6.9		6.9	
Misc.		5.0	5.0	5.0	5.0	5.0	5.0	5.0
Dead Load		94.5	96.0	109.2	116.1	96.0	116.1	96.0
Dead Load - Horiz Projection		94.5	96.0	109.2	116.1	96.0	116.1	96.0
Partitions		0.0	0.0	0.0	0.0	0.0	0.0	0.0
Live Load		40.0	40.0	40.0	40.0	40.0	0.0	0.0
Live Load - Reduced R_2 = 1.00		40.0	40.0	40.0	40.0	40.0	0.0	0.0
Total Load		134.5	136.0	149.2	156.1	136.0	116.1	96.0

Roof

CBC Live Load Category: 14. Garages: passenger [Table 1607.1]
Slope: 0.00:12

Material	Sloped	Deck	Joists	Girders	Columns	Model	Seismic	Model Seismic
Finish		0.0	0.0	0.0	0.0	0.0	0.0	0.0
Solar / Other		0.0	0.0	0.0	0.0	0.0	0.0	0.0
Topping		87.5	87.5	87.5	87.5	87.5	87.5	87.5
Sheathing / Decking		0.0	0.0	0.0	0.0	0.0	0.0	0.0
Insulation		0.0	0.0	0.0	0.0	0.0	0.0	0.0
M.E.P.		2.0	2.0	2.0	2.0	2.0	2.0	2.0
Ceiling			0.0	0.0	0.0	0.0	0.0	0.0
Sprinklers			1.5	1.5	1.5	1.5	1.5	1.5
Joists			0.0	0.0	0.0		0.0	
Girders				13.2	13.2		13.2	
Columns					3.5		3.5	
Misc.		5.0	5.0	5.0	5.0	5.0	5.0	5.0
Dead Load		94.5	96.0	109.2	112.6	96.0	112.6	96.0
Partitions		0.0	0.0	0.0	0.0	0.0	0.0	0.0
Live Load		40.0	40.0	40.0	40.0	40.0	0.0	0.0
Total Load		134.5	136.0	149.2	152.6	136.0	112.6	96.0

Diaphragm Weight Information:

Level	Area (sq ft)	Diaphragm Unit Weight (psf)	Diaphragm Weight (kips)	Wall Unit Weight (psf)	Wall Trib Height (ft)	Wall Length (ft)	Wall Weight (kips)	Level Weight (kips)
ROOF	11102	113	1,251	0	0.0	0.0		**1,251**
2nd	11102	116	1,289	0	0.0	0.0		**1,289**
1st	11102	116	1,289	0	0.0	0.0		**1,289**
Σ			3,829				0	**3,829**

Vertical Distribution (LSP)-TIER 1

Vertical Distribution of Seismic Forces (North-South) *Y Direction* *ASCE 41-13 §7.4.1.3.2*

$V_{N/S}$ = 2,829 *kips* Total Base Shear
k = 1.0 For T = 0.36 sec

	w_x (kips)	h_x (ft)	$w_x h^k_x$	C_{vx}	F_x (kips)	V_x (kips)
Roof	1,251	28.00	35,017	0.49	1,381	1,381
2nd Level	1,289	19.00	24,491	0.34	966	2,346
1st Level	1,289	9.50	12,245	0.17	483	2,829
Total	*3,829*		*71,753*	*1.00*	*2,829*	

Vertical Distribution of Seismic Forces (East-West) *X Direction* *ASCE 41-13 §7.4.1.3.2*

$V_{E/W}$ = 2,829 *kips* Total Base Shear
k = 1.0 For T = 0.36 sec

	w_x (kips)	h_x (ft)	$w_x h^k_x$	C_{vx}	F_x (kips)	V_x (kips)
Roof	1,251	28.00	35,017	0.49	1,381	1,381
2nd Level	1,289	19.00	24,491	0.34	966	2,346
1st Level	1,289	9.50	12,245	0.17	483	2,829
Total	*3,829*		*71,753*	*1.00*	*2,829*	

SEISMIC HAZARD ANALYSIS
ASCE 41-13 §2.4

Site Coordinates

Latitude =	36.5901	deg	County of Monterey Courthouse
Longitude =	-121.8804	deg	Monterey, California

Spectral Response Acceleration Parameters
ASCE 41-13 §2.4.1.4

Site Class =	**D**		Site Soil Classification
S_S =	0.541	g	Mapped Short-period Spectral Response Acceleration
S_1 =	0.193	g	Mapped 1-sec period Spectral Response Acceleration
S_{XS} =	0.739	g	Short-period Spectral Response Acceleration at BSE-1E
S_{X1} =	0.391	g	1-sec period Spectral Response Acceleration at BSE-1E

SEISMIC FORCE
ASCE 41-13 §4.5.2

Building Properties

Type N/S =	C1		Building Type in North-South Direction	*ASCE 41-13 Table 3-1*
Type E/W =	C1		Building Type in East-West Direction	*ASCE 41-13 Table 3-1*
Height, h_n =	28.00	ft	Height above base to roof level	
Stories =	3		Number of stories	
Weight N/S =	3828.6	k	Seismic Weight of Building in North-South Direction	
Weight E/W =	3828.6	k	Seismic Weight of Building in East-West Direction	

Building Period
ASCE 41-13 §4.5.2.4

North-South Direction:

C_t =	0.02		Period Adjustment Factor	
β =	0.90		Empirical Fundamental Period Adjustment Factor	
T =	0.361	sec	Fundamental Period	$=C_t*h_n^{\beta}$

East-West Direction:

C_t =	0.02		Period Adjustment Factor	
β =	0.90		Empirical Fundamental Period Adjustment Factor	
T =	0.361	sec	Fundamental Period	$=C_t*h_n^{\beta}$

Pseudo-Seismic Force
ASCE 41-13 §4.5.2.1

North-South Direction:

S_a =	0.74	g	Spectral Response Acceleration	$=S_{x1}/T < S_{xs}$
C =	1.00		Modification Factor	*Table 4-8*
V =	0.74	*W	Pseudo-Seismic Force in Terms of Weight	*=C*Sa*W*
V =	**2829.3**	k	Pseudo-Seismic Force	

East-West Direction:

S_a =	0.74	g	Spectral Response Acceleration	$=S_{x1}/T < S_{xs}$
C =	1.00		Modification Factor	*Table 4-8*
V =	0.74	*W	Pseudo-Seismic Force in Terms of Weight	*=C*Sa*W*
V =	**2829.3**	k	Pseudo-Seismic Force	

Column Axial Stress Caused by Overturning

f'c=	4000	psi
0.3f'c=	1200	psi

Eqn (4-12)

$$p = \frac{1}{M_s}\left(\frac{2}{3}\right)\left(\frac{Vh_n}{Ln_f}\right)\left(\frac{1}{A_{col}}\right)$$ (Section 4.5.3.6)

N-S Direction			E-W Direction		
nf =	8		nf =	2	
V =	2829	kips	V =	2829	kips
hn =	28	ft	hn =	28	ft
L =	61	ft	L =	182	ft
Ms =	2		Ms =	2	
Acol =	4	ft2	Acol =	4	ft2
p=	13.5	k/ft2	p=	18.1	k/ft2
p=	93.9	psi	p=	126	psi
DCR=	0.08	OK	DCR=	0.1	OK

Compliant

Column Shear Stress Check

$$v = \frac{1}{M_s}\left(\frac{n_c}{n_c - n_f}\right)\left(\frac{Vh_n}{Ln_f}\right)$$ (Section 4.5.3.2)

stress check < 126.5 psi

N-S Direction

nc =	16
nf =	8
Ms =	2

	Ac	Vj	vj	vj	DCR	
	ft2	kips	k/ft2	psi		
2nd	64	1,381	21.6	149.8	1.2	Non Compliant
1st	64	2,346	36.7	254.6	2.0	Non Compliant
Ground	89	2,829	31.8	221.0	1.7	Non Compliant

E-W Direction

nc =	16
nf =	2
Ms =	2

	Ac	Vj	vj	vj	DCR	
	ft2	kips	k/ft2	psi		
2nd	64	1,381	12.3	85.6	0.7	OK
1st	64	2,346	21.0	145.5	1.2	Non Compliant
Ground	89	2,829	18.2	126.3	1.0	OK

No floors pass the column shear stress test, check is non-compliant.

Prestressed Frame Elements

Max. Fe in slab =	23	kip/ft	Max. Fe in beam =	635	kip
slab t =	7	inch	beam + slab area =	2271	inch
fp=	274	psi	fp=	280	psi

Max allowed= 700 psi or 666.7 psi

Check is compliant

No Shear Failures

f'c= 4000 psi   f'c= 5000 psi - 1st flr & 2nd flr

	b (in)	d (in)	area (in2)	stl area (in2)	ρl	Av	s (in)	fyl (ksi)	fyv (ksi)
2nd	24	24	576	6.24	0.01083	0.2	12	60	60
1st	24	24	576	6.24	0.01083	0.2	11.5	60	60
Ground	-	32	804	9.36	0.01164	0.11	2.25	60	60

	V Vc+Vs	P demand	Mp Spcol	L	V_p = 2M/L	Failure Mechanism
2nd	95.4	85.9	375	7.5	100.0	SHEAR
1st	104.9	174.4	450	8	112.5	SHEAR
Ground	203.2	263.0	800	8	200.0	FLEXURE

First and second floor columns have shear failures, therefore the check is non-compliant.

Typical Beam

Properties

f'_c = **4000** psi
f_y = **40** ksi
f_{yt} = **40** ksi

Shear Reinf.

2 Legs
#3 **@ 4** "oc

Beam Dimensions

Width = **24.0** in
Depth = **18.5** in
Cover = **1.5** in
d = **16.1** in

V_c = **49.0** k
V_s = **35.5** k

Pre-Stressed Concrete Beam Checks

			Failure Mechanism
Lower Limit:	V_n =	**84.4** k	Flexure
Upper Limit:	V_n =	**157.9** k	

V_p= 2M/L   Shear based on flexural capacity
V_p= **41.2** k

All beams are flexure controlled, check is compliant.

Strong Column - Weak Beam

N-S Direction

	Mcol	1.2(Mbm)	
2nd	375	1124	Non Compliant
1st	900	1087	Non Compliant
Ground	1250	1087	OK

E-W Direction

	Mcol	1.2(Mbm)	
2nd	375	159	OK
1st	900	159	OK
Ground	1250	159	OK

The beam-column direction is non compliant except at the ground floor. The slab-column direction is compliant.

Beam Bars

Based on existing drawing sheet S1 all beams have (2)#8 continuous top and bottom.
Check is compliant.

Column Bar Splices

	rebar	35db (inches)	per S1 (inches)	
2nd	#11	49	no splice	OK
1st	#11	49	no splice	OK
Ground	#11	49	63	OK

Column splices at the 1st level are compliant. No other column splices per sheet S1. Check is compliant.

Beam Bar Splices

Lap splices are located at lb/4 from center of joint. Check is compliant

Column Tie Spacing

	d/4 (inches)	8db (inches)	spacing (inches)	
32" TYP	6.4	-	2.5	OK
24" TYP	6.0	-	12	Non Compliant
32" hinge	-	3.0	2.25	OK
24" hinge (3)	-	4.0	6	Non Compliant
24" hinge (2)	-	4.0	2.25	OK

Check is non-compliant for some of the column conditions.

Stirrup Spacing

	d/2 (inches)	min(8db,d/4) (inches)	spacing (inches)	
Beam TYP	8.5	-	18	Non Compliant
Beam hinge	-	4.3	4	OK

Check is non-compliant.

Joint Transverse Reinforcing
Non-compliant based on two checks above

Overturning

least horizontal dimension / building height =	1.2	
0.6Sa =	0.4	OK

Compliant

Existing Beam Flexural Capacity

Beam: Typ Beam, M+

Properties			Flexural Reinf.			Effective Prestress		
f'_c =	4000	psi	4	#8	bars	Beam		
f_y =	50	ksi	A_s =	3.16	in^2	F_e =	325	kips
f_{yt} =	50	ksi	a =	1.04	in	dp=	6.5	in
			β_1 =	0.85				
Beam Dimensions			c =	1.23	in	slab contribution is negligible		
Width =	136.0	in	ε_s =	0.036				
Depth =	18.5	in						
Cover =	1.5	in	M_n =	203.8	k-ft			
d =	16.0	in	$M_{n,ps}$ =	161.9	k-ft			
			$M_{n,\ total}$ =	365.7	k-ft			

Beam: Typ Beam, M-

Properties			Flexural Reinf.			Effective Prestress		
f'_c =	4000	psi	4	#8	bars	Beam		
f_y =	50	ksi	4	#4	bars	F_e =	325	kips
f_{yt} =	50	ksi	A_s =	3.96	in^2	dp=	12.25	in
			a =	8.82	in			
Beam Dimensions			β_1 =	0.85		Slab		
Width =	24.0	in	c =	10.38	in	F_e =	197	kips
Depth =	18.5	in	ε_s =	0.002		dp=	12.75	in
Cover =	1.5	in						
d =	16.0	in	M_n =	191.2	k-ft			
			$M_{n,ps}$ =	349.2	k-ft			
			$M_{n,\ total}$ =	540.4	k-ft			

Beam: 3rd Flr Beam, M-

Properties			Flexural Reinf.			Effective Prestress		
f'_c =	4000	psi	4	#8	bars	Beam		
f_y =	50	ksi	4	#4	bars	F_e =	325	kips
f_{yt} =	50	ksi	A_s =	3.96	in^2	dp=	12.25	in
			a =	10.21	in			
Beam Dimensions			β_1 =	0.85		Slab		
Width =	24.0	in	c =	12.01	in	F_e =	310	kips
Depth =	18.5	in	ε_s =	0.001		dp=	12.75	in
Cover =	1.5	in						
d =	16.0	in	M_n =	179.8	k-ft			
			$M_{n,ps}$ =	391.1	k-ft			
			$M_{n,\ total}$ =	570.8	k-ft			

Existing Slab Flexural Capacity

ASCE 41-13, section 10.4.4.3

1 be=	3.75 ft	column =	2 ft
2 be=	4.92 ft	slab thickness =	0.58 ft
b1=	2.58 ft	γf =	0.6
b2=	2.58 ft	γv =	0.4

Beam: Slab, M+, ASCE 41 10.4.4.3 #1

Properties		Flexural Reinf.		Effective Prestress	
f'_c =	4000 psi	2.5	#5 bars		
f_y =	50 ksi	A_s =	0.78 in^2	F_e =	86.25 kips
f_{yt} =	50 ksi	a =	0.82 in	dp=	5.75 in
		β_1 =	0.85		
Beam Dimensions		c =	0.96 in		
Width =	45.0 in	ε_s =	0.012		
Depth =	7.0 in	M_n =	13.8 k-ft		
Cover =	1.5 in	$M_{n,ps}$ =	38.4 k-ft		
d =	4.7 in	$M_{n, total}$ =	52.2 k-ft		

Beam: Slab, M-, ASCE 41 10.4.4.3 #1

Properties		Flexural Reinf.		Effective Prestress	
f'_c =	4000 psi	2.5	#5 bars		
f_y =	50 ksi	A_s =	0.78 in^2	F_e =	86.25 kips
f_{yt} =	50 ksi	a =	0.82 in	dp=	1.25 in
		β_1 =	0.85		
Beam Dimensions		c =	0.96 in		
Width =	45.0 in	ε_s =	0.001		
Depth =	7.0 in	M_n =	2.5 k-ft		
Cover =	5.0 in	$M_{n,ps}$ =	6.0 k-ft		
d =	1.2 in	$M_{n, total}$ =	8.6 k-ft		

∑Mn = 60.8 kips

∑Mn/γv = 151.9 kips

Beam: Slab, M+, ASCE 41 10.4.4.3 #2

Properties			**Flexural Reinf.**			**Effective Prestress**		
f'_c =	4000	psi	3.28	#5	bars			
f_y =	50	ksi	A_s =	1.02	in^2	F_e =	113.1	kips
f_{yt} =	50	ksi	a =	0.82	in	dp=	5.75	in
			β_1 =	0.85				
Beam Dimensions			c =	0.96	in			
Width =	59.0	in	ε_s =	0.012				
Depth =	7.0	in	M_n =	18.1	k-ft			
Cover =	1.5	in	$M_{n,ps}$ =	50.3	k-ft			
d =	4.7	in	$M_{n, total}$ =	68.5	k-ft			

Beam: Slab, M-, ASCE 41 10.4.4.3 #2

Properties			**Flexural Reinf.**			**Effective Prestress**		
f'_c =	4000	psi	3.28	#5	bars			
f_y =	50	ksi	A_s =	1.02	in^2	F_e =	113.1	kips
f_{yt} =	50	ksi	a =	0.82	in	dp=	1.25	in
			β_1 =	0.85				
Beam Dimensions			c =	0.96	in			
Width =	59.0	in	ε_s =	0.001				
Depth =	7.0	in	M_n =	3.3	k-ft			
Cover =	5.0	in	$M_{n,ps}$ =	7.9	k-ft			
d =	1.2	in	$M_{n, total}$ =	11.2	k-ft			

$\sum Mn$ = 79.7 kips
$\sum Mn/\gamma f$ = 132.8 kips

SEISMIC HAZARD ANALYSIS (TIER 2)

ASCE 41-13 §2.4.1

Site Coordinates

Latitude = 36.5901 deg — County of Monterey Courthouse
Longitude = -121.880 deg — Monterey, California

Spectral Response Acceleration Parameters

ASCE 41-13 §2.4.1.4

Site Class =	D		Site Soil Classification
S_S =	0.541	g	Mapped Short-period Spectral Response Acceleration
S_1 =	0.193	g	Mapped 1-sec period Spectral Response Acceleration
S_{XS} =	0.739	g	Short-period Spectral Response Acceleration at BSE-1E
S_{X1} =	0.391	g	1-sec period Spectral Response Acceleration at BSE-1E

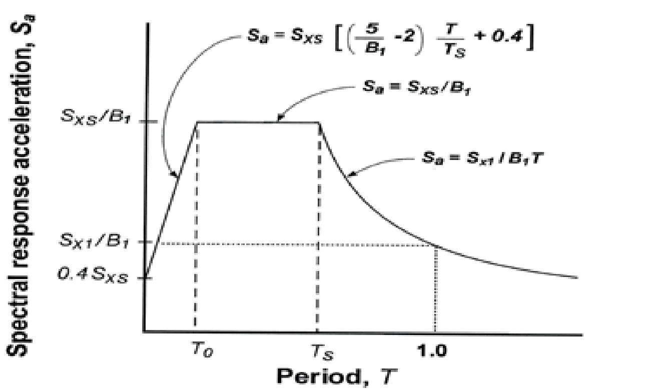
General Horizontal Response Spectrum

ASCE 41-13 §2.4.1.7

β =	5%		Effective Viscous Damping Ratio	
B_1 =	1.00		Damping Coefficient	*= 4 / (5.6 - ln(100β))*
T_S =	0.53	sec	Period at Constant Velocity Region	*$= S_{X1} / S_{XS}$*
T_0 =	0.11	sec	Period at Constant Acceleration Region	*$= 0.2\, T_S$*
$0.4S_{XS}$=	0.30	g	Peak Ground Acceleration	
S_{XS}/B_1 =	0.74	g	Short period Spectral Response Acceleration	
S_{X1}/B_1 =	0.39	g	1-sec period Design Spectral Response Acceleration	

Spectral Acceleartion at Building Period

ASCE 41-13 §2.4



ASCE 31-13 Figure 2-1: General Horizontal Response Spectrum

Existing Building

North-South Direction

T=	0.361	sec		*Per LSP calcs*
S_a=	**0.737**			*Eq. 2-9*

East-West Direction

T=	0.361	sec	Building period in N-S direction	*Per LSP calcs*
S_a=	**0.737**		Spectral Response Acceleration at Building Period	*Eq. 2-9*

Retrofit Building

North-South Direction

T=	0.243	sec	Building period in N-S direction	*Per LSP calcs*
S_a=	**0.737**		Spectral Response Acceleration at Building Period	*Eq. 2-10*

East-West Direction

T=	0.361	sec	Building period in N-S direction	*Per LSP calcs*
S_a=	**0.737**		Spectral Response Acceleration at Building Period	*Eq. 2-10*

LINEAR STATIC PROCEDURE (LSP)-TIER 2

ASCE 41-13 §7.4.1

Stories = 3 — Number of stories in building

Period Determination for LSP - Method 2 - Empirical

ASCE 41-13 §7.4.1.2

North-South Direction

Building Type =	C1			
C_t =	0.018		Factor for adjustment of period	*ASCE 41-13 §7.4.1.2.2*
β =	0.90		Factor for adjustment of period	
h_n =	28.0	ft	Roof Height	
T =	**0.361**	sec	Building period in N-S direction	*ASCE 41-13 Eq. 7-18*

East-West Direction

Building Type =	C1			
C_t =	0.018		Factor for adjustment of period	
β =	0.90		Factor for adjustment of period	
h_n =	28.0	ft	Roof Height	
T =	**0.361**	sec	Building period in E-W direction	*ASCE 41-13 Eq. 7-18*

Pseudo-Seismic Force for LSP (North-South)

ASCE 41-13 §7.4.1.3.1

$V = C_1 C_2 C_m S_a W$			Pseudo-Lateral Force	*ASCE 41-13 Eq. 7-21*
C_1 =	1.0		Modification Factor, Inelastic Displacements	*ASCE 41-13 Eq. 7-22*
C_2 =	1.0		Modification Factor, Cyclic Behavior	*ASCE 41-13 Eq. 7-23*
$C_1 C_2$ =	1.1		Alternative Value for Modification Factors	*ASCE 41-13 Table 7-3*
Use alternate C_1C_2?	Yes			
C_m =	0.9		Effective Mass Factor	*ASCE 41-13 Table 7-4*
$S_a(T)$ =	**0.737**	g	Spectral Response Acceleration	*for T = 0.36sec*
W =	3828.6	kips	Effective Seismic Weight	
$V_{N/S}$ =	**0.730**	*W	Pseudo-Lateral Force	
$V_{N/S}$ =	**2,794.4**	kips	Pseudo-Lateral Force	

Pseudo-Seismic Force for LSP (East-West)

ASCE 41-13 §7.4.1.3.1

$V = C_1 C_2 C_m S_a W$			Pseudo-Lateral Force	*ASCE 41-13 Eq. 7-21*
C_1 =	1.0		Modification Factor, Inelastic Displacements	*ASCE 41-13 Eq. 7-22*
C_2 =	1.0		Modification Factor, Cyclic Behavior	*ASCE 41-13 Eq. 7-23*
$C_1 C_2$ =	1.1		Alternative Value for Modification Factors	*ASCE 41-13 Table 7-3*
Use alternate C_1C_2?	Yes			
C_m =	0.9		Effective Mass Factor	*ASCE 41-13 Table 7-4*
$S_a(T)$ =	**0.737**	g	Spectral Response Acceleration	*for T = 0.36sec*
W =	3828.6	kips	Effective Seismic Weight	
$V_{E/W}$ =	**0.730**	*W	Pseudo-Lateral Force	
$V_{E/W}$ =	**2,794.4**	kips	Pseudo-Lateral Force	

Vertical Distribution (LSP)-TIER 2

Vertical Distribution of Seismic Forces (North-South) *Y Direction* *ASCE 41-13 §7.4.1.3.2*

$V_{N/S}$ = 2,794 *kips* Total Base Shear
k = 1.0 For T = 0.36 sec

	w_x (kips)	h_x (ft)	$w_x h^k_x$	C_{vx}	F_x (kips)	V_x (kips)	F_{px} (kips)
Roof	1,251	28.00	35,017	0.49	1,364	1,364	1,364
2nd Level	1,289	19.00	24,491	0.34	954	2,318	1,176
1st Level	1,289	9.50	12,245	0.17	477	2,794	941
Total	*3,829*		*71,753*	*1.00*	*2,794*		

Vertical Distribution of Seismic Forces (East-West) *X Direction* *ASCE 41-13 §7.4.1.3.2*

$V_{E/W}$ = 2,794 *kips* Total Base Shear
k = 1.0 For T = 0.36 sec

	w_x (kips)	h_x (ft)	$w_x h^k_x$	C_{vx}	F_x (kips)	V_x (kips)	F_{px} (kips)
Roof	1,251	28.00	35,017	0.49	1,364	1,364	1,364
2nd Level	1,289	19.00	24,491	0.34	954	2,318	1,176
1st Level	1,289	9.50	12,245	0.17	477	2,794	941
Total	*3,829*		*71,753*	*1.00*	*2,794*		

Vertical Distribution (LSP)-TIER 2 MODEL

Vertical Distribution of Seismic Forces (North-South) *Y Direction* *ASCE 41-13 §7.4.1.3.2*

$V_{N/S}$ = 1,854 *kips* Total Base Shear
k = 1.0 For T = 0.36 sec

	w_x (kips)	h_x (ft)	$w_x h^k_x$	C_{vx}	F_x (kips)	V_x (kips)
Roof	1,251	18.67	23,349	0.65	1,213	1,213
2nd Level	1,289	9.58	12,353	0.35	642	1,854
Total	*2,540*		*35,701*	*1.00*	*1,854*	

Vertical Distribution of Seismic Forces (East-West) *X Direction* *ASCE 41-13 §7.4.1.3.2*

$V_{E/W}$ = 1,854 *kips* Total Base Shear
k = 1.0 For T = 0.36 sec

	w_x (kips)	h_x (ft)	$w_x h^k_x$	C_{vx}	F_x (kips)	V_x (kips)
Roof	1,251	18.67	23,349	0.65	1,213	1,213
2nd Level	1,289	9.58	12,353	0.35	642	1,854
Total	*2,540*		*35,701*	*1.00*	*1,854*	

Column Shear Stress Check
Tier 2: Section 5.5.2.1.4

k=	0.9		f'ce=	6000 psi		3rd Floor				
	per table 6-1		f'ce=	7500 psi		1st & 2nd Floor				
	b (in)	d (in)	area (in2)	stl area (in2)	ρl	Av	s (in)	fyl (ksi)	fyv (ksi)	ρv
2nd	24	21.1875	508.5	6.24	0.01227	0.4	12	62.5	50	0.0014
1st	24	21.1875	508.5	6.24	0.01227	0.4	11.5	62.5	50	0.0014
Ground	-	28.125	621.262	9.36	0.01507	0.22	2.25	62.5	50	0.0004

	Vo Vc+Vs	P demand	Mp Spcol	L	V_p = 2M/L	Vp/Vo	Cond.	Vn kips	$P/A_gF'c$	V/bd√fc
2nd	111.6	85.89	450	7.5	120.0	1.08	iii	111.6	0.03	3.52
1st	122.3	174.42	550	8	137.5	1.12	iii	122.3	0.05	3.70
Ground	237.8	262.95	915	8	228.8	0.96	ii	228.8	0.06	3.25

	Max V Eqx (kips)	Max V Eqy (kips)	mkVn	DCR
2nd	47.07	128.79	100.43	1.28
1st	67.78	151.42	110.08	1.38
Ground	127.18	210.82	214.00	0.99

Tier 2 check is non-compliant.

Flat Slab Frames
See Tier 2- Beam Flexure spreadsheet for flexure compliance.
Punching shear is not a failure mechanism for the flat slab frames due to continuous beams over each column.

No Shear Failures
Sec 5.5.2.3.4

1st and 2nd floor columns are noncompliant, the adequacy of the columns for shear was checked in the previous section. The beams are all controlled by flexural failure and therefore do not need to be check

Strong column - Weak Beam
Tier 2: Section 5.5.2.1.5

Evaluate columns for shear and flexure demands with m=2
Column shear demands per shear check above.

	Max V Eqx (kips)	Max V Eqy (kips)	m	Vn	mkVn	DCR
2nd	47.07	128.79	2	107.8	194.1	0.7
1st	67.78	151.42	2	117.8	212.0	0.7
Ground	127.18	210.82	2	228.8	411.8	0.5

All columns pass the shear check with m=2

	Max M Eqx (k-ft)	Max M Eqy (k-ft)	m	Mn	mkMn	DCR
2nd	180.28	522.02	2	450.0	810.0	0.6
1st	461.08	876.1	2	550.0	990.0	0.9
Ground	508.72	843.28	2	915.0	1647.0	0.5

All columns pass the flexure check with m=2

The slab-column direction is compliant per Tier 1 checks. The beam-column direction is compliant per Tier 2 Checks

Column Tie Spacing
Tier 2: Section 5.5.2.3.7

	Vp/Vo	Condition	$P/A_gF'c$	V/bd√fc	m
2nd	1.08	iii	0.03	3.52	1
1st	1.12	iii	0.05	3.70	1
Ground	0.96	ii	0.06	3.25	1

Column force demands were checked in previous sections and were found non-compliant. The m-factor for column tie spacing matches that of the previous section. Therefore, the columns are non-compliant for tie spacing as well.

Stirrup Spacing
Tier 2: Section 5.5.2.3.7

Beam: Typical End of Beam

Properties			**Shear Reinf.**		
f'_{ce} =	6000	psi		2	Legs
f_{ye} =	75	ksi	#3	@ 4	"oc
f_{yte} =	75	ksi	k=	0.75	
			m=	2.00	
Beam Dimensions			V_c =	60.0	k
Width =	24.0	in	V_s =	66.5	k
Depth =	18.5	in			
Cover =	1.5	in			
d =	16.1	in			

Pre-Stressed Concrete Beam Checks

Lower Limit:	mkV_n =	189.7 k	DCR= 0.27	OK
Upper Limit:	mkV_n =	324.6 k		

V_p= 2M/L Shear based on flexural capacity
V_p= 51.0 k

Beam: Typical Mid span of Beam

Properties			**Shear Reinf.**		
f'_{ce} =	6000	psi		2	Legs
f_{ye} =	75	ksi	#3	@ 18	"oc
f_{yte} =	75	ksi	k=	0.75	
			m=	2.00	
Beam Dimensions			V_c =	60.0	k
Width =	24.0	in	V_s =	14.8	k
Depth =	18.5	in			
Cover =	1.5	in			
d =	16.1	in			

Pre-Stressed Concrete Beam Checks

Lower Limit:	$\phi_v V_n$ =	112.1 k	DCR= 0.39	OK
Upper Limit:	$\phi_v V_n$ =	247.0 k		

V_p= 2M/L Shear based on flexural capacity
V_p= 44.0 k

See Beam Flexural Capacity Spreadsheet for moment check.

Check is compliant for flexure and shear.

Joint Transverse Reinforcing
Section 5.5.2.3.8

Typical Joint:

f'_{ce}= 6000 psi

k= 0.75

	A_j (in^2)	γ	V_n (Kips)	m	mkVn	V	DCR	Compliance
Roof	576.0	10.0	446.2	1.0	334.6	128.8	0.29	OK
2nd	576.0	10.0	446.2	1.0	334.6	151.4	0.34	OK
1st	1017.9	10.0	788.4	1.0	591.3	210.8	0.27	OK

Check is compliant.

Torsional Irregularity

Bldg Width: x-dir 61 ft
y-dir 183.75 ft

	XCM (ft)	YCM (ft)	XCR (ft)	YCR (ft)	% DIFF X-DIR	% DIFF Y-DIR	Compliance
Roof	91.9	30.2	91.9	60.2	0.00	0.49	Non-Compliant
2nd	91.9	29.8	93.0	60.1	-0.01	0.50	Non-Compliant

The overall structure is non-compliant for tosional irregularity in the 'Y-direction'.
Half of the first floor is founded but assumed torsionally irregular.

Beam Capacities

Typical Beam

$V_p/V_{0=}$ 0.65

Flexure Controlled

Beam: Typ Beam, M+

$\rho-\rho'/\rho_{bal}$= 0 (Same reinforcement top and bot)

$V/b_w d\sqrt{f'_c}$= 0.0002

m= 3

Properties			**Flexural Reinf.**			**Effective Prestress**		
f'_{ce} =	6000	psi	4	#8	bars	Beam		
f_{ye} =	62.5	ksi	A_s =	3.16	in^2	F_e =	325	kips
f_{yte} =	62.5	ksi	a =	0.75	in	dp=	6.5	in
			β_1 =	0.75				
Beam Dimensions			c =	1.00	in	slab contribution is negligible		
Width =	136.0	in	ε_s =	0.045				
Depth =	18.5	in						
Cover =	1.5	in	M_n =	257.1	k-ft			
d =	16.0	in	$M_{n,ps}$ =	165.8	k-ft			
			$M_{n,\ total}$ =	423.0	k-ft			
			mkMn=	951.7	k-ft			
			Mu=	576.9	k-ft			
			DCR=	0.6	OK			

Beam: Typ Beam, M-

$\rho-\rho'/\rho_{bal}$= 0 (Same reinforcement top and bot)

$V/b_w d\sqrt{f'_c}$= 0.0014

m= 3

Properties			**Flexural Reinf.**			**Effective Prestress**		
f'_{ce} =	6000	psi	4	#8	bars	Beam		
f_{ye} =	62.5	ksi	4	#4	bars	F_e =	325	kips
f_{yte} =	62.5	ksi	A_s =	3.96	in^2	dp=	12.25	in
			a =	6.29	in			
Beam Dimensions			β_1 =	0.75		Slab		
Width =	24.0	in	c =	8.38	in	F_e =	197	kips
Depth =	18.5	in	ε_s =	0.003		dp=	12.75	in
Cover =	1.5	in						
d =	16.0	in	M_n =	265.2	k-ft			
			$M_{n,ps}$ =	404.3	k-ft			
			$M_{n,\ total}$ =	669.5	k-ft			
			mkMn=	1506.4	k-ft			
			Mu=	572.3	k-ft			
			DCR=	0.4	OK			

Slab Capacities
ASCE 41-13, section 10.4.4.3

1 be=	3.75 ft		column =	2 ft
2 be=	4.92 ft		slab thickness =	0.58 ft
b1=	2.58 ft		γf =	0.6
b2=	2.58 ft		γv =	0.4

k= 0.9
m= 2 (Punching shear not a failure mechanism. V_g/V_0 goes to 0)

Beam: Slab, M+, ASCE 41 10.4.4.3 #1

Properties		**Flexural Reinf.**		**Effective Prestress**	
f'_{ce} =	6000 psi	2.5	#5 bars		
f_{ye} =	62.5 ksi	A_s =	0.78 in^2	F_e =	86.25 kips
f_{yte} =	62.5 ksi	a =	0.59 in	dp=	5.75 in
		β_1 =	0.75		
Beam Dimensions		c =	0.78 in		
Width =	45.0 in	ε_s =	0.015		
Depth =	7.0 in	M_n =	17.7 k-ft		
Cover =	1.5 in	$M_{n,ps}$ =	39.2 k-ft		
d =	4.7 in	$M_{n, total}$ =	102.5 k-ft		

Beam: Slab, M-, ASCE 41 10.4.4.3 #1

Properties		**Flexural Reinf.**		**Effective Prestress**	
f'_{ce} =	6000 psi	2.5	#5 bars		
f_{ye} =	62.5 ksi	A_s =	0.78 in^2	F_e =	86.25 kips
f_{yte} =	62.5 ksi	a =	0.59 in	dp=	1.25 in
		β_1 =	0.75		
Beam Dimensions		c =	0.78 in		
Width =	45.0 in	ε_s =	0.002		
Depth =	7.0 in	M_n =	3.6 k-ft		
Cover =	5.0 in	$M_{n,ps}$ =	6.9 k-ft		
d =	1.2 in	$M_{n, total}$ =	18.9 k-ft		

$\sum Mn$ = 121.4 k-ft
$\sum Mn/\gamma v$ = 303.5 k-ft

Beam: Slab, M+, ASCE 41 10.4.4.3 #2

Properties			**Flexural Reinf.**			**Effective Prestress**		
f'_{ce} =	6000	psi	3.28	#5	bars			
f_{ye} =	62.5	ksi	A_s =	1.02	in^2	F_e =	113.1	kips
f_{yte} =	62.5	ksi	a =	0.59	in	dp=	5.75	in
			β_1 =	0.75				
Beam Dimensions			c =	0.78	in			
Width =	59.0	in	ε_s =	0.015				
Depth =	7.0	in	M_n =	23.3	k-ft			
Cover =	1.5	in	$M_{n,ps}$ =	51.4	k-ft			
d =	4.7	in	$M_{n,\ total}$ =	134.4	k-ft			

Beam: Slab, M-, ASCE 41 10.4.4.3 #2

Properties			**Flexural Reinf.**			**Effective Prestress**		
f'_{ce} =	6000	psi	3.28	#5	bars			
f_{ye} =	62.5	ksi	A_s =	1.02	in^2	F_e =	113.1	kips
f_{yte} =	62.5	ksi	a =	0.59	in	dp=	1.25	in
			β_1 =	0.75				
Beam Dimensions			c =	0.78	in			
Width =	59.0	in	ε_s =	0.002				
Depth =	7.0	in	M_n =	4.7	k-ft			
Cover =	5.0	in	$M_{n,ps}$ =	9.0	k-ft			
d =	1.2	in	$M_{n,\ total}$ =	24.7	k-ft			

∑Mn = 159.2 k-ft
∑Mn/γf = 265.3 k-ft

Slab Demand

Story	Mu (k-ft)	DCR	Check
Roof	22.37	0.084	**OK**
2nd	23.87	0.090	**OK**
1st	23.87	0.090	**OK**

LINEAR STATIC PROCEDURE (LSP)-RETROFIT

ASCE 41-13 §7.4.1

# Stories =	3		Number of stories in building	

Period Determination for LSP - Method 2 - Empirical

ASCE 41-13 §7.4.1.2

North-South Direction

Building Type =	C1			
C_t =	0.018		Factor for adjustment of period	*ASCE 41-13 §7.4.1.2.2*
β =	0.90		Factor for adjustment of period	
h_n =	28.0	ft	Roof Height	
T =	**0.361**	sec	Building period in N-S direction	*ASCE 41-13 Eq. 7-18*

East-West Direction

Building Type =	C2			
C_t =	0.020		Factor for adjustment of period	
β =	0.75		Factor for adjustment of period	
h_n =	28.0	ft	Roof Height	
T =	**0.243**	sec	Building period in E-W direction	*ASCE 41-13 Eq. 7-18*

Pseudo-Seismic Force for LSP (North-South)

ASCE 41-13 §7.4.1.3.1

$V = C_1 C_2 C_m S_a W$			Pseudo-Lateral Force	*ASCE 41-13 Eq. 7-21*
C_1 =	1.0		Modification Factor, Inelastic Displacements	*ASCE 41-13 Eq. 7-22*
C_2 =	1.0		Modification Factor, Cyclic Behavior	*ASCE 41-13 Eq. 7-23*
$C_1 C_2$ =	1.1		Alternative Value for Modification Factors	*ASCE 41-13 Table 7-3*
Use alternate C_1C_2?	Yes			
C_m =	0.9		Effective Mass Factor	*ASCE 41-13 Table 7-4*
$S_a(T)$ =	**0.737**	g	Spectral Response Acceleration	*for T = 0.36sec*
W =	3828.6	kips	Effective Seismic Weight	
$V_{N/S}$ =	**0.730**	*W	Pseudo-Lateral Force	
$V_{N/S}$ =	**2,794.4**	kips	Pseudo-Lateral Force	

Pseudo-Seismic Force for LSP (East-West)

ASCE 41-13 §7.4.1.3.1

$V = C_1 C_2 C_m S_a W$			Pseudo-Lateral Force	*ASCE 41-13 Eq. 7-21*
C_1 =	1.0		Modification Factor, Inelastic Displacements	*ASCE 41-13 Eq. 7-22*
C_2 =	1.0		Modification Factor, Cyclic Behavior	*ASCE 41-13 Eq. 7-23*
$C_1 C_2$ =	1.1		Alternative Value for Modification Factors	*ASCE 41-13 Table 7-3*
Use alternate C_1C_2?	Yes			
C_m =	0.9		Effective Mass Factor	*ASCE 41-13 Table 7-4*
$S_a(T)$ =	**0.737**	g	Spectral Response Acceleration	*for T = 0.24sec*
W =	3828.6	kips	Effective Seismic Weight	
$V_{E/W}$ =	**0.730**	*W	Pseudo-Lateral Force	
$V_{E/W}$ =	**2,794.4**	kips	Pseudo-Lateral Force	

Deficiency Based Retrofit

The deficiences that remain after the Tier 2 evaluation will now be mitigated with a retrofit. The scope of the Tier 2 deficiency-based retrofit need not expand beyond that necessary to modify the building to comply with a Tier 1 screening or a Tier 2 Evaluation.

Design Values

Column f'ce=	6000 psi	fye=	75 ksi
Column f'ce=	7500 psi - 1st flr & 2nd flr	k=	0.9 (Table 6-1)
Shear wall f'ce=	5200 psi		

New Concrete Shear walls

twall = 10 inch   Lwall = 25 ft   k= 1

max axial (kips) from ETABS

Direction	X	X	
	Pier A	Pier D	MIN
2nd	118	118	118
1st	307	307	307
Gound	496	-	496

max shear (kips) from ETABS

Direction	X	X	
	Pier A	Pier D	MAX
2nd	644	644	644
1st	977	977	977
Gound	1216	-	1216

max moment (kip-ft) from ETABS

Direction	X	X	
	Pier A	Pier D	MAX
2nd	5796	5796	5796
1st	15108	15108	15108
Gound	26660	-	26660

New Shear Wall Design

	Acv (in2)	α	ρt	Vn	m	mkVn	DCR
2nd	3000	2	0.00344	1207.7	2.5	3019.2	0.21
1st	3000	2	0.00344	1207.7	2.5	3019.2	0.32
Gound	3000	2	0.00344	1207.7	2.5	3019.2	0.40

	Mn (kip-ft)	ρl	mkMn	DCR	Typ. Reinf	steel lbs/yd3
2nd	12000	0.00344	30000	0.19	(2) #5@18"oc	91
1st	14000	0.00344	35000	0.43	(2) #5@18"oc	91
Gound	16100	0.00344	40250	0.66	(2) #5@18"oc	91

Shear Transfer (Shear Friction) New Wall

	v (kip/in)	V (kips)	Reinf	Av	µ	Vn	mkVn	DCR
Roof	2.15	38.6	(2) #5@18"oc	0.62	1	46.5	104.6	0.37
2nd	3.26	58.6	(2) #5@18"oc	0.62	1	46.5	104.6	0.56
1st	4.05	73.0	(2) #5@18"oc	0.62	1	46.5	104.6	0.70

Existing Concrete Shear Wall

t= 10 in L_{typ}= 30 ft fye= 75 ksi

	Acv (in2)	α	ρt	Vn	$V/t_w l_w \sqrt{f'_c}$	m	mkVn	DCR
2nd	3600	2	0.00258	1216.7	0.00248	2.5	3041.7	0.21
1st	6600	2	0.00258	2230.6	0.00376	2.5	5576.5	0.18

Shear Transfer (Shear Friction) Existing Wall

	v (kip/in)	V (kips)	Reinf	Av	µ	Vn	mkVn	DCR
Roof	2.15	25.7	#5@12"oc	0.31	1	23.3	52.3	0.49
2nd	3.26	39.1	#5@12"oc	0.31	1	23.3	52.3	0.75

Diaphragm Capacity Check at Shearwall

	Vc (kip/ft)	Vs (kip/ft)	Vn (kip/ft)	m	L of Failure Plane (ft)	mkVn (kip/ft)	V (kip)	DCR
Roof	13.01	3.89	16.9	2.5	30.5	1159.9	584.5	0.5
2nd	13.01	3.89	16.9	2.5	30.5	1159.9	504.1	0.4
1st	13.01	3.89	16.9	2.5	30.5	1159.9	403.2	0.3

Column Shear Stress Check
Tier 2: Section 5.5.2.1.4

	b (in)	d (in)	area (in2)	stl area (in2)	pl	Av	s (in)	fyl (ksi)	fyv (ksi)	ρv
2nd	24	21.1875	508.5	6.24	0.012271386	0.4	12	62.5	50	0.0014
1st	24	21.1875	508.5	6.24	0.012271386	0.4	11.5	62.5	50	0.0014
Gound	-	28.125	621.2622191	9.36	0.015066102	0.22	2.25	62.5	50	0.0004

	Vo Vc+Vs	P demand	Mp Spcol	L	V_p = 2M/L	Vp/Vo		Vn kips	P/AF'c	V/bdfc
2nd	128.4	85.89	450	7.5	120.0	0.93	ii	120.0	0.03	2.74
1st	157.9	174.42	550	8	137.5	0.87	ii	137.5	0.05	3.54
Gound	237.8	262.95	915	8	228.8	0.96	ii	228.8	0.06	3.15

	Max V Eqx (kips)	Max V Eqy (kips)	m	mkVn	DCR
2nd	1.59	100.19	1	128.43	0.78
1st	4.2	144.702	1	157.92	0.92
Gound	6.81	204.102	1	214.00	0.95

Tier 2 check after retrofit results in all columns compliant with new loads.

No Shear Failures
Sec 5.5.2.3.4

Tier 2 check after FRP retrofit results in all columns compliant as shown in the previous section.

Torsional Irregularity

Bldg Width: x-dir 61 ft
y-dir 183.75 ft

	XCM (ft)	YCM (ft)	XCR (ft)	YCR (ft)	% DIFF X-DIR	% DIFF Y-DIR	Compliance
Roof	91.875	30.5	91.875	30.5	0.00	0.00	OK
2nd	91.875	30.5	92.02	30.33	0.00	0.00	OK

With the addition of a concrete shear wall, the building is compliant for torsional irregularity.

Helical Pile Foundation

Overturning Demand
Compression= 606 k
Uplift= 318 k

Helical Pile Capacity Chance Helical Anchors Table-1A with an increase per ASCE 41 8.4.1.1

Comp P_{all} (Kips)	Ten. P_{all} (Kips)	m	Comp mkPall (Kips)	Ten. mkPall (Kips)
72	80	3	216	240
		Anchors Req'd:	3.0	2.0

Lateral Demand Per Anchor (4 anchors EA side)= 152.0 kips
*Lateral Capacity to be verified from analysis once soils report obtained.

Min spacing= 42 inches (3ø)

TABLE: Material Properties - Concrete							
Name	E	v	α	G	Unit Weight	Unit Mass	Fc
	lb/in²		1/F	lb/in²	lb/ft³	lb-s²/ft⁴	lb/in²
3000Psi	3122018.6	0.2	0.0000055	1300841.1	150	4.662	3000
4000Psi	3604996.5	0.2	0.0000055	1502081.9	150	4.662	4000
5000Psi	4030508.7	0.2	0.0000055	1679378.6	150	4.662	5000

TABLE: Material Properties - Rebar						
Name	E	α	Unit Weight	Unit Mass	Fy	Fu
	lb/in²	1/F	lb/ft³	lb-s²/ft⁴	lb/in²	lb/in²
A615Gr40	29000000	0.0000065	490	15.23	40000	60000
A615Gr60	29000000	0.0000065	490	15.23	60000	90000

TABLE: Material Properties - Tendon						
Name	E	α	Unit Weight	Unit Mass	Fy	Fu
	lb/in²	1/F	lb/ft³	lb-s²/ft⁴	lb/in²	lb/in²
A416Gr270	28500000	0.0000065	490	15.23	245100	270000

TABLE: Frame Sections									
Name	Material	Shape	t3	t2	Area	AS2	AS3	I22 Modifier	I33 Modifier
			in	in	in²	in²	in²		
ConcBm	4000Psi	Concrete Rectangular	18.5	24	444	370	370	1	1
ConcCol 4000psi	4000Psi	Concrete Rectangular	24	24	576	480	480	0.3	0.3
ConcCol 5000psi	5000Psi	Concrete Rectangular	24	24	576	480	480	0.3	0.3

TABLE: Shell Sections - Slab									
Name	Material	Slab Type	Element Type	Slab Thickness	m11 Modifier	m22 Modifier	m12 Modifier	v13 Modifier	v23 Modifier
				in					
7" Prestressed Sla	4000Psi	Uniform	Shell-Thin	7	0.5	0.5	0.5	0.4	0.4

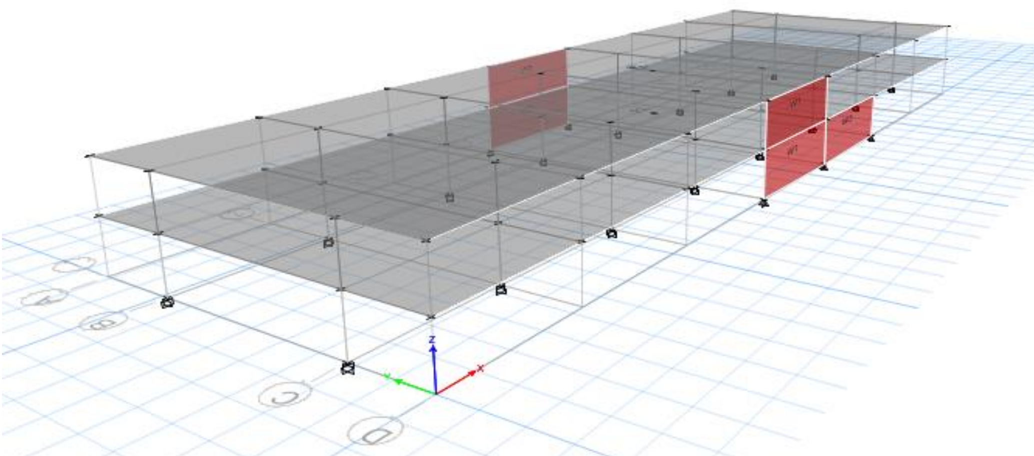
TABLE: Shell Sections - Wall								
Name	Material	Element Type	Thickness	m11 Modifier	m22 Modifier	m12 Modifier	v13 Modifier	v23 Modifier
			in					
Conc Shearwall	3000Psi	Shell-Thin	10	0.5	0.5	0.5	0.4	0.4

TABLE: Modal Periods and Frequencies			
Case	Mode	Period	Frequency
		sec	cyc/sec
Modal	1	0.429	2.329
Modal	2	0.28	3.574
Modal	3	0.278	3.593
Modal	4	0.244	4.097
Modal	5	0.238	4.203
Modal	6	0.237	4.223
Modal	7	0.236	4.234
Modal	8	0.236	4.237
Modal	9	0.232	4.315
Modal	10	0.23	4.351

TABLE: Base Reactions				
Load Case	FX	FY	FZ	
	kip	kip	kip	
Dead	0	0	2716.476	
Live	0	0	896.7	
EQx	-2029.569	0	0	<- Compare with calculated model base shear 1,834, OK
EQy	0	-2029.569	0	<- Compare with calculated model base shear 1,834, OK
SuperDead	0	0	156.923	

TABLE: Load Combinations				
Name	Load Case	Scale Factor	Type	Auto
EQx+1.1(D+L)	Dead	1.1	Linear Add	No
EQx+1.1(D+L)	EQx	1		
EQx+1.1(D+L)	Live	1.1		
EQy+1.1(D+L)	Dead	1.1	Linear Add	No
EQy+1.1(D+L)	Live	1.1		
.9D+EQx	Dead	0.9	Linear Add	No
.9D+EQx	EQx	-1		
.9D+EQy	Dead	0.9	Linear Add	No
EQx Env	EQx	1	Envelope	No
EQy Env	EQy	1	Envelope	No
EQx+1.1(D+L) Env	EQx	1	Envelope	No
EQx+1.1(D+L) Env	Dead	1.1		
EQx+1.1(D+L) Env	Live	1.1		
EQy+1.1(D+L) Env	EQy	1	Envelope	No
EQy+1.1(D+L) Env	Dead	1.1		
EQy+1.1(D+L) Env	Live	1.1		
.9D+EQx Env	Dead	0.9	Envelope	No
.9D+EQx Env	EQx	-1		
.9D+EQy Env	Dead	0.9	Envelope	No
.9D+EQy Env	EQy	-1		

3D ETABS Model



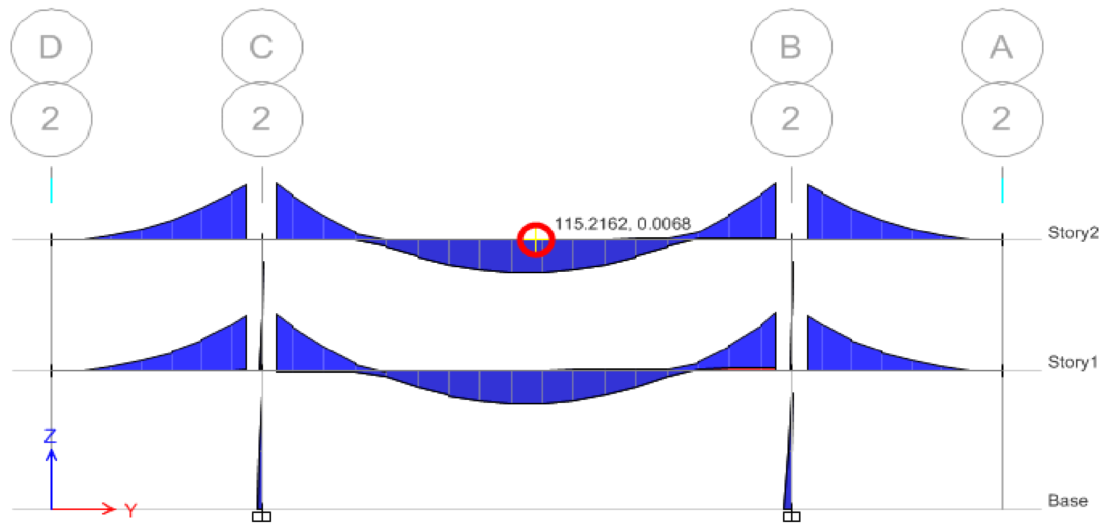
Example Column output looking at Eqx at the 1st story columns.

TABLE: Column Forces

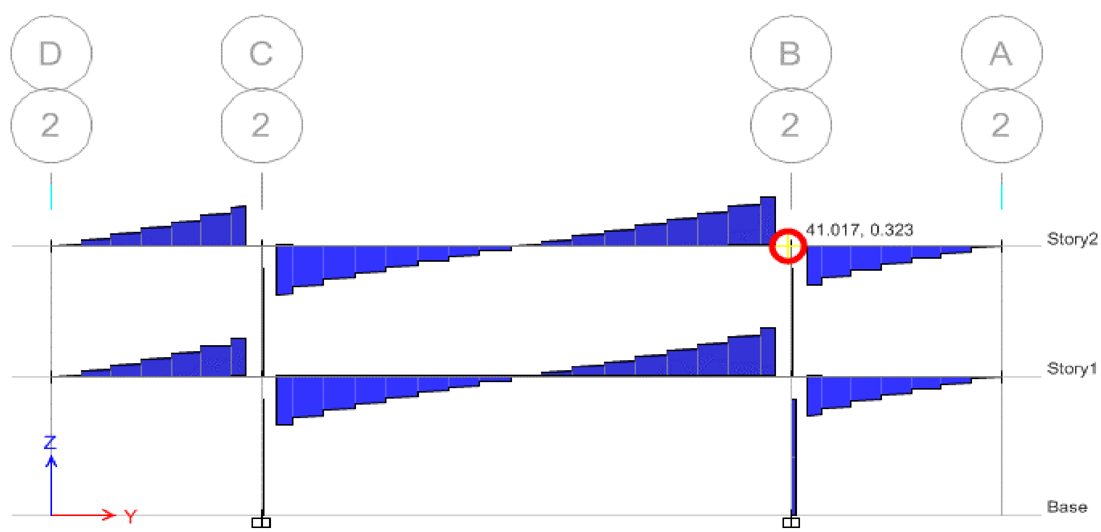
Story	Column	Load Case/Combo	Station ft	P kip	V2 kip	V3 kip	T kip-ft	M2 kip-ft	M3 kip-ft
Story1	C9	EQx+1.1(D+L) Env Max	0	1.871	1.591	4.451	0.0165	26.7091	13.5106
Story1	C9	EQx+1.1(D+L) Env Max	4.0208	1.871	1.591	4.451	0.0165	8.8104	7.1129
Story1	C9	EQx+1.1(D+L) Env Max	8.0417	1.871	1.591	4.451	0.0165	0.4469	36.3453
Story1	C9	EQx+1.1(D+L) Env Min	0	-114.812	-7.277	0.143	-1.5835	1.0939	-22.1755
Story1	C9	EQx+1.1(D+L) Env Min	4.0208	-112.159	-7.277	0.143	-1.5835	0.5175	3.2318
Story1	C9	EQx+1.1(D+L) Env Min	8.0417	-109.505	-7.277	0.143	-1.5835	-9.0883	0.7153
Story1	C9	.9D+EQx Env Max	0	-1.871	-1.591	0.207	1.5835	2.0336	-13.5106
Story1	C9	.9D+EQx Env Max	4.0208	-1.871	-1.591	0.207	1.5835	1.1996	5.7967
Story1	C9	.9D+EQx Env Max	8.0417	-1.871	-1.591	0.207	1.5835	9.0883	29.7371
Story1	C9	.9D+EQx Env Min	0	-93.937	-5.954	-4.451	0.0135	-26.7091	-18.1436
Story1	C9	.9D+EQx Env Min	4.0208	-91.766	-5.954	-4.451	0.0135	-8.8104	-7.1129
Story1	C9	.9D+EQx Env Min	8.0417	-89.595	-5.954	-4.451	0.0135	0.3656	-0.7153
Story1	C10	EQx+1.1(D+L) Env Max	0	0.77	1.817	3.236	0.0165	19.3121	14.1994
Story1	C10	EQx+1.1(D+L) Env Max	4.0208	0.77	1.817	3.236	0.0165	6.302	6.892
Story1	C10	EQx+1.1(D+L) Env Max	8.0417	0.77	1.817	3.236	0.0165	3.1066	-0.4155
Story1	C10	EQx+1.1(D+L) Env Min	0	-198.864	0.117	-0.271	-1.5835	0.3506	0.3538
Story1	C10	EQx+1.1(D+L) Env Min	4.0208	-196.21	0.117	-0.271	-1.5835	0.7799	-0.271
Story1	C10	EQx+1.1(D+L) Env Min	8.0417	-193.556	0.117	-0.271	-1.5835	-6.7081	-1.2946
Story1	C10	.9D+EQx Env Max	0	-0.77	0.208	-0.222	1.5835	0.7588	0.6158
Story1	C10	.9D+EQx Env Max	4.0208	-0.77	0.208	-0.222	1.5835	1.6503	-0.2217
Story1	C10	.9D+EQx Env Max	8.0417	-0.77	0.208	-0.222	1.5835	6.7081	0.4155
Story1	C10	.9D+EQx Env Min	0	-162.707	-1.817	-3.236	0.0135	-19.3121	-14.1994
Story1	C10	.9D+EQx Env Min	4.0208	-160.535	-1.817	-3.236	0.0135	-6.302	-6.892
Story1	C10	.9D+EQx Env Min	8.0417	-158.364	-1.817	-3.236	0.0135	2.5418	-1.0593
Story1	C21	EQx+1.1(D+L) Env Max	0	-1.005	3.401	3.237	0.0165	19.3175	23.8082
Story1	C21	EQx+1.1(D+L) Env Max	4.0208	-1.005	3.401	3.237	0.0165	6.3003	10.132
Story1	C21	EQx+1.1(D+L) Env Max	8.0417	-1.005	3.401	3.237	0.0165	-1.916	-0.5762
Story1	C21	EQx+1.1(D+L) Env Min	0	-199.304	0.11	0.519	-1.5835	2.2543	0.3095
Story1	C21	EQx+1.1(D+L) Env Min	4.0208	-196.65	0.11	0.519	-1.5835	0.1692	-0.3055
Story1	C21	EQx+1.1(D+L) Env Min	8.0417	-193.997	0.11	0.519	-1.5835	-6.7169	-3.5442
Story1	C21	.9D+EQx Env Max	0	1.005	0.195	1.09	1.5835	4.7511	0.5354
Story1	C21	.9D+EQx Env Max	4.0208	1.005	0.195	1.09	1.5835	0.3695	-0.25
Story1	C21	.9D+EQx Env Max	8.0417	1.005	0.195	1.09	1.5835	6.7169	3.5442
Story1	C21	.9D+EQx Env Min	0	-163.067	-3.401	-3.237	0.0135	-19.3175	-23.8082
Story1	C21	.9D+EQx Env Min	4.0208	-160.896	-3.401	-3.237	0.0135	-6.3003	-10.132
Story1	C21	.9D+EQx Env Min	8.0417	-158.725	-3.401	-3.237	0.0135	-4.0122	-1.0353
Story1	C22	EQx+1.1(D+L) Env Max	0	-0.4	3.393	2.011	0.0165	11.8881	23.7838
Story1	C22	EQx+1.1(D+L) Env Max	4.0208	-0.4	3.393	2.011	0.0165	3.8023	10.1398
Story1	C22	EQx+1.1(D+L) Env Max	8.0417	-0.4	3.393	2.011	0.0165	-2.8481	-0.2552

Story1	C22	EQx+1.1(D+L) Env Min	0	-195.255	0.046	0.709	-1.5835	2.8513	0.1139
Story1	C22	EQx+1.1(D+L) Env Min	4.0208	-192.601	0.046	0.709	-1.5835	-0.0253	-0.1971
Story1	C22	EQx+1.1(D+L) Env Min	8.0417	-189.948	0.046	0.709	-1.5835	-7.5027	-3.5042
Story1	C22	.9D+EQx Env Max	0	0.4	0.104	1.522	1.5835	6.0972	0.2588
Story1	C22	.9D+EQx Env Max	4.0208	0.4	0.104	1.522	1.5835	-0.0207	-0.1612
Story1	C22	.9D+EQx Env Max	8.0417	0.4	0.104	1.522	1.5835	4.2834	3.5042
Story1	C22	.9D+EQx Env Min	0	-159.754	-3.393	-2.011	0.0135	-11.8881	-23.7838
Story1	C22	.9D+EQx Env Min	4.0208	-157.583	-3.393	-2.011	0.0135	-3.8023	-10.1398
Story1	C22	.9D+EQx Env Min	8.0417	-155.412	-3.393	-2.011	0.0135	-6.1385	-0.5813
Story1	C22	.9D+EQx Env Min	0	42.306	0.104	1.522	0.1135	6.0972	0.2588
Story1	C23	EQx+1.1(D+L) Env Max	0	3.843	3.407	9.452	0.0165	30.6038	23.8248
Story1	C23	EQx+1.1(D+L) Env Max	4.0208	3.843	3.407	9.452	0.0165	0.6559	10.1266
Story1	C23	EQx+1.1(D+L) Env Max	8.0417	3.843	3.407	9.452	0.0165	-5.1681	-1.7411
Story1	C23	EQx+1.1(D+L) Env Min	0	-169.877	0.343	1.448	-1.5835	6.4798	1.0191
Story1	C23	EQx+1.1(D+L) Env Min	4.0208	-167.224	0.343	1.448	-1.5835	-7.402	-0.7961
Story1	C23	EQx+1.1(D+L) Env Min	8.0417	-164.57	0.343	1.448	-1.5835	-45.4078	-3.7755
Story1	C23	.9D+EQx Env Max	0	-3.843	0.606	7.734	1.5835	25.0395	1.7864
Story1	C23	.9D+EQx Env Max	4.0208	-3.843	0.606	7.734	1.5835	-0.6559	-0.6513
Story1	C23	.9D+EQx Env Max	8.0417	-3.843	0.606	7.734	1.5835	5.1681	3.5715
Story1	C23	.9D+EQx Env Min	0	-138.991	-3.407	-1.448	0.0135	-6.4798	-23.8248
Story1	C23	.9D+EQx Env Min	4.0208	-136.819	-3.407	-1.448	0.0135	-6.0562	-10.1266
Story1	C23	.9D+EQx Env Min	8.0417	-134.648	-3.407	-1.448	0.0135	-37.1518	-3.0891
Story1	C24	EQx+1.1(D+L) Env Max	0	-3.753	3.404	9.59	0.0165	31.0606	23.817
Story1	C24	EQx+1.1(D+L) Env Max	4.0208	-3.753	3.404	9.59	0.0165	-0.3807	10.1291
Story1	C24	EQx+1.1(D+L) Env Max	8.0417	-3.753	3.404	9.59	0.0165	4.7434	3.9649
Story1	C24	EQx+1.1(D+L) Env Min	0	-170.101	-0.808	-1.274	-1.5835	-5.5048	-2.5316
Story1	C24	EQx+1.1(D+L) Env Min	4.0208	-167.447	-0.808	-1.274	-1.5835	-7.4976	0.3318
Story1	C24	EQx+1.1(D+L) Env Min	8.0417	-164.794	-0.808	-1.274	-1.5835	-46.0558	-3.5587
Story1	C24	.9D+EQx Env Max	0	3.753	-0.661	7.846	1.5835	25.4133	-2.0713
Story1	C24	.9D+EQx Env Max	4.0208	3.753	-0.661	7.846	1.5835	0.3807	0.5863
Story1	C24	.9D+EQx Env Max	8.0417	3.753	-0.661	7.846	1.5835	-4.7434	3.5587
Story1	C24	.9D+EQx Env Min	0	-139.174	-3.404	1.274	0.0135	5.5048	-23.817
Story1	C24	.9D+EQx Env Min	4.0208	-137.002	-3.404	1.274	0.0135	-6.1344	-10.1291
Story1	C24	.9D+EQx Env Min	8.0417	-134.831	-3.404	1.274	0.0135	-37.682	3.244

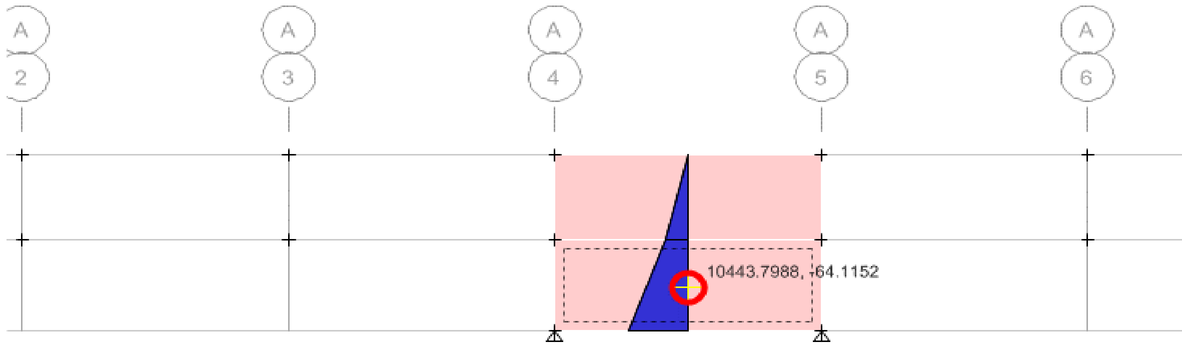
Example graphic of Beam Moment Demands under Max Eqy combo at Gridline 2. (k-ft)



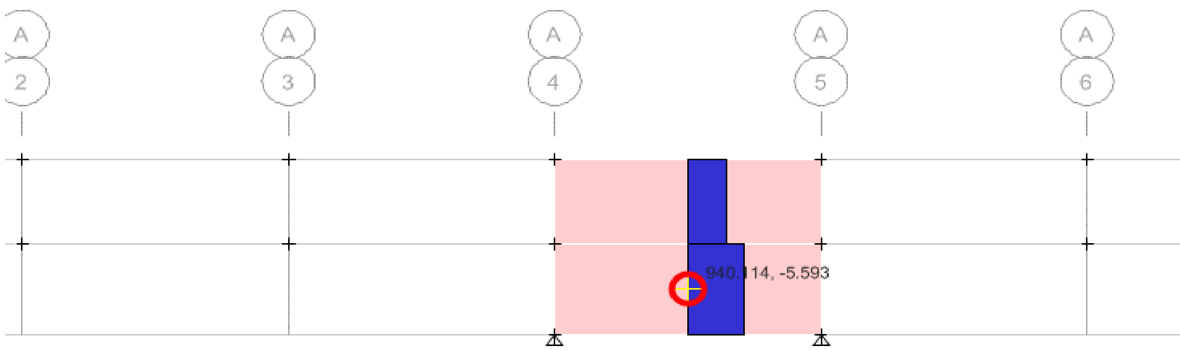
Example graphic of Beam Shear Demands under Max Eqy combo at Gridline 2. (kips)



Example graphic of Shear Wall Moments under Max Eqx Combo at Gridline A. (k-ft)



Example graphic of Shear Wall Shears under Max Eqx Combo at Gridline A. (kips)



TITLE/PROJECT NAME	DATE	JOB #
Mont. Count. Court. Garage	**11/29/17**	**17661**
SECTION	ENG/CKR	PAGE
2nd Floor Column	**GJC**	**1**

Col. Level: **2 to 3** **Col. Group:** **1** m= **1.00** k= **0.90**

Column Data

f'_c =	6	ksi
F_y=	75	ksi
F_v=	50	ksi
Col. Clr Ht. =	9	ft
BC =	2	Fix-Fix
L_{eff} =	54	in
$L_p = 0.08L_{eff}+0.15F_yd_{bl}$		
L_p =	19.79	in

Column Dimensions

h =	24	in
b =	24	in
$cover_{ties}$ =	2	in
d =	20.813	in
A_g =	452.39	in^2

Column Reinforcing

$Reinf_{long}$ =	(4) # 11	
A_s =	4.50	in^2
ties =	# 4	
spacing =	12	in o.c.
A_v =	0.40	in^2
ρ_v =	0.0018	
V_c =	77.38	k
V_s =	34.69	k

FRP data

n =	2	plies
t_f =	0.051	in
w_f =	12	in
$A_{fv} = 2nt_fw_f$		in^2
A_{fv} =	2.448	
ε_j =	0.004	
ε_{ju} =	0.022	
E_j =	3000	ksi
f_j =	12	ksi
f_{ju} =	66	ksi
ϑ =	90	°
s_j =	12	in
ψ_f =	0.95	
$\rho_j = 2t_f((B+H)/(BH))$		
ρ_j =	0.017	

Column Shear Enhancement

ACI 440.2R-08

$V_f = A_{fv}f_{fe}(\sin\alpha + \cos\alpha)d_{fv}/s_f$

V_f = 58.75 k

Seible and Innamorato

ϑ = 45 deg

$V_f = 2f_jt_jD\cot\varphi$

V_f = 58.75 k

ICC-ES AC125

$V_f = 2.86tjf_jH\sin^2\varphi$

V_f = 67.15 k

Design Shear Strength

$\varphi V_n = \varphi(V_c + V_s + \psi_f V_f)$

φ = 0.85

φV_n = **128.43** k

TITLE/PROJECT NAME	DATE	JOB #
Mont. Count. Court. Garage	11/29/17	17661
SECTION	ENG/CKR	PAGE
1st Floor Column	GJC	1

Col. Level: 1 to 2 **Col. Group:** 1 m= 1.00 k= 0.90

Column Data

f'_c =	7.5	ksi
F_y=	75	ksi
F_v=	50	ksi
Col. Clr Ht. =	9	ft
BC =	2	Fix-Fix
L_{eff} =	54	in
$L_p = 0.08L_{eff}+0.15F_yd_{bl}$		
L_p =	19.79	in

Column Dimensions

h =	24	in
b =	24	in
$cover_{ties}$ =	2	in
d =	20.813	in
A_g =	452.39	in^2

Column Reinforcing

$Reinf_{long}$ =	(4) # 11	
A_s =	4.50	in^2
ties =	# 4	
spacing =	11.5	in o.c.
A_v =	0.40	in^2
ρ_v =	0.0018	
V_c =	86.52	k
V_s =	36.20	k

FRP data

n =	3	plies
t_f =	0.051	in
w_f =	12	in
$A_{fv} = 2nt_fw_f$		in^2
A_{fv} =	3.672	
ε_j =	0.004	
ε_{ju} =	0.022	
E_j =	3000	ksi
f_j =	12	ksi
f_{ju} =	66	ksi
ϑ =	90	°
s_j =	12	in
ψ_f =	0.95	
$\rho_j = 2t_f((B+H)/(BH))$		
ρ_j =	0.0255	

Column Shear Enhancement

ACI 440.2R-08

$V_f = A_{fv}f_{fe}(\sin\alpha + \cos\alpha)d_{fv}/s_f$

V_f = 88.13 k

Seible and Innamorato

ϑ = 45 deg

$V_f = 2f_jt_jD\cot\varphi$

V_f = 88.13 k

ICC-ES AC125

$V_f = 2.86tjf_jH\sin^2\varphi$

V_f = 100.72 k

Design Shear Strength

$\varphi V_n = \varphi(V_c + V_s + \psi_f V_f)$

φ = 0.85

φV_n = 157.92 k

Section 2
Conceptual Design Cost Estimate

County of Monterey

Monterey Courthouse Complex Seismic Retrofit

Monterey, CA

RIM Architects
CONCEPTUAL DESIGN COST ESTIMATE – R2
Job No. 17257.000
20 December 2017

O'Connor Construction Management, Inc.

INTRODUCTORY NOTES

This estimate is based on verbal direction from the client and the following items, received 02 December 2017:

As-Built Plans - Courthouse	1966 Architectural A1 thru A30 (23 sheets) 1966 Structural S1 thru S15 (15 sheets) 1970 Architectural/Structural A1 thru S4 (08 sheets)
As-Built Plans - Annex	1965 Architectural A1 (01 sheet) 1965 Architectural A3 (01 sheet) 1966 Architectural A4 (01 sheet)
Photos	Courthouse Interior (09 photos) Annex Interior (15 photos) Exterior (152 photos) Basement Interior (16 photos) Multipurpose Room (01 photo) Parking Garage (64 photos) Roof (12 photos)
Specifications and Reports	Tier 1 Screening Procedure Initial Findings, ZFA Structural Engineers, November 27, 2017

The following items are excluded from this estimate:

- Professional fees.
- Building permits and fees.
- Inspections and tests.
- Furniture, fixtures & equipment.
- Moving of existing furniture and equipment.
- Installation of owner furnished equipment.
- Construction change order contingency.
- Overtime.
- Hazardous material abatement/removal.
- Items referenced as NOT INCLUDED or NIC in estimate.

The midpoint of construction of February 2021 is based on:

- Construction start date of May 2020
- Estimated construction duration of 18 months

- This estimate is based on a Design-Bid-Build delivery method.
- This estimate is based on prevailing wage labor rates.
- This estimate is based on a detailed measurement of quantities. We have made allowances for items that were not clearly defined in the drawings. The client should verify these allowances.
- This estimate is based on a minimum of four competitive bids and a stable bidding market.

INTRODUCTORY NOTES

- This estimate should be updated if more definitive information becomes available, or if there is any change in scope.
- We strongly advise the client to review this estimate in detail. If any interpretations in this estimate appear to differ from those intended by the design documents, they should be addressed immediately.

CONCEPTUAL DESIGN COST ESTIMATE - R2

OCMI JOB #: 17257.000 | 20 December 2017

PROJECT SUMMARY

ELEMENT	TOTAL COST	GFA	$/SF AREA
01. MAIN BUILDING	$7,639,070	57,300	$133.32
02. ANNEX	$3,162,716	24,210	$130.64
03. PARKING STRUCTURE	$469,822	34,200	$13.74
TOTAL CONSTRUCTION COST	**$11,271,608**		

DETAILED PROJECT SUMMARY

ELEMENT	TOTAL COST	GFA	$/SF AREA
01. MAIN BUILDING	$4,524,429	57,300	$78.96
02. ANNEX	$1,873,197	24,210	$77.37
03. PARKING STRUCTURE	$278,264	34,200	$8.14
TOTAL NET DIRECT COST	**$6,675,890**		

GENERAL MARKUPS

Design Contingency	20.0%	$1,335,178
General Conditions/Requirements	13.0%	$1,041,439
Contractor Overhead and Profit	7.0%	$633,675
Insurance	1.0%	$96,862
Bonds: Contractor	1.0%	$97,830
Bonds: Subcontractor	1.3%	$123,511
Escalation to Midpoint 02/2021	12.7%	$1,267,222
TOTAL CONSTRUCTION COST		**$11,271,608**

CONCEPTUAL DESIGN COST ESTIMATE - R2

OCMI JOB #: 17257.000 | 20 December 2017

BUILDING SUMMARY

ELEMENT		TOTAL COST	$/SF AREA
01 FOUNDATIONS		$151,179	$2.64
02 SUBSTRUCTURE		$43,657	$0.76
03 SUPERSTRUCTURE		$1,973,225	$34.44
04 EXTERIOR CLOSURE		$1,200,000	$20.94
05 ROOFING		$20,000	$0.35
06 INTERIOR CONSTRUCTION		$968,815	$16.91
07 CONVEYING			
08 MECHANICAL		$70,000	$1.22
09 ELECTRICAL		$80,000	$1.40
10 EQUIPMENT			
11 SITEWORK		$17,553	$0.31
NET DIRECT BUILDING COST		$4,524,429	$78.96
DESIGN CONTINGENCY	20.00%	$904,886	$15.79
SUBTOTAL		$5,429,315	$94.75
GENERAL CONDITIONS/REQUIREMENTS	13.00%	$705,811	$12.32
SUBTOTAL		$6,135,126	$107.07
CONTRACTOR OVERHEAD AND PROFIT	7.00%	$429,459	$7.49
SUBTOTAL		$6,564,585	$114.57
INSURANCE	1.00%	$65,646	$1.15
SUBTOTAL		$6,630,230	$115.71
BONDS: CONTRACTOR	1.00%	$66,302	$1.16
SUBTOTAL		$6,696,533	$116.87
BONDS: SUBCONTRACTOR	1.25%	$83,707	$1.46
SUBTOTAL		$6,780,239	$118.33
ESCALATION TO MIDPOINT 02/2021	12.67%	$858,830	$14.99
TOTAL BUILDING COST		**$7,639,070**	**$133.32**

GROSS FLOOR AREA: 57,300 SF

DETAILED BUILDING SUMMARY

ELEMENT	AMOUNT	TOTAL COST	$/SF AREA	TOTAL $/SF AREA
01 FOUNDATIONS		$151,179		$2.64
011 Standard Foundations	$151,179		$2.64	
012 Special Foundations				
02 SUBSTRUCTURE		$43,657		$0.76
021 Slab On Grade	$43,657		$0.76	
022 Basement Excavation				
023 Basement Walls				
03 SUPERSTRUCTURE		$1,973,225		$34.44
031 Floor and Roof Construction	$1,973,225		$34.44	
032 Stair Construction				
04 EXTERIOR CLOSURE		$1,200,000		$20.94
041 Exterior Walls	$1,200,000		$20.94	
042 Exterior Doors/Windows				
05 ROOFING		$20,000		$0.35
051 Roofing	$20,000		$0.35	
06 INTERIOR CONSTRUCTION		$968,815		$16.91
061 Partitions				
062 Interior Finishes	$968,315		$16.90	
063 Specialties				
064 Interior Doors/Windows	$500		$0.01	
07 CONVEYING				
071 Elevators				
08 MECHANICAL		$70,000		$1.22
081 Plumbing	$25,000		$0.44	
082 H.V.A.C.	$30,000		$0.52	
083 Fire Protection	$15,000		$0.26	
084 Special Mechanical				
09 ELECTRICAL		$80,000		$1.40
091 Standard Electrical	$80,000		$1.40	
092 Special Electrical				
10 EQUIPMENT				
101 Fixed/Movable Equipment				
102 Furnishings				
103 Special Construction				
11 SITEWORK		$17,553		$0.31
111 Site Preparation	$17,553		$0.31	
112 Site Improvements				
113 Site Utilities				
114 Off-Site Work				
NET DIRECT BUILDING COST		**$4,524,429**		**$78.96**

DESCRIPTION	QUANTITY	UNIT	UNIT RATE	ESTIMATED COST
ELEMENT - FOUNDATIONS				
011 STANDARD FOUNDATIONS				
Spread footings below shear walls, dowel to existing footings				
Excavation, hand	328	CY	82.92	$27,236
Drill in and epoxy dowel, #5 at 12" o.c. top and bottom at all interfaces	292	EA	76.41	$22,313
Rebar at 175 LBS/CY	36,925	LB	1.36	$50,372
Concrete	211	CY	187.93	$39,653
Backfill with sand	117	CY	36.97	$4,342
Haul	211	CY	34.42	$7,263
TOTAL - 011 STANDARD FOUNDATIONS				**$151,179**
ELEMENT - SUBSTRUCTURE				
021 SLAB ON GRADE				
Slab on grade, 5"	2,178	SF	11.28	$24,569
Drill in and epoxy dowel to join existing	360	EA	41.41	$14,928
Incidental to steel columns installation, patch	64	EA	65.00	$4,160
TOTAL - 021 SLAB ON GRADE				**$43,657**
ELEMENT - SUPERSTRUCTURE				
031 FLOOR AND ROOF CONSTRUCTION				
Shear wall, cast in place concrete, 12" thick, 25' 0" long				
4 EA per floor				
Quantities				
Formwork				
Basement	5,200	SF	19.25	$100,093
Plaza level	6,400	SF	19.25	$123,192
Third floor	6,400	SF	19.25	$123,192
Fourth floor	6,400	SF	19.25	$123,192
Concrete				
Basement	48	CY	341.90	$16,411
Plaza level	60	CY	341.90	$20,514
Third floor	56	CY	341.90	$19,146
Fourth floor	56	CY	341.90	$19,146
Rebar				
Basement 270 LBS/CY	12,960	LB	1.70	$22,084
Plaza level 185 LBS/CY	11,100	LB	1.70	$18,915
Third floor 130 LBS/CY	7,280	LB	1.70	$12,405
Fourth floor 90 LBS/CY	5,040	LB	1.70	$8,588
Dowel through existing beams				
Basement to plaza level	141	EA	395.01	$55,829
Plaza level to level 02	141	EA	395.01	$55,829
Level 02 to level 03	141	EA	395.01	$55,829

DESCRIPTION	QUANTITY	UNIT	UNIT RATE	ESTIMATED COST
Level 03 to roof structure	141	EA	395.01	$55,829
Columns				
Exterior and/or adjacently to curtainwall				
Secondary gravity system				
Tube steel directly inside of columns				
Basement, HSS9x9x5/8	12,533	LB	6.69	$83,847
Plaza level, HSS9x9x5/8	11,078	LB	6.69	$74,114
Third floor, HSS7x7x1/2	555	LB	6.69	$3,713
Fourth floor, HSS7x7x1/2	1,665	LB	6.69	$11,140
Anchor to existing slab/beam				
Basement, 8 EA per column	128	EA	248.40	$31,795
Plaza level, HSS9x9x5/8	88	EA	248.40	$21,859
Third floor, HSS7x7x1/2	8	EA	248.40	$1,987
Fourth floor, HSS7x7x1/2	24	EA	248.40	$5,962
Interior and/or not adjacently to curtainwall				
FRP reinforcement				
2-4 layers				
Basement	1,512	SF	58.00	$87,696
Plaza level, HSS9x9x5/8	3,024	SF	58.00	$175,392
Third floor, HSS7x7x1/2	4,608	SF	58.00	$267,264
Fourth floor, HSS7x7x1/2	1,728	SF	58.00	$100,224
Chamfer column corners	30,942	LF	5.00	$154,710
Collector strengthening				
at north of gridline 6 at all levels				
Option A				
steel angle app. 50 LBS/LF with epoxy anchors at 16" o.c. spacing to the existing concrete beam would extend app. 30 LF beyond the new shear wall				
L8x8x2	16,800	LB	5.50	$92,400
Epoxy anchor	261	EA	45.00	$11,728
Destructive testing				
Concrete cores	6	EA	500.00	$3,000
Reinforcing steel coupons	3	EA	400.00	$1,200
Scanning	1	LS	15,000.00	$15,000
TOTAL - 031 FLOOR AND ROOF CONSTRUCTION				**$1,973,225**

ELEMENT - EXTERIOR CLOSURE

041 EXTERIOR WALLS

DESCRIPTION	QUANTITY	UNIT	UNIT RATE	ESTIMATED COST
Precast connections				
Precast panel elements from the façade of the building				
Replace each existing connection with ductile connection, per connection	1,000	EA	1,200.00	$1,200,000

CONCEPTUAL DESIGN COST ESTIMATE - R2

DESCRIPTION	QUANTITY	UNIT	UNIT RATE	ESTIMATED COST
TOTAL - 041 EXTERIOR WALLS				**$1,200,000**
ELEMENT - ROOFING				
051 ROOFING				
Cut and patch existing incidental to shear wall and HSS column anchoring to roof structure	1	LS	20,000.00	$20,000
TOTAL - 051 ROOFING				**$20,000**
ELEMENT - INTERIOR CONSTRUCTION				
061 PARTITIONS				
Included in Element 062				
TOTAL - 061 PARTITIONS				
ELEMENT - INTERIOR CONSTRUCTION				
062 INTERIOR FINISHES				
Cut and patch existing improvements as necessary for the installation of shear walls, HSS columns and FRP wraps				
Floor				
With regular finish, carpet, vct, per location	102	EA	450.00	$45,900
With ceramic tile finish, per location	9	EA	760.00	$6,840
With premium finish, such as wood, per location	46	EA	1,100.00	$50,600
With regular finish, carpet, vct, per location	1,635	SF	12.00	$19,620
With ceramic tile finish, per location	72	SF	25.00	$1,800
With premium finish, such as wood, per location	513	SF	45.00	$23,085
Wall				
With regular finish, per location	117	EA	2,080.00	$243,360
With ceramic tile finish, per location	9	EA	2,460.00	$22,140
With premium finish, per location	46	EA	3,080.00	$141,680
Finish exposed HSS columns	371	LF	60.00	$22,230
Ceiling				
ACT, per location	102	EA	400.00	$40,800
Hard Lid, per location	9	EA	680.00	$6,120
With premium finish, such as wood, per location	46	EA	1,050.00	$48,300
Cut and patch existing incidental to exterior precast panel anchoring				
Floor, per location	500	EA	310.00	$155,000
Ceiling, per location	500	EA	190.00	$95,000
Paint as needed, Allowance per floor area	57,300	SF	0.80	$45,840
TOTAL - 062 INTERIOR FINISHES				**$968,315**

CONCEPTUAL DESIGN COST ESTIMATE - R2

DESCRIPTION	QUANTITY	UNIT	UNIT RATE	ESTIMATED COST
ELEMENT - INTERIOR CONSTRUCTION				
064 INTERIOR DOORS/WINDOWS				
Doors				
Remove and reinstall	2	EA	250.00	$500
TOTAL - 064 INTERIOR DOORS/WINDOWS				**$500**
ELEMENT - MECHANICAL				
081 PLUMBING				
Minor incidental work, Allowance	1	LS	25,000.00	$25,000
TOTAL - 081 PLUMBING				**$25,000**
ELEMENT - MECHANICAL				
082 H.V.A.C.				
Minor incidental work, Allowance	1	LS	30,000.00	$30,000
TOTAL - 082 H.V.A.C.				**$30,000**
ELEMENT - MECHANICAL				
083 FIRE PROTECTION				
Minor incidental work, Allowance	1	LS	15,000.00	$15,000
TOTAL - 083 FIRE PROTECTION				**$15,000**
ELEMENT - ELECTRICAL				
091 STANDARD ELECTRICAL				
Minor incidental work, Allowance	1	LS	80,000.00	$80,000
TOTAL - 091 STANDARD ELECTRICAL				**$80,000**
ELEMENT - SITEWORK				
111 SITE PREPARATION				
Demolition				
Incidental to new footings				
Sawcut slab on grade	360	LF	12.98	$4,679
Slab on grade	2,178	SF	2.68	$5,834
Incidental to steel columns installation, cut and patch existing concrete	64	EA	110.00	$7,040

CONCEPTUAL DESIGN COST ESTIMATE - R2

OCMI JOB #: 17257.000 | 20 December 2017

BUILDING SUMMARY

ELEMENT		TOTAL COST	$/SF AREA
01 FOUNDATIONS		$53,445	$2.21
02 SUBSTRUCTURE		$6,313	$0.26
03 SUPERSTRUCTURE		$907,267	$37.47
04 EXTERIOR CLOSURE		$504,000	$20.82
05 ROOFING		$8,400	$0.35
06 INTERIOR CONSTRUCTION		$325,613	$13.45
07 CONVEYING			
08 MECHANICAL		$29,400	$1.21
09 ELECTRICAL		$33,600	$1.39
10 EQUIPMENT			
11 SITEWORK		$5,159	$0.21
NET DIRECT BUILDING COST		$1,873,197	$77.37
DESIGN CONTINGENCY	20.00%	$374,639	$15.47
SUBTOTAL		$2,247,836	$92.85
GENERAL CONDITIONS/REQUIREMENTS	13.00%	$292,219	$12.07
SUBTOTAL		$2,540,055	$104.92
CONTRACTOR OVERHEAD AND PROFIT	7.00%	$177,804	$7.34
SUBTOTAL		$2,717,859	$112.26
INSURANCE	1.00%	$27,179	$1.12
SUBTOTAL		$2,745,038	$113.38
BONDS: CONTRACTOR	1.00%	$27,450	$1.13
SUBTOTAL		$2,772,488	$114.52
BONDS: SUBCONTRACTOR	1.25%	$34,656	$1.43
SUBTOTAL		$2,807,144	$115.95
ESCALATION TO MIDPOINT 02/2021	12.67%	$355,572	$14.69
TOTAL BUILDING COST		**$3,162,716**	**$130.64**

GROSS FLOOR AREA: 24,210 SF

DETAILED BUILDING SUMMARY

ELEMENT	AMOUNT	TOTAL COST	$/SF AREA	TOTAL $/SF AREA
01 FOUNDATIONS		$53,445		$2.21
011 Standard Foundations	$53,445		$2.21	
012 Special Foundations				
02 SUBSTRUCTURE		$6,313		$0.26
021 Slab On Grade	$6,313		$0.26	
022 Basement Excavation				
023 Basement Walls				
03 SUPERSTRUCTURE		$907,267		$37.47
031 Floor and Roof Construction	$907,267		$37.47	
032 Stair Construction				
04 EXTERIOR CLOSURE		$504,000		$20.82
041 Exterior Walls	$504,000		$20.82	
042 Exterior Doors/Windows				
05 ROOFING		$8,400		$0.35
051 Roofing	$8,400		$0.35	
06 INTERIOR CONSTRUCTION		$325,613		$13.45
061 Partitions				
062 Interior Finishes	$325,113		$13.43	
063 Specialties				
064 Interior Doors/Windows	$500		$0.02	
07 CONVEYING				
071 Elevators				
08 MECHANICAL		$29,400		$1.21
081 Plumbing	$10,500		$0.43	
082 H.V.A.C.	$12,600		$0.52	
083 Fire Protection	$6,300		$0.26	
084 Special Mechanical				
09 ELECTRICAL		$33,600		$1.39
091 Standard Electrical	$33,600		$1.39	
092 Special Electrical				
10 EQUIPMENT				
101 Fixed/Movable Equipment				
102 Furnishings				
103 Special Construction				
11 SITEWORK		$5,159		$0.21
111 Site Preparation	$5,159		$0.21	
112 Site Improvements				
113 Site Utilities				
114 Off-Site Work				
NET DIRECT BUILDING COST		**$1,873,197**		**$77.37**

CONCEPTUAL DESIGN COST ESTIMATE - R2

DESCRIPTION	QUANTITY	UNIT	UNIT RATE	ESTIMATED COST
ELEMENT - FOUNDATIONS				
011 STANDARD FOUNDATIONS				
Spread footings below shear walls, Figure 8				
Connect new to existing footing				
Spread footings				
Excavation, hand	80	CY	82.92	$6,592
Drill in and epoxy dowel, #5 at 12" o.c. top and bottom at all interfaces	292	EA	76.41	$22,313
Rebar at 175 LBS/CY	9,275	LB	1.36	$12,653
Concrete	53	CY	187.93	$9,960
Backfill with sand	27	CY	36.97	$998
Haul	27	CY	34.42	$929
TOTAL - 011 STANDARD FOUNDATIONS				**$53,445**
ELEMENT - SUBSTRUCTURE				
021 SLAB ON GRADE				
Slab on grade	540	SF	2.68	$1,446
Drill in and epoxy dowel to join existing	89	EA	41.41	$3,697
Incidental to steel columns installation, patch	18	EA	65.00	$1,170
TOTAL - 021 SLAB ON GRADE				**$6,313**
ELEMENT - SUPERSTRUCTURE				
031 FLOOR AND ROOF CONSTRUCTION				
Shear wall, cast in place concrete, 12" thick, 12' 0" long				
5 EA per floor				
Quantities				
Formwork				
Basement	1,320	SF	19.25	$25,408
Plaza level	1,320	SF	19.25	$25,408
Concrete				
Basement	24	CY	341.90	$8,358
Plaza level	24	CY	341.90	$8,358
Rebar				
Basement 270 LBS/CY	2,673	LB	1.70	$4,555
Plaza level 185 LBS/CY	2,673	LB	1.70	$4,555
Dowel through existing beams				
Basement to plaza level	80	EA	324.81	$25,985
Plaza level to level 02	80	EA	324.81	$25,985
Level 03 to roof structure	80	EA	324.81	$25,985
Columns				
Exterior and/or adjacently to curtainwall				
Secondary gravity system				
Tube steel directly inside of columns				

DESCRIPTION	QUANTITY	UNIT	UNIT RATE	ESTIMATED COST
Basement, HSS9x9x5/8	14,883	LB	6.69	$99,568
Plaza level, HSS9x9x5/8	19,135	LB	6.69	$128,016
Anchor to existing slab/beam				
Basement, 8 EA per column	152	EA	318.60	$48,427
Plaza level, HSS9x9x5/8	152	EA	318.60	$48,427
Interior and/or not adjacently to curtainwall				
FRP reinforcement				
2-4 layers				
Basement	3,024	SF	58.00	$175,392
Plaza level	3,024	SF	58.00	$175,392
Chamfer column corners	1,584	LF	5.00	$7,920
Collector strengthening				
at north of gridline 6 at all levels				
Option A				
Steel angle app. 50 LBS/LF with epoxy anchors at 16" o.c. spacing to the existing concrete beam would extend app. 30 LF beyond the new shear wall				
L8x8x2	7,056	LB	5.50	$38,808
Epoxy anchor	256	EA	45.00	$11,520
Destructive testing				
Concrete cores	6	EA	500.00	$3,000
Reinforcing steel coupons	3	EA	400.00	$1,200
Scanning	1	LS	15,000.00	$15,000
TOTAL - 031 FLOOR AND ROOF CONSTRUCTION				**$907,267**
ELEMENT - EXTERIOR CLOSURE				
041 EXTERIOR WALLS				
Precast connections				
Precast panel elements from the façade of the building				
Replace each existing connection with ductile connection, per connection	420	EA	1,200.00	$504,000
TOTAL - 041 EXTERIOR WALLS				**$504,000**
ELEMENT - ROOFING				
051 ROOFING				
Cut and patch existing incidental to shear wall and HSS column anchoring to roof structure	1	LS	8,400.00	$8,400
TOTAL - 051 ROOFING				**$8,400**

CONCEPTUAL DESIGN COST ESTIMATE - R2

DESCRIPTION	QUANTITY	UNIT	UNIT RATE	ESTIMATED COST
ELEMENT - INTERIOR CONSTRUCTION				
061 PARTITIONS				
Included in Element 062				
TOTAL - 061 PARTITIONS				
ELEMENT - INTERIOR CONSTRUCTION				
062 INTERIOR FINISHES				
Cut and patch existing improvements as necessary for the installation of shear walls, HSS columns and FRP wraps				
Floor				
With regular finish, carpet, vct, per location	43	EA	450.00	$19,278
With ceramic tile finish, per location	2	EA	760.00	$1,520
With premium finish, such as wood, per location	2	EA	1,100.00	$2,200
With regular finish, carpet, vct, per location	687	SF	12.00	$8,240
Wall				
With regular finish, per location	49	EA	2,080.00	$102,211
With ceramic tile finish, per location	2	EA	2,460.00	$4,920
With premium finish, per location	2	EA	3,080.00	$6,160
Finish exposed HSS columns	396	LF	60.00	$23,760
Ceiling				
ACT, per location	43	EA	400.00	$17,136
Hard Lid, per location	4	EA	680.00	$2,720
With premium finish, such as wood, per location	12	EA	1,050.00	$12,600
Cut and patch existing incidental to exterior precast panel anchoring				
Floor, per location	210	EA	310.00	$65,100
Ceiling, per location	210	EA	190.00	$39,900
Paint as needed, Allowance per floor area	24,210	SF	0.80	$19,368
TOTAL - 062 INTERIOR FINISHES				**$325,113**
ELEMENT - INTERIOR CONSTRUCTION				
064 INTERIOR DOORS/WINDOWS				
Doors				
Remove and reinstall	2	EA	250.00	$500
TOTAL - 064 INTERIOR DOORS/WINDOWS				**$500**
ELEMENT - MECHANICAL				
081 PLUMBING				
Minor incidental work, Allowance	1	LS	10,500.00	$10,500

DESCRIPTION	QUANTITY	UNIT	UNIT RATE	ESTIMATED COST
TOTAL - 081 PLUMBING				**$10,500**
ELEMENT - MECHANICAL				
082 H.V.A.C.				
Minor incidental work, Allowance	1	LS	12,600.00	$12,600
TOTAL - 082 H.V.A.C.				**$12,600**
ELEMENT - MECHANICAL				
083 FIRE PROTECTION				
Minor incidental work, Allowance	1	LS	6,300.00	$6,300
TOTAL - 083 FIRE PROTECTION				**$6,300**
ELEMENT - ELECTRICAL				
091 STANDARD ELECTRICAL				
Minor incidental work, Allowance	1	LS	33,600.00	$33,600
TOTAL - 091 STANDARD ELECTRICAL				**$33,600**
ELEMENT - SITEWORK				
111 SITE PREPARATION				
Demolition				
Incidental to new footings				
Sawcut slab on grade	134	LF	12.98	$1,733
Slab on grade	540	SF	2.68	$1,446
Incidental to steel columns installation, cut and patch existing concrete	18	EA	110.00	$1,980
TOTAL - 111 SITE PREPARATION				**$5,159**

CONCEPTUAL DESIGN COST ESTIMATE - R2

BUILDING SUMMARY

ELEMENT		TOTAL COST	$/SF AREA
01 FOUNDATIONS		$24,843	$0.73
02 SUBSTRUCTURE			
03 SUPERSTRUCTURE		$238,421	$6.97
04 EXTERIOR CLOSURE			
05 ROOFING			
06 INTERIOR CONSTRUCTION		$15,000	$0.44
07 CONVEYING			
08 MECHANICAL			
09 ELECTRICAL			
10 EQUIPMENT			
11 SITEWORK			
NET DIRECT BUILDING COST		$278,264	$8.14
DESIGN CONTINGENCY	20.00%	$55,653	$1.63
SUBTOTAL		$333,917	$9.76
GENERAL CONDITIONS/REQUIREMENTS	13.00%	$43,409	$1.27
SUBTOTAL		$377,326	$11.03
CONTRACTOR OVERHEAD AND PROFIT	7.00%	$26,413	$0.77
SUBTOTAL		$403,739	$11.81
INSURANCE	1.00%	$4,037	$0.12
SUBTOTAL		$407,776	$11.92
BONDS: CONTRACTOR	1.00%	$4,078	$0.12
SUBTOTAL		$411,854	$12.04
BONDS: SUBCONTRACTOR	1.25%	$5,148	$0.15
SUBTOTAL		$417,002	$12.19
ESCALATION TO MIDPOINT 02/2021	12.67%	$52,820	$1.54
TOTAL BUILDING COST		**$469,822**	**$13.74**

GROSS FLOOR AREA: 34,200 SF

CONCEPTUAL DESIGN COST ESTIMATE - R2

OCMI JOB #: 17257.000 | 20 December 2017

DETAILED BUILDING SUMMARY

ELEMENT	AMOUNT	TOTAL COST	$/SF AREA	TOTAL $/SF AREA
01 FOUNDATIONS		$24,843		$0.73
011 Standard Foundations	$6,443		$0.19	
012 Special Foundations	$18,400		$0.54	
02 SUBSTRUCTURE				
021 Slab On Grade				
022 Basement Excavation				
023 Basement Walls				
03 SUPERSTRUCTURE		$238,421		$6.97
031 Floor and Roof Construction	$238,421		$6.97	
032 Stair Construction				
04 EXTERIOR CLOSURE				
041 Exterior Walls				
042 Exterior Doors/Windows				
05 ROOFING				
051 Roofing				
06 INTERIOR CONSTRUCTION		$15,000		$0.44
061 Partitions				
062 Interior Finishes	$15,000		$0.44	
063 Specialties				
064 Interior Doors/Windows				
07 CONVEYING				
071 Elevators				
08 MECHANICAL				
081 Plumbing				
082 H.V.A.C.				
083 Fire Protection				
084 Special Mechanical				
09 ELECTRICAL				
091 Standard Electrical				
092 Special Electrical				
10 EQUIPMENT				
101 Fixed/Movable Equipment				
102 Furnishings				
103 Special Construction				
11 SITEWORK				
111 Site Preparation				
112 Site Improvements				
113 Site Utilities				
114 Off-Site Work				
NET DIRECT BUILDING COST		**$278,264**		**$8.14**

CONCEPTUAL DESIGN COST ESTIMATE - R2

DESCRIPTION	QUANTITY	UNIT	UNIT RATE	ESTIMATED COST
ELEMENT - FOUNDATIONS				
011 STANDARD FOUNDATIONS				
Pile caps	8	CY	805.44	$6,443
TOTAL - 011 STANDARD FOUNDATIONS				**$6,443**
ELEMENT - FOUNDATIONS				
012 SPECIAL FOUNDATIONS				
Helical piles, 8 EA, assume 20' 0" depth	160	LF	65.00	$10,400
Mobilization	1	LS	8,000.00	$8,000
TOTAL - 012 SPECIAL FOUNDATIONS				**$18,400**
ELEMENT - SUPERSTRUCTURE				
031 FLOOR AND ROOF CONSTRUCTION				
Shear wall, cast in place concrete, 10" thick, rebar #5				
18" on center each way, each side				
from grade to the roof structure, 3 levels				
Dowel reinforcing through the existing slabs				
Roughen existing slab surfaces to 1/4" amplitude				
Quantities				
Form				
Ground	587	SF	15.25	$8,950
First floor	405	SF	13.25	$5,363
Second floor	380	SF	13.25	$5,034
Concrete				
Ground	107	CY	314.90	$33,799
First floor	75	CY	281.90	$21,049
Second floor	70	CY	281.90	$19,733
Rebar				
Ground floor 95 LBS/CY of concrete	10,197	LB	1.60	$16,356
First floor 95 LBS/CY of concrete	7,537	LB	1.40	$10,582
Second floor 95 LBS/CY of concrete	6,650	LB	1.40	$9,337
Dowel through existing slabs				
Ground	36	EA	76.41	$2,734
First floor	36	EA	76.41	$2,734
Second floor	36	EA	76.41	$2,734
Column reinforcement with fiber reinforced polymer				
jacketing				
First floor	768	SF	62.00	$47,616
Second floor	720	SF	55.00	$39,600
Chamfer column corners	720	LF	5.00	$3,600
Destructive testing				

CONCEPTUAL DESIGN COST ESTIMATE - R2

DESCRIPTION	QUANTITY	UNIT	UNIT RATE	ESTIMATED COST
Concrete cores	6	EA	500.00	$3,000
Reinforcing steel coupons	3	EA	400.00	$1,200
Scanning	1	LS	5,000.00	$5,000
TOTAL - 031 FLOOR AND ROOF CONSTRUCTION				**$238,421**
ELEMENT - INTERIOR CONSTRUCTION				
062 INTERIOR FINISHES				
Miscellaneous incidental work	1	LS	15,000.00	$15,000
TOTAL - 062 INTERIOR FINISHES				**$15,000**