Sacramento County Correctional Facility Master Planning Services

Final Report: A Vision for the Future

Appendix B: Main Jail, Seismic Resistance Analysis

HMA DLRGROUP
Published on October 30, 2026
Prepared at the Direction of Counsel

MARTIN/MARTIN
CONSULTING ENGINEERS

Sacramento Main Jail

Structural Evaluation Report



Property Address: 651 I Street, Sacramento, CA 95814

Report Date: September 21, 2026

Produced For: DLR Group

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Project No.: 26.0417.S.01

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I. EXECUTIVE SUMMARY

During the Phase II assessment of existing physical conditions, it was determined that a structural assessment relative to seismic resistance and current code requirements had not been completed for the Sacramento Main Jail. As a result, concurrently with the analysis of Strategic Development Approaches for the future, the county authorized the Planning Team to prepare a structural assessment. Martin/ Martin was retained to perform a Tier I and Tier II analysis of the existing structure per ASCE 41-17.

The facility, constructed in 1984, consists of a reinforced concrete tower supported by a podium structure and deep pile foundations. While the building has adequately performed in service and exhibits significant structural redundancy, it was designed to seismic standards that predate current requirements for essential public safety facilities. The evaluation found that the jail is expected to provide life-safety performance during a major earthquake. However, the building does not meet the Immediate Occupancy performance objective that would be expected for a critical facility under current seismic standards. Significant structural damage and extended operational disruption could occur following a design-level earthquake.

The most significant vulnerabilities are concentrated at the building's transfer level, where large transfer girders redirect seismic forces from the tower into the podium and foundation system. Several of these girders were found to exceed acceptance criteria, creating potential weaknesses in the building's vertical load path during a seismic event. Additional deficiencies were identified in diaphragm force-transfer regions, portions of the reinforced concrete shear wall system, and localized foundation components. The site's location within a mapped liquefaction hazard area also warrants further geotechnical investigation.

From the Tier 2 analysis, conceptual retrofit schemes were developed which focus on strengthening transfer girders, diaphragm connections, and providing foundation continuity. To achieve an Immediate Occupancy performance objective as required per current standards, additional analysis and retrofit is recommended to better define retrofit requirements and expected performance of the structure through an ASCE 41 Tier 3 evaluation.

The study also concluded that proposed operational improvements, including ADA upgrades, housing reconfiguration, medical and program space modifications, recreation area improvements, and future expansion opportunities, are generally feasible and can be incorporated into future modernization efforts.

In conclusion, the Sacramento Main Jail is a fundamentally robust structure; however, it does not meet current performance expectations for a critical facility. There are not enough resources available to relocate 2,000+ people in custody, should the Main Jail experience downtime following a seismic hazard event. Strategic seismic upgrades are recommended to improve resiliency, avoid post-earthquake downtime, and support the facility's long-term operational needs.

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II. INTRODUCTION

The County of Sacramento has requested to conduct a structural evaluation of the Main Jail to identify structural deficiencies and determine the need for retrofit improvements. This evaluation primarily concerns seismic performance of the structure, as the original design in 1984 is inconsistent with today's structural design provisions. Per the 2025 California Building Code, this building would be considered a Risk Category IV structure if designed with the latest provisions. It is understood from the record documents that the original structure did not consider an 'Importance Factor' that would be required for Essential Facilities. It is presented and discussed within this report that the lower level of seismic forces used in the original design are the primary cause for the deficient components in the seismic force resisting system.

In conjunction with the structural evaluation, the proposed reprogramming, accessibility and utilization upgrades to the Main Jail are also studied based on the *Main Jail - Opportunities for Improvement* presentation prepared by Health Management Associates. This report also presents the proposed improvements and establishes the overall feasibility of implementation. The recommendations derived from this evaluation are to be incorporated into the ongoing Facilities Conditions Assessment (FCA) work.

A. EXISTING DOCUMENTATION

The record drawings for design of the Main Jail completed in 1984 were provided. From this information, the following summary table was developed.

Table 1. Existing Building Information Summary

Year Originally Built:	1984
No. Stories Above Grade:	10 (1 Podium levels, 8 Tower levels w/ Mezzanines, Penthouse)
No. Stories Below Grade:	1 (Basement level)
Original Design Code:	Uniform Building Code - 1982
History Of Significant Structural Alteration (Non-Seismic)	No known significant structural alterations.
History Of Seismic Retrofit	No known significant seismic retrofits.

B. BUILDING DESCRIPTION

From the record drawings, the structural framing description for the building was determined. The primary structural systems are itemized in this section.

FOUNDATION
The foundation system consists of bearing walls and columns over pile caps. The piles are 14-inch square prestressed piles with a minimum length of 64 feet. Information about the soil design assumptions, and pile design capacities are provided in the record geotechnical reports.

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PODIUM

The podium consists of two levels (ground floor, second floor). The perimeter bearing walls of the podium are retaining walls from the basement level to ground level. The lateral system for these levels is concrete shear walls. The floor framing utilizes concrete 1-way, 2-way, and waffle slab floor systems over concrete beams and girders.

TOWER

The tower consists of concrete 1-way, 2-way, and waffle slab floor systems over concrete beams and girders. At the perimeter of each quadrant of the tower are concrete moment frames. Per the record drawings these are ductile frames and there is a total of 22 frame elevations. This tower utilizes a dual lateral system with interior concrete shear walls at the stair cores, elevator core, and cell dividing walls perpendicular to the perimeter moment frames. Between each tower floor there is a mezzanine level that provides access to each cell block. Floor-to-floor heights including mezzanines are 9'-0" typically.

PENTHOUSE

The penthouse is steel framed with steel moment frames at select locations about the perimeter of the story. The roof consists of a lightweight concrete fill over metal deck.

C. SITE VISIT

Patrick Crosby, PE, SE, F. DBIA and Matthew Roloff, PE visited the site on June 10, 2026, to assess the existing condition of this facility, gather data necessary to complete the Tier 1 and Tier 2 evaluation, and to determine if any modifications have been made to the structure. With this, several locations for the proposed tenant improvement alternatives were also observed to provide a thorough understanding of the structural impact of proceeding with the improvements.

Several primary structural components of the building were observed across all floor levels. No structural deterioration or cracking was observed for the visibly accessible structural components across the building.

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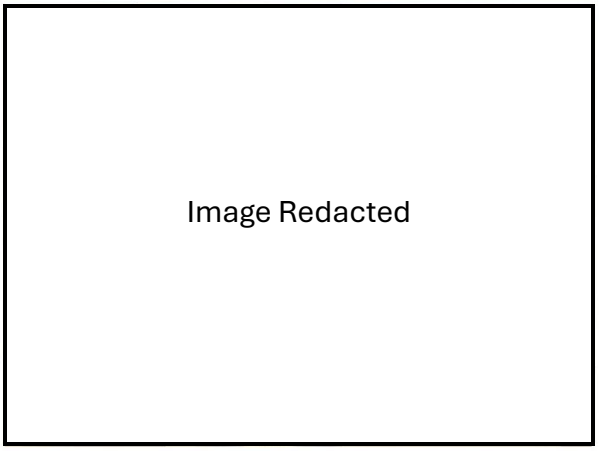


Figure 1. Waffle slab and intermediate girder. The record drawings noted 6" minimum rib thickness and a top width of 8". At the site it can be observed that these rib sections are wider (approx. 8" wide at the tip and 10" at the widest section).

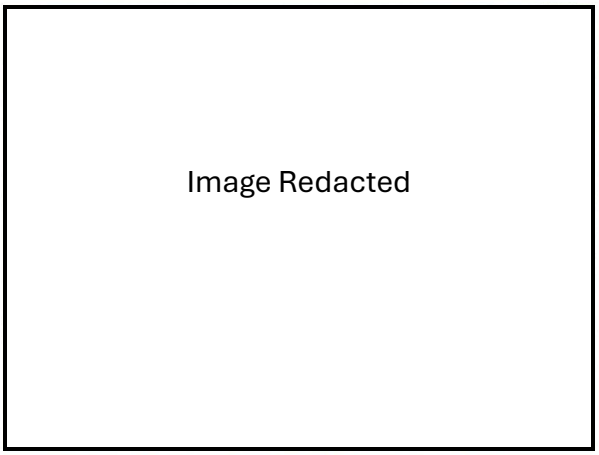


Figure 2. Exposed 7ft deep transfer girder at Sallyport. This girder is located along Grid 2.4 and supports moment frame 'D'.

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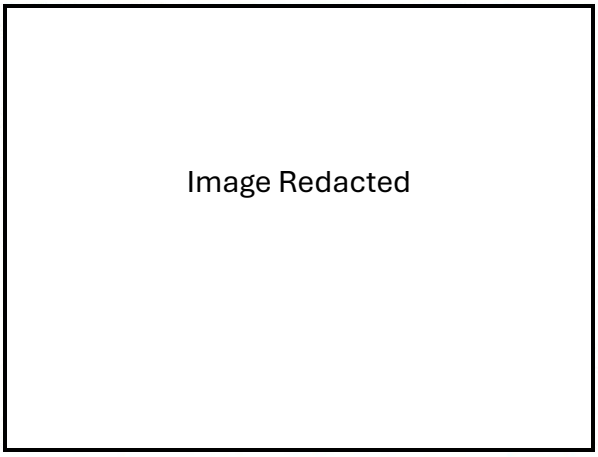


Figure 3. Recreation Yard overview. It is observed that most walls appear to be concrete, however from the record drawings it is determined that only identified piers are structural components. Through field verification and drawing review it is confirmed that a majority of the wall infill is a combination of metal stud with plaster and lath finish to give the appearance of concrete.

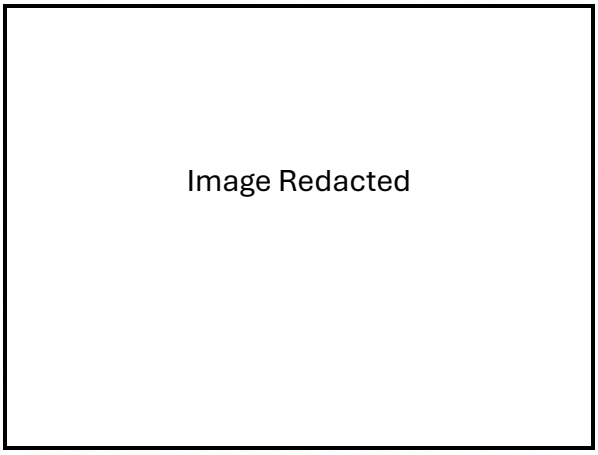


Figure 4. Penthouse moment frame connection and precast cladding attachment. The identified moment frame beams and columns are substantially large in size.

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III. ANALYSIS CRITERIA

Martin/Martin, Inc. (Martin/Martin) evaluated the subject building using the California Existing Building Code (CEBC), and the American Society of Civil Engineers (ASCE) Standard 41-17, *Seismic Evaluation and Retrofit of Existing Buildings* (ASCE 41-17). We identified and evaluated deficiencies using ASCE 41-17 Tier 1 and Tier 2 procedures through hand calculations and an analytical model.

The building is evaluated for the performance objectives of Immediate Occupancy using the Seismic Hazard Level of BSE-1E, and Life Safety Structural Performance using the Seismic Hazard Level BSE-2E. The Immediate Occupancy objective sets the criteria to allow for the structure to remain safe and retain its pre-earthquake strength. The Life Safety objective sets the criteria to allow for post-earthquake damage in the structure while still retaining a margin of safety against partial or total collapse. The BSE-1E Hazard Level is based on a probabilistic hazard with a 20% in 50-year probability of exceedance and the BSE-2E Hazard Level is based on a probabilistic hazard with a 5% in 50-year probability of exceedance. At this time, formal gravity actions and non-structural evaluations are excluded from this study.

The original building was constructed before 2000, which means that the building does not meet the benchmark requirements (ASCE 41-17 Section 3.3) and must be evaluated in accordance with ASCE 41-17.

A. SITE SEISMICITY PARAMETERS

The Multi-Period General Horizontal Response Spectrum was developed using the *ASCE Hazard Tool* with a site class of D. This response spectrum defines the peak acceleration of the building based on the specific site location and soil properties. From the record geotechnical report, the categorized site class per the current adopted ASCE 7-22 provision would classify the site as CD. It was decided to increase the site class by one tier so as to not underestimate loading demand. The Seismicity Level is considered *High* per Section 2.5. This site is also identified to be located within a liquefaction zone. At this time no soil-structure-interaction or liquefaction effects are considered.

B. TIER 1 SCREENING CHECKLISTS DESCRIPTION

From review of the existing documentation, the building is identified as Type C1 and Type C2 which correspond to Concrete Moment Frames and concrete Shear Walls. Note that this building utilizes a combination of systems in the same direction for both orthogonal directions. The Tier 1 screening is based on the following (3) types of checklists: (1) a Basic Configuration Checklist, (2) a checklist specific to the building structural system (in this case, multiple systems: C1 and C2), and (3) a checklist specific to the nonstructural systems within the building. Note that nonstructural systems are not included in this evaluation. Each checklist item is categorized by its observed condition of Compliant, Noncompliant, Not Applicable, or Unknown (C, NC, NA, U). Completed Tier 1 checklists are provided in Appendix A.

C. TIER 2 ANALYSIS DESCRIPTION

Where a potential deficiency is identified per Tier 1 checklists, a Tier 2 procedure is required to analyze the structure or specific component of the structure to confirm whether a flagged item is deficient or not. In accordance with Section 5.2.4, a Linear Dynamic Procedure is conducted by Section 7.4.2. Component

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gravity loads and combinations are in accordance with 7.2.3. In accordance with Section 5.2.5, the acceptance criteria for Tier 2 are established per 7.5.2.2. The design actions are in accordance with Section 7.5.2.1, and component capacities are in accordance with Section 7.5.2.2 and Chapters 8 through 12. Per Section 6.2.3.1, the knowledge factor *k* is taken as 1.0 for material strengths defined in the construction documents.

IV. SEISMIC EVALUATION

A. TIER 1 DEFICIENCIES

The deficiencies identified in the Tier 1 checklists and corresponding Tier 2 evaluation are further discussed herein. We describe the deficiency within the structure's context (X.a) followed by an explanation about how the deficiency was further evaluated to determine whether retrofit is required (X.b). There are 16 identified deficiencies out of a total of 51 applicable Tier 1 evaluation checks.

CHECKLIST 17.1.2IO: BASIC CONFIGURATION

1. Building System General - Load Path
   a. There are several instances where the concrete frames do not have a direct load path down into the foundations below. This is apparent at the second level where the moment frames terminate, and the lateral system becomes a shear wall system only. These frames sit on transfer girders that span to supporting walls or columns.
   b. There is no Tier 2 deficiency-based evaluation for this condition, and further analysis is required. Both the supporting transfer girders, and the diaphragms required to transfer loading from the frames to parallel walls, are evaluated for adequacy.

2. Configuration - Vertical Irregularities
   a. This item is consistent with the *Load Path* deficiency identified. The vertical irregularity is due to the frequent use of transfer girders at particular frames.
   b. The Tier 2 deficiency-based procedure requires analysis of the elements and connections below the vertical discontinuities to be evaluated as force-controlled elements per section 5.4.2.3. Struts and diaphragm adequacy shall also be evaluated at the transfer level per section 5.4.2.3.

3. Building System Configuration - Geometry
   a. At the transition level, the net horizontal dimension of the seismic-force-resisting system changes by more than 30% at the transition from a moment frame system to a shear wall system.
   b. The Tier 2 deficiency-based procedure requires analysis using a linear dynamic procedure per section 5.4.2.4.

4. Building System Configuration - Mass
   a. Due to the configuration of the mass in the building, particularly at the podium level, there is a substantial mass difference between the lower levels versus a typical tower level.

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b. The Tier 2 deficiency-based procedure requires an analysis of the seismic-force-resisting elements per section 5.4.2.5.

5. Geological Site Hazards - Liquefaction
   a. The building is identified to be located within a liquefaction zone per the California Geological Survey. The site consists of soft soil located near the Sacramento River.
   b. There is no Tier 2 deficiency-based procedure for analysis of geological site hazards. The foundations are further evaluated based on the record geotechnical report and structural foundation drawings. A Tier 3 analysis may be required and is outside the scope of this report.

6. Foundation Configuration - Overturning
   a. Due to the geometry of the building, the moment frame and shear wall aspect ratios are higher than the limitation set by the Tier 1 screening.
   b. The Tier 2 deficiency-based procedure requires an analysis of the seismic-force-resisting elements per section 5.4.3.3.

7. Foundation Configuration - Ties Between Foundation Elements
   a. From the drawings it is observed that pile caps are not explicitly tied together. There are no discrete ties, and the basement slab does not dowel to any of the foundations.
   b. The Tier 2 deficiency-based procedure requires that the magnitude of differential movement in the foundation be evaluated, and the foundation system be evaluated for all gravity and seismic forces in combination with the forces induced by potential differential settlement per section 5.4.3.4.

CHECKLIST 17.11IO: BUILDING TYPE C1 - CONCRETE MOMENT FRAMES

8. SFRS - Redundancy
   a. The total number of lines of frames is adequate, and the total number of bays is greater than or equal to 3 for all frames except for Frame B and Frame X.
   b. The Tier 2 deficiency-based procedure requires an analysis of the seismic-force-resisting elements per section 5.5.1.1.

9. SFRS – No Shear Failures
   a. It was unknown at the time of review whether this criteria was met.
   b. The Tier 2 deficiency-based procedure requires an analysis of the seismic-force-resisting elements per section 5.5.2.3.4.

10. SFRS – Strong Column – Weak Beam
    a. Based on analysis of the components, the moment capacity of the moment frame beams was greater than the capacity of columns.
    b. The Tier 2 deficiency-based procedure requires an analysis of the seismic-force-resisting elements with modification to m-factors per section 5.5.2.1.5.

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11. Connections - Concrete Columns
    a. Consistent with the *Vertical Irregularities* deficiency, this item is noncompliant since several of the frame columns do not have a direct load path to the foundation and instead rely on transfer girders to resolve axial load.
    b. The Tier 2 deficiency-based procedure requires an analysis of the connection between moment frame column connection to transfer girders be evaluated per section 5.7.3.1.

12. Connections – Uplift at Pile Caps
    a. The prestressed piles do have a positive attachment to pile caps though do not appear to be substantial.
    b. The Tier 2 deficiency-based procedure requires that pile connections and pile cap reinforcement be evaluated for uplift and shear forces per section 5.7.3.5.

CHECKLIST 17.12IO: BUILDING TYPE C2 - CONCRETE SHEAR WALLS WITH STIFF DIAPHRAGMS

13. SFRS – Complete Frames
    a. Many of the shear walls also act as gravity supporting walls.
    b. The Tier 2 deficiency-based procedure requires an analysis of the seismic-force-resisting elements considering the combined gravity and seismic demands per section 5.5.2.5.1.

14. Diaphragms – Openings at Shear Walls
    a. Both elevator and stair core walls feature openings adjacent to shear walls.
    b. The Tier 2 deficiency-based procedure requires the diaphragm forces and adequacy of the diaphragm is evaluated to provide acceptable transfer of the loads to the walls considering the available length and presence of any drag struts per section 5.6.1.3. The out-of-plane wall capability between support points shall also be evaluated.

15. Connections – Uplift at Pile Caps
    a. The prestressed piles do have a positive attachment to pile caps though do not appear to be substantial.
    b. The Tier 2 deficiency-based procedure requires that pile connections and pile cap reinforcement be evaluated for uplift and shear forces per section 5.7.3.5.

16. Foundation System - Deep Foundations
    a. At the time of completion of the Tier 1 screening, it was unclear whether the piles were capable of transferring lateral forces between the structure and the soil.
    b. There is no Tier 2 deficiency-based evaluation for this condition, and further analysis is required. See the foundation analysis in the following section.

B. TIER 2 DIFICIENCY BASED ANALYSIS

ANALYTICAL MODEL DESCRIPTION

A three-dimensional analysis model was created in ETABS 2023 to represent the structural behavior of this building. All primary structural beams, columns, walls, and slabs that are part of the vertical and seismic system were included. Additional loading was included for the self-weight of the precast cladding panels,

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and an additional super-imposed dead load was included to represent the miscellaneous architectural, mechanical, electrical, and plumbing systems. Live loading was assigned based on the programming from the architectural drawings.

From the analytical model, the fundamental period was determined and compared against the empirical formula from the ASCE 41-17 equation 7-18. Since this building consists of dual lateral systems for the tower, we expect the analytical period to be in between the bounds for concrete moment frame and other systems and shown in the table below. The analytical periods determined by the ETABS model were used during response spectrum analysis.

Table 2. Fundamental Period Determination

Method	Period
ETABS Model	1.41 sec
Empirical (concrete moment frame)	1.76 sec
Empirical (all other framing systems)	0.91 sec

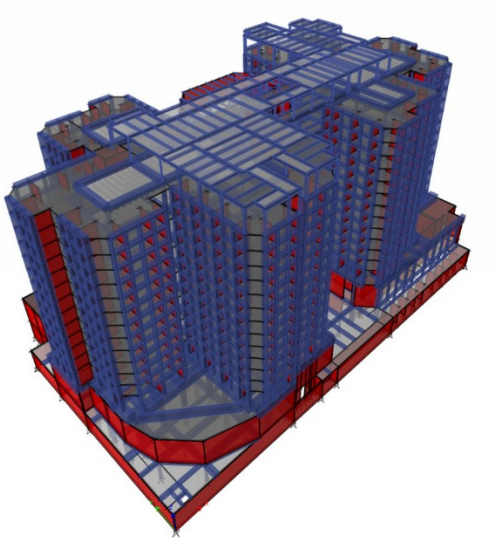


Figure 5. Three-dimensional visualization of the analytical ETABS model used in evaluation structural components of the Main Jail per ASCE 41-17. This model was developed using the record structural and architectural drawings.

SEISMIC LOADING

A linear response spectrum analysis was performed per the described analysis criteria and includes 5% modal damping as prescribed for per section 7.2.3.6. The number of modes used to compute the forces for the response spectrum analysis were selected such that there is 90% modal mass participation in each

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orthogonal direction. Stiffness and strength properties of structural members are modified as required by Table 10-5 in ASCE 41-17.

FOUNDATIONS

The foundations were assessed to determine if adequate number of piles exist beneath the lateral force resisting wall segments for seismic loading from BSE-1E and BSE-2E hazard levels. Pile capacities were provided from the record geotechnical report. Through deformation-controlled action, the number of piles required beneath a given shear wall may be compared against the actual number of piles required.

From the analysis, it is observed that locally there are an insufficient number of piles along the South and West perimeter basement walls of the podium. At these locations the identified wall lines are founded on single lines of piles instead of larger pile caps with multiple rows of piles which are identified for the remainder of the foundation system.

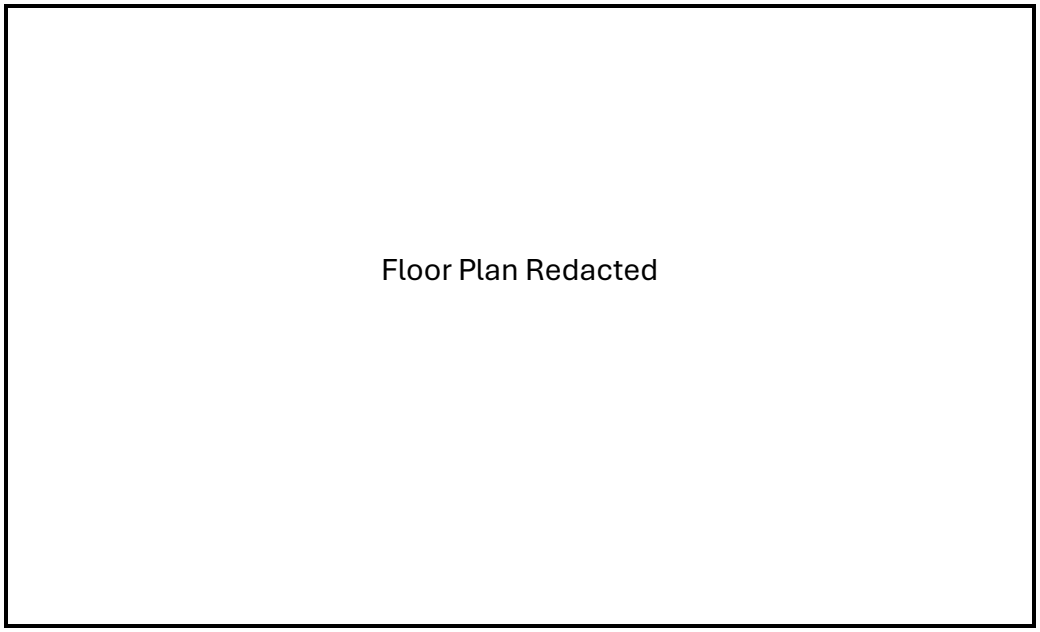


Figure 6. Basement bearing and shear walls are shown in blue.

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Loading Condition	Minimum Factor of Safety	Allowable Load per Pile-tons 12" square	14" square	16" square
Axial Downward-				
Dead Load	3	87	107	133
Dead plus Live Load	2	130	160	200
Total Load, including seismic or wind	1.5	174	214	266
Axial Uplift	2	40	50	60
Lateral	2	7	12	16

Figure 7. Reference pile capacities from record Geotechnical Report.



Figure 8. A deficient number of piles are identified at highlighted footings.

TRANSFER GIRDERS

In reviewing the record drawings, the most significant finding was the frequent use of transfer girders at the second level to support the perimeter moment frames and cell shear walls. These members are critical to the overall performance of the building and must be able to withstand very large forces to provide a sound load path for vertical and lateral forces from the tower. The transfer beams were evaluated under force-controlled actions per Section 7.5.2.2 for shear and flexure.

It is observed that the elements with the highest stress occur at transfer beams directly below the cell dividing walls. Per the record drawings these identified sections have much less reinforcing than the major transfer girders. Of 34 representative transfer girders evaluated, 9 did not meet the acceptance criteria.

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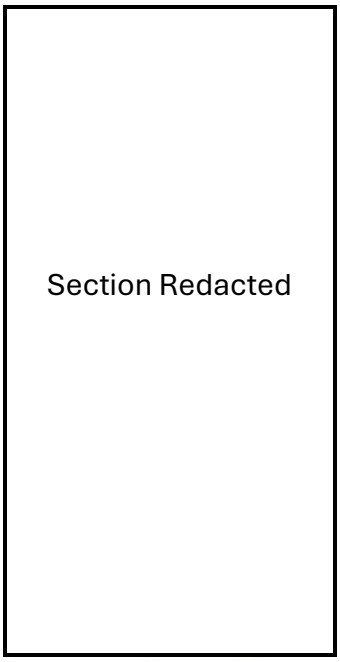


Figure 9. Visualization of transfer girder configuration at second floor. A large transfer girder supports moment frame 'J'. Lateral forces are redirected at level 2 and travel across the diaphragm to nearby shear walls, instead of a direct load path to the foundation. Transfer girders also resolve overturning forces from the frame and support interior beams.

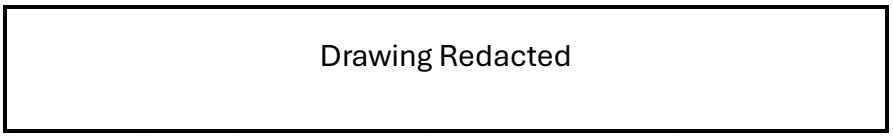


Figure 10. Representative transfer girder ID number and reinforcing from Record Drawings. Typical transfer girder framing at the second level is 7'-0" in depth and varying widths.

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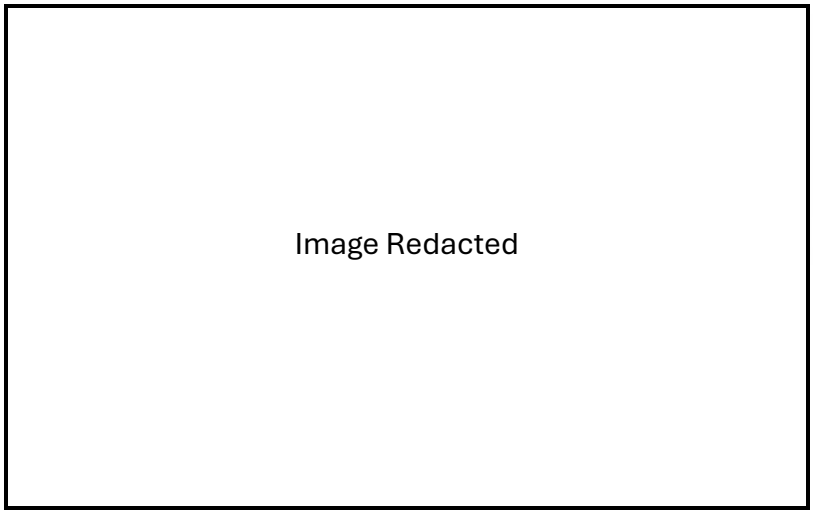


Figure 11. Isometric view of critical transfer girder beams. Orange colored members are spanning from vertical support. Blue components are integrated into bearing walls directly below.

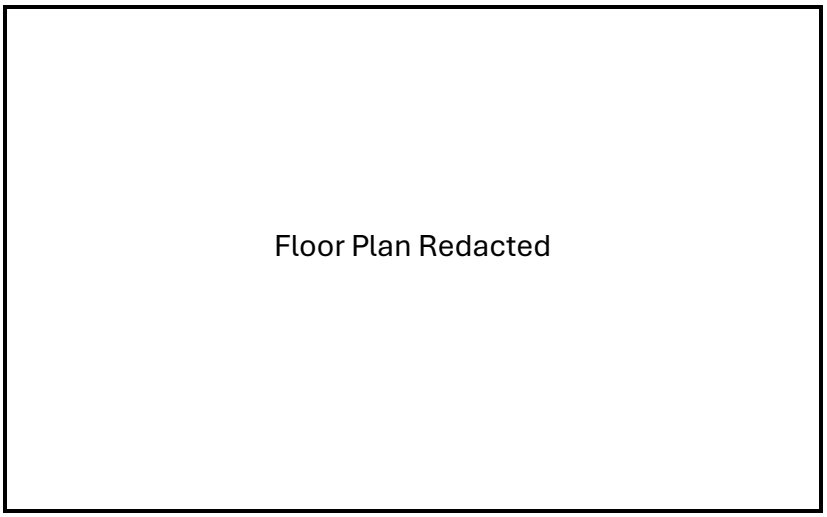


Figure 12. Second level framing plan identifying evaluated transfer girders. Green components meet acceptance criteria; Red components represent shear failures; Orange components represent flexural failures.

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DIAPHRAGM TRANSFER

The diaphragms across the project are evaluated to verify whether there is adequate capacity to transfer forces from the tower to the podium shear walls. This was interpreted graphically through use of the numerical model. The diaphragms were evaluated as a force-controlled action, using the reinforcing as scheduled per the record drawings. It was observed that the reinforcing ratio and capacities of individual diaphragm definitions are very similar and thus taken as an average value of 28 kips/ft. From the shear force diagrams, it was identified that several locations at the second and third levels are deficient. The performance objective of Immediate Occupancy under BSE-1E hazard level is the controlling case for both orthogonal directions. Any regions on the force diagram identified as dark blue are exceeding the available capacity.

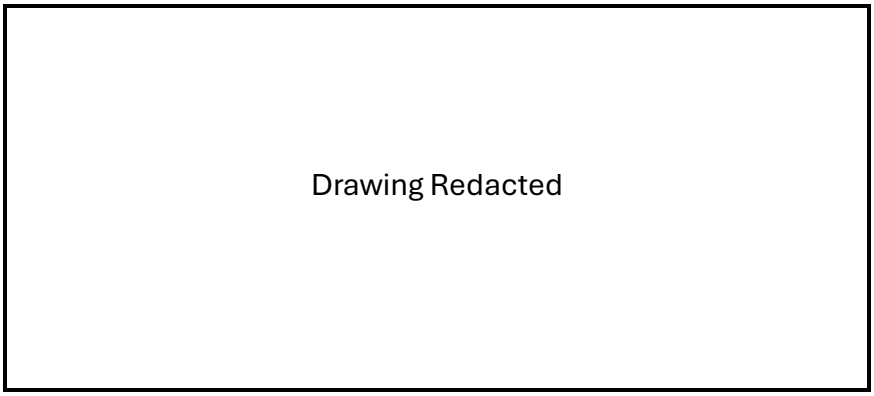


Figure 13. Diaphragm reinforcing schedule per the record drawings (second floor).

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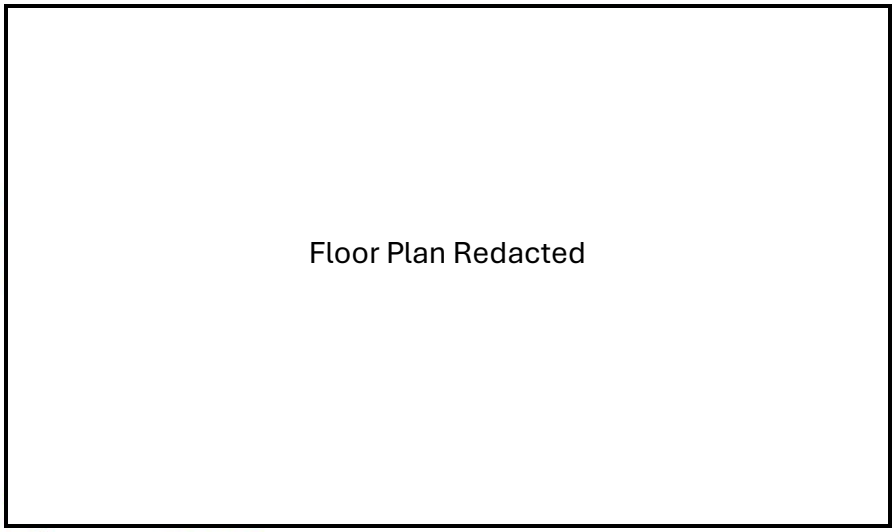


Figure 14. Diaphragm shear forces for BSE-1EX loading at Second floor level. Dark blue represents exceeded capacity of the diaphragm.

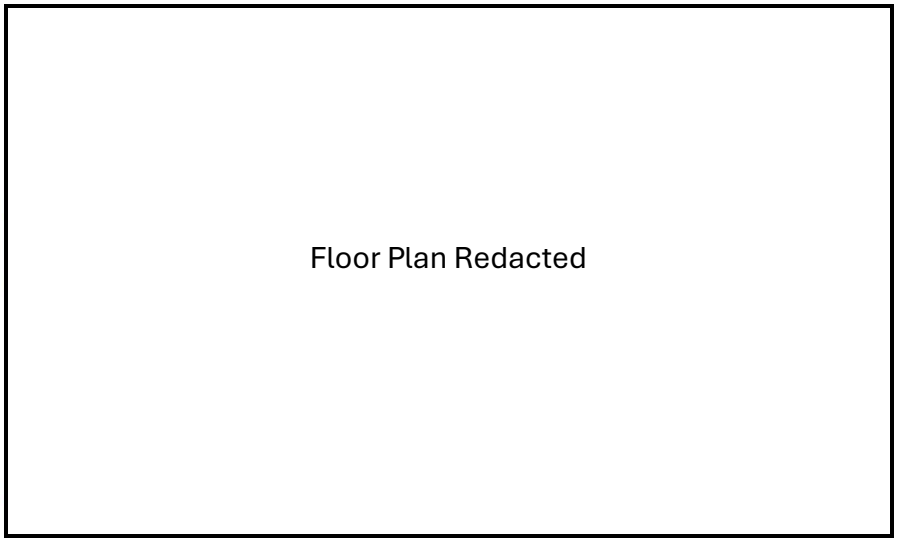


Figure 15. Diaphragm shear forces for BSE-1EY loading at Second floor level. Dark blue represents exceeded capacity of the diaphragm.

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There is also a unique transfer of forces observed between the 3rd Mezzanine and 3rd floor levels. The perimeter wall of the recreation yard acts as a shear wall for a single story with no vertical continuity to the foundation level. There is a small backstay effect at this level. Any regions on the force diagram identified as dark blue are exceeding the available capacity. It can be observed from the shear force diagram for the wall piers along Grid C that shear is reduced from the 3rd Mezzanine Level and Reintroduced and 3rd level.

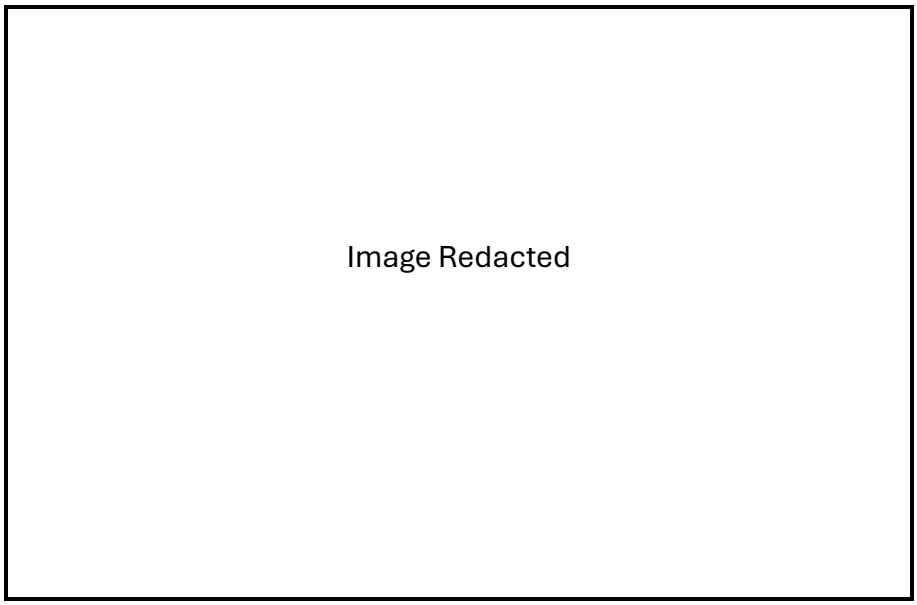


Figure 16. Distribution of story shears along Gridline C for BSE-1EX loading. The reduction in shear force for interior piers adjacent to the recreation level 3M is observed.

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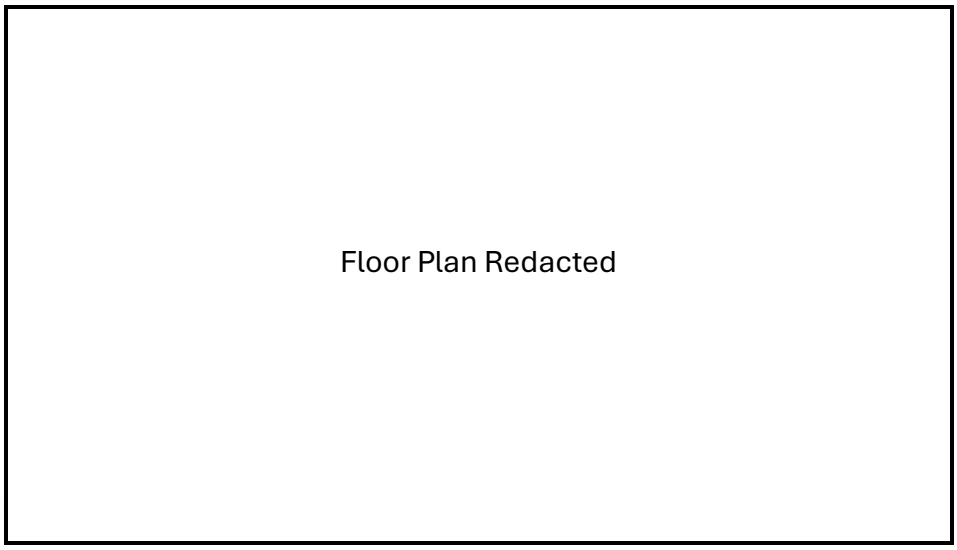


Figure 17. Diaphragm shear forces for BSE-1EX loading at 3M level. Dark blue represents exceed capacity of the diaphragm. Shear forces are attracted to the perimeter wall of the recreation floor and travel to the third level diaphragm below.

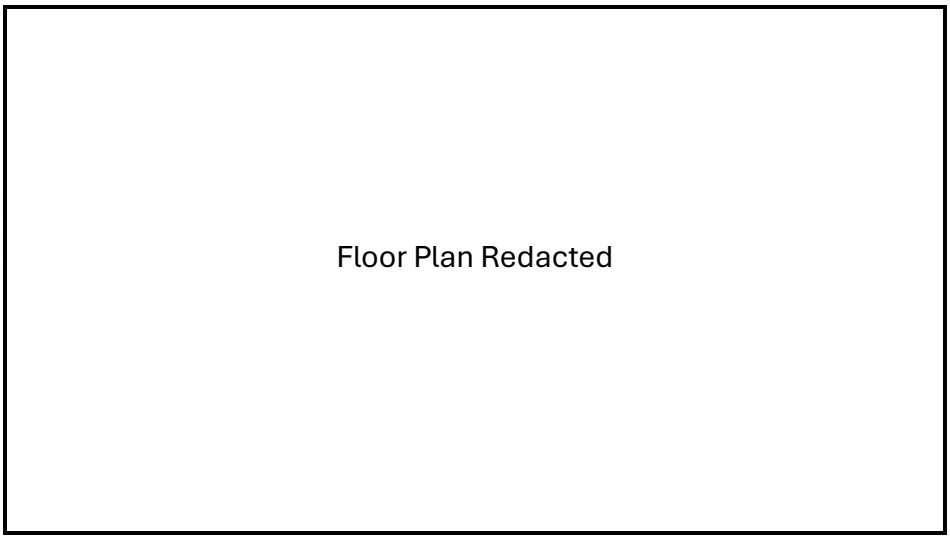


Figure 18. Diaphragm shear forces for BSE-1EX loading at the third-floor level. Dark blue represents exceed capacity of the diaphragm. Shear forces are attracted to the perimeter wall of the recreation floor and travel to the third level diaphragm, and then return to the interior piers along Gridline C.

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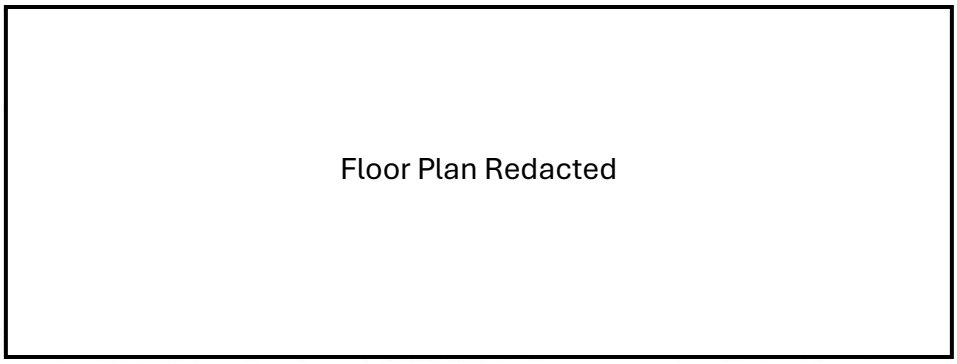


Figure 19. Identified shear walls below the 3M level.

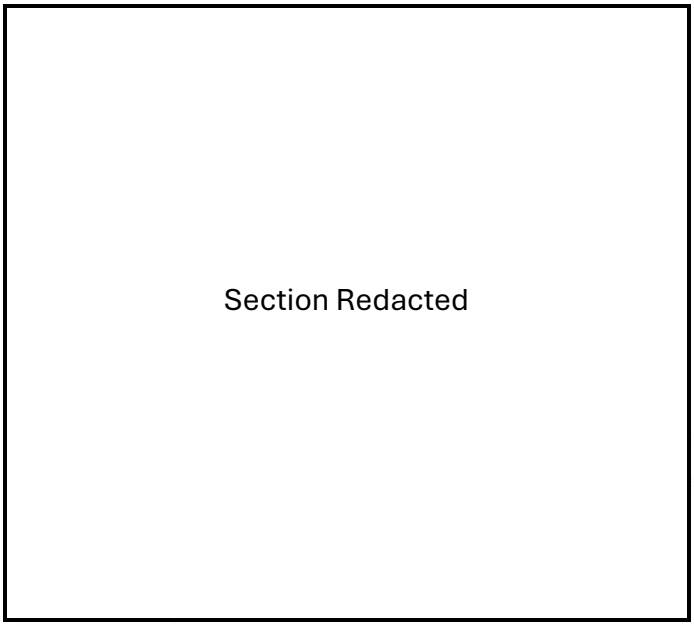


Figure 20. Visualization of shear attraction at the perimeter wall of the 3M recreation floor. Vertical members of the lateral force resisting system are highlighted in blue.

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CONCRETE SHEAR WALLS

Within the tower, full height interior shear walls are identified and can be classified as either primary shear walls or cell dividing walls. The primary shear walls occur about the stairs and elevator cores, and along grid C between the recreation yard and main corridor. The primary walls have a direct load path from foundation to the tower roof level. The cell dividing walls terminate at the second-floor transfer level.

All identified walls are detailed as ductile with confined boundary elements, and the capacity of each wall pier segment was determined based on the reinforcing provided in the record drawings. The focus of the wall analysis is at the transition level between the second and third story. All wall piers are identified as flexure dominated under deformation-controlled action. The m-factor is taken as 2 and 4 for Immediate Occupancy and Life Safety respectively.

It is observed that the plan East-West walls about the end stair cores are attracting more force than expected. At the cell dividing walls, there are a select few that also attract more force than adjacent walls when considering they all share the same strength and stiffness.

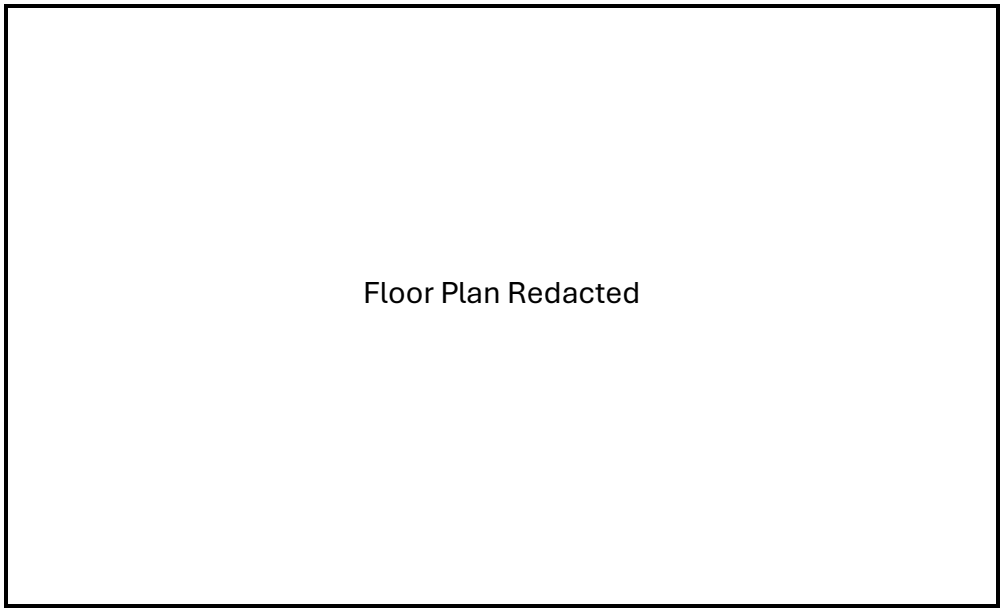


Figure 21. Typical tower plan identifying primary shear walls and cell dividing walls.

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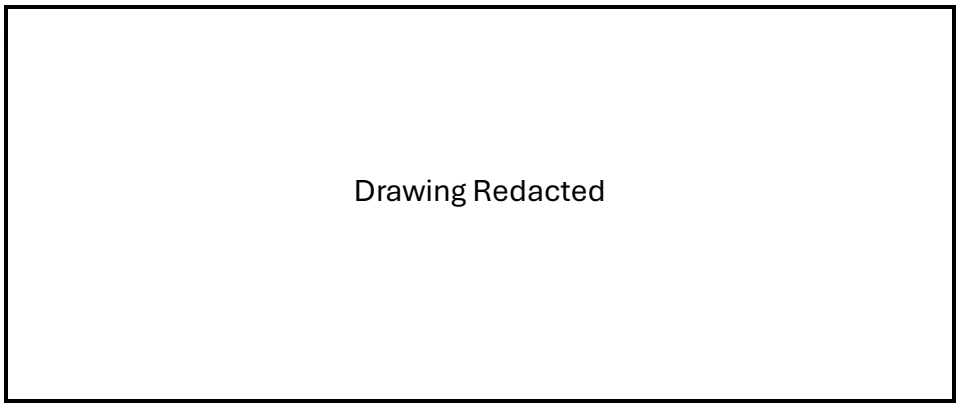


Figure 22. Primary shear wall detailing from record drawings.

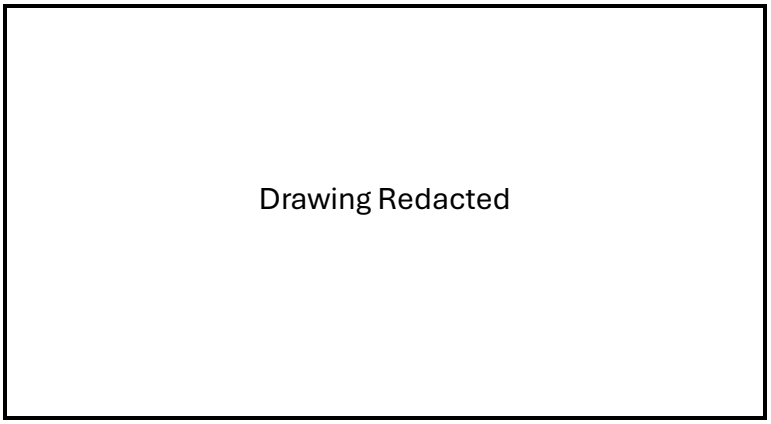


Figure 23. Cell dividing wall detailing from record drawings.

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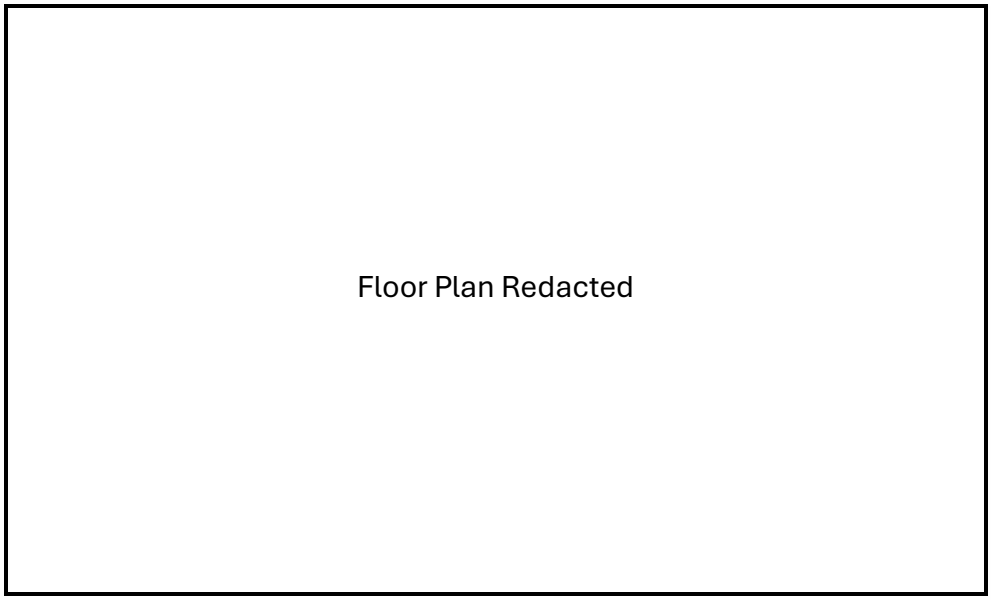


Figure 24. Identified walls that do not meet acceptance criteria. DCR > 1.0 colored red; DCR < 1.0 colored blue.

Table 3. Demand-capacity ratio for shear walls as a percentage of expected component capacity

Component	BSE-1E Hazard	BSE-2E Hazard
Primary	122% (flexure)	115% (flexure)
Cell Walls	108% (flexure)	88% (flexure)

CONCRETE MOMENT FRAMES

The perimeter frames on all exterior sides of each pod are identified as moment frames. From the document review, it was well understood that the members of the moment frame system were detailed as ductile moment frames. Splices of longitudinal column and beam reinforcing were intentionally placed away from joints. Closely spaced ties in the beams, columns, and joints were noted in the drawings. Under deformation-controlled actions for the beams, columns, and joints, the following DCR values are presented. The m-factor for columns is taken as 1.5 and 2.7 for Immediate Occupancy and Life Safety respectively. The m-factor for beams is taken as 3 and 6 for Immediate Occupancy and Life Safety respectively. The m-factor for joints is taken as 1 for all performance objectives.

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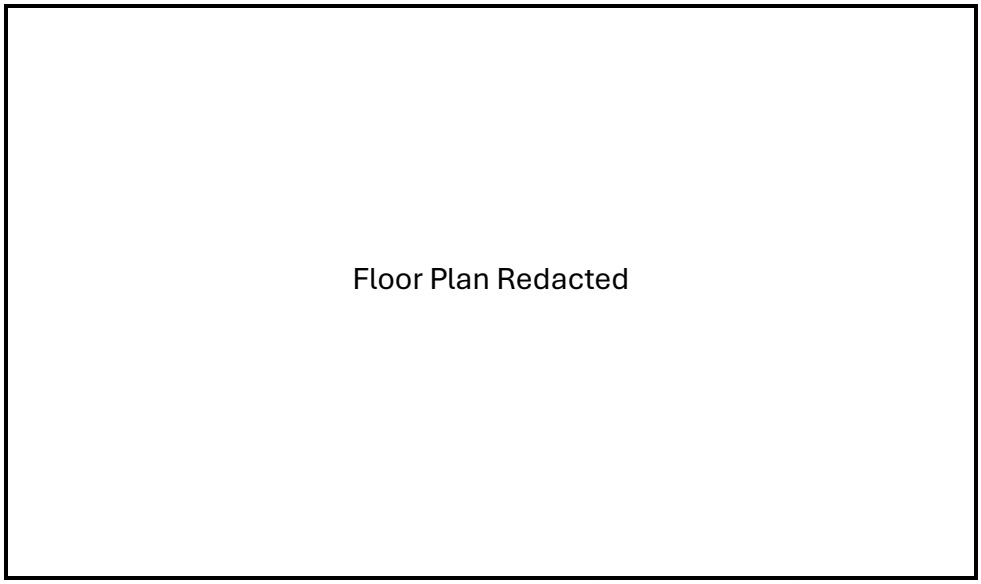


Figure 25. Frame labelling consistent with the record drawings. All moment frames are located about the perimeter elevations of the tower.

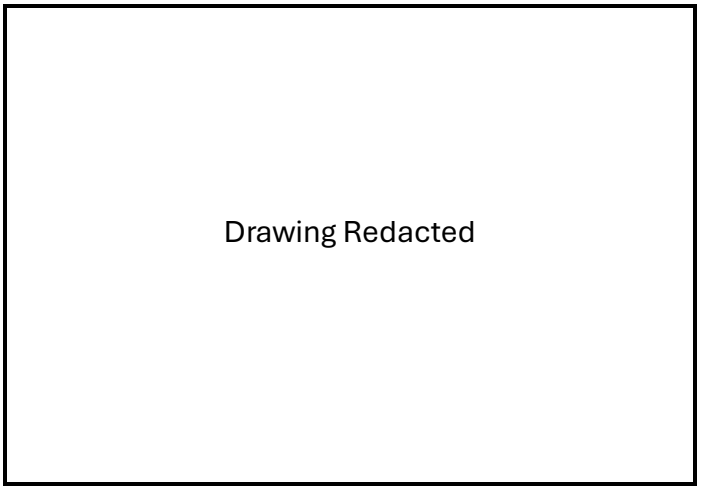


Figure 26. Typical moment frame joint detailing from record drawings.

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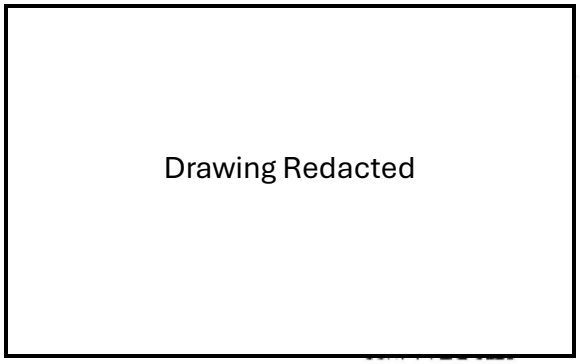


Figure 27. Typical moment frame joint ties from record drawings.

Table 4. Demand-capacity ratio for moment frame components as a percentage of expected component capacity

Component	BSE-1E Hazard	BSE-2E Hazard
Beam	32% (flexure)	28% (flexure)
Column	97% (compression)	76% (compression)
Joint	30% (shear)	50% (shear)

PENTHOUSE STEEL FRAMING

The penthouse lateral system utilizes steel moment frames in both orthogonal directions. It was observed that the sizes of the steel beams and columns were exceptionally large. It was understood by interpretation that during design of this penthouse, it was considered one of the building levels as opposed to the current code provisions which allow it to be considered an appendage. This penthouse was not a primary focus of the evaluation and therefore not formally evaluated. It was observed from the record drawings that typical detailing of steel moment frames was utilized.

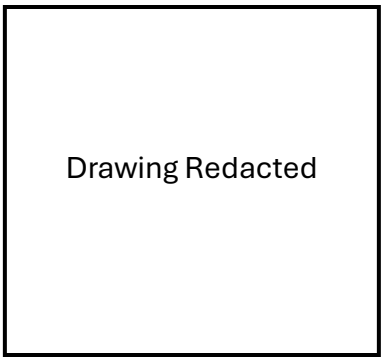


Figure 28. Typical steel moment frame detailing at penthouse from record drawings.

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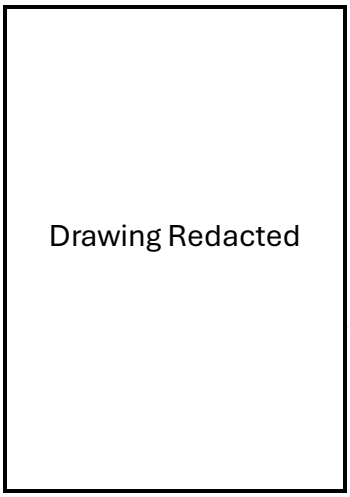


Figure 29. Typical steel moment frame column anchorage at penthouse from record drawings.

C. CONCLUSIONS AND RECOMMENDATIONS

OVERALL BUILDING PERFORMANCE:
This building was evaluated holistically to understand the expected performance of the structure subject to the seismic loading and performance criteria set forth in ASCE 41-17. Representative components of the lateral force resisting system and other members critical to the performance of the building were analyzed to best capture overall performance of the building. The original design of this building per UBC 1982 did not classify this structure as an essential facility. As expected, this structure does not meet the Immediate Occupancy performance objective that would be required by today's standards. By extrapolation it is understood that this will likely meet a Life Safety performance objective for a BSE-1E hazard level with minimal remediation. If the structure were to undergo a design level earthquake, it is expected that the structure will experience downtime before becoming fully operational again. If the building is required to meet the Immediate Occupancy performance objective, retrofit of the structure is required. It is also understood that a partial retrofit at localized components of the building is not applicable as deficiencies are identified across the entire building. The structural systems that were evaluated and identified as deficient are further discussed in this section, and conceptual retrofit approaches are presented.

TOWER:
The lateral system of the tower is very redundant when considering the total number of moment frames and shear walls. Although several of these components approach or exceed capacity, it is expected that forces would redistribute and find alternate load justifiable paths to the transfer level. It is recommended that performance of the tower be justified through a more refined analysis. A nonlinear analysis will demonstrate the redistribution of forces once a particular component reaches maximum capacity.

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TRANSFER LEVEL:
Several transfer girders and transfer diaphragms were identified to be insufficient in resolving forces from the tower to the shear walls below. A comprehensive analysis is required to evaluate all remaining transfer girders not included in this study. With this, all identified members determined to exceed capacity shall be strengthened to provide an adequate load path. Strengthening beams with fiber reinforced polymer (FRP) is the recommended approach.

FOUNDATION:
Considering the buildings' excellent performance with regards to settlement, there is no concern for vertical capacity of the piles. It can be observed however that the foundation system lacks discrete ties to keep floating pile caps from moving independently. At the identified locations where an insufficient number of piles is observed, it is recommended that new mat slab or grade beam foundation elements be constructed to effectively engage a sufficient number of existing lateral resisting piles.

RECOMMENDED ADDITIONAL STUDIES:
It is recommended that the county proceeds with a Tier 3 Systematic Evaluation and Retrofit procedure. A Tier 3 analysis per ASCE 41 extensively evaluates all components of the structure with retrofit solutions implemented such that the building meets the performance objective. With this, there is an opportunity to refine the analysis model to completely capture all conditions and include interaction with the site soil characteristics through the use of a nonlinear procedure.

A current geotechnical investigation shall also be completed to validate the subsurface assumptions using the latest adopted provisions as well as confirm the potential for liquefaction at this site.

V. TENANT IMPROVEMENT ALTERNATIVES

The following opportunities have been identified as considerations for tenant improvement scopes and were developed conceptually to understand the structural implications of these new improvements on the existing structure.

At this time, it is understood that the proposed alternatives do not have substantial impact to the overall seismic performance of the building. Should any of these alternatives be selected for full design and construction, the following observations and assumptions shall be considered. For each alternative, a proposed structural concept, and description of the extent of structural modification are presented.

A. ACCESSIBILITY IMPROVEMENTS

The accessibility improvements proposed include adding accessible cells with showers, interview spaces, medical spaces, non-contact visiting room, and holding cells by the courthouse. These changes would not alter the seismic resisting system nor add substantial mass that would affect the structural performance. It is understood that there would be localized structural modification as required for anchorage and attachment of nonstructural components.

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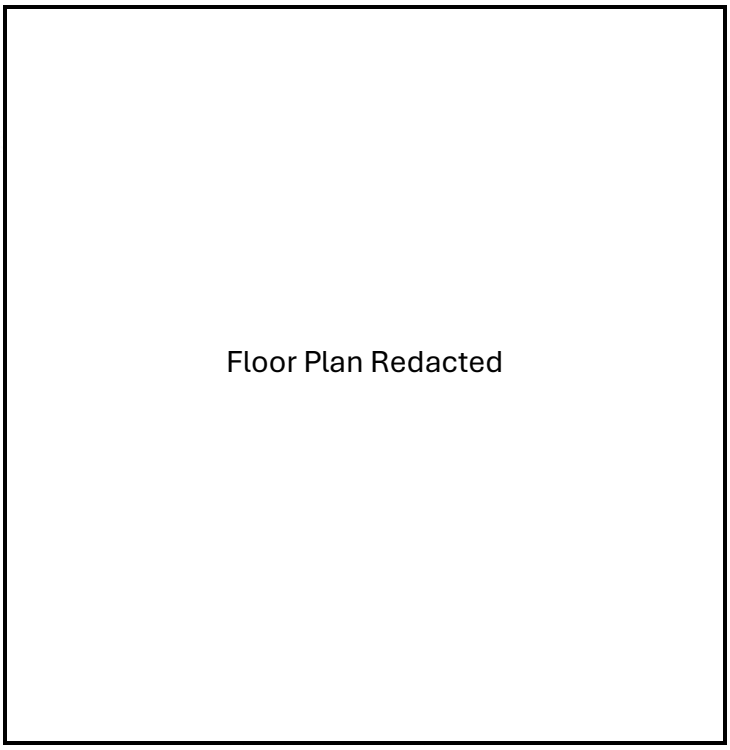


Figure 30: Compiled figure of proposed accessibility improvements.

B. ENHANCE UTILIZATION

To enhance utilization of housing space, the proposed improvement is to modify cells within housing pods on each typical floor. This is expected to have minimal structural impact and does not affect the seismic lateral resisting system nor add significant seismic mass. Identified structural dividing walls must remain. See figure below of proposed plan.

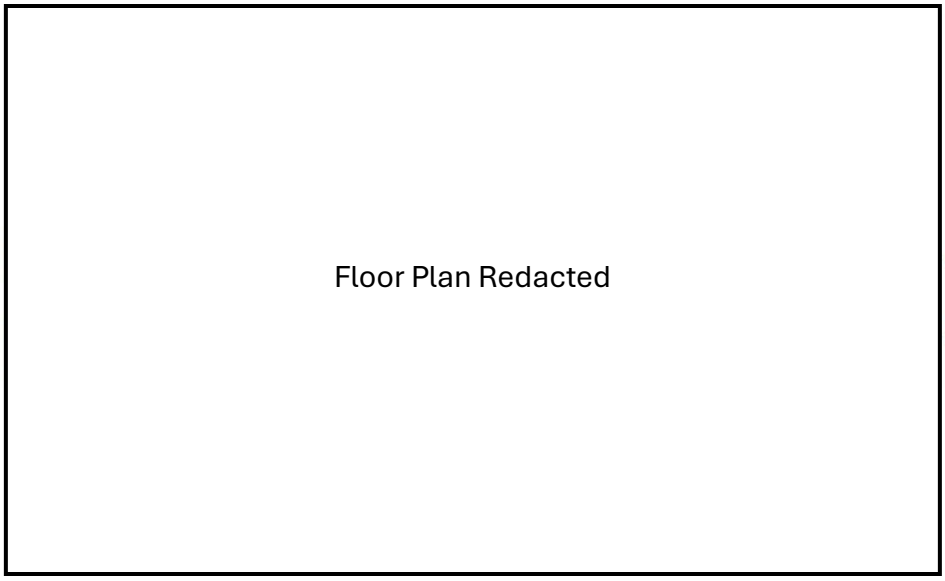


Figure 31: Proposed plan for new divided housing pods with locations of partitions per floor.

C. INCREASE PROGRAM, SUPPORT AND OFFICE SPACES

The proposed conversion of a cell pod to a new program pod that supports medical, staff, and programming space for inmates has minimal structural impact. Structural walls would remain intact with no modifications with the focus on design of interior spaces and altering partition walls. See the figure below.

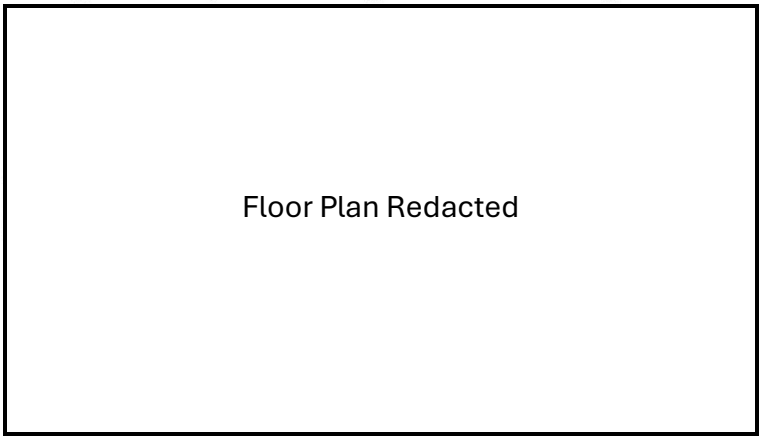


Figure 32: Proposed pod change to support increased programming and staff spaces.

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D. INCREASE ACCESS TO RECREATION

The mezzanine-level recreation areas currently serve multiple housing pods across multiple levels. To improve access, security, and operational flexibility, the proposed improvement consists of dividing the existing recreation space into separate areas. The proposed layout is illustrated in the floor plan below.

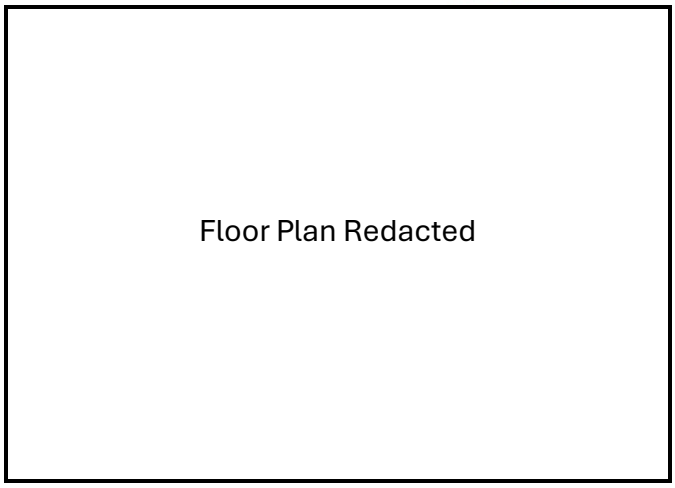


Figure 33: Mezzanine floor plan showing the proposed division of the recreation space and the location of a typical structural section.

The proposed partition system would have a minimal impact on the existing structure and would not affect the building's primary gravity or lateral force-resisting systems. As a result, only minor localized structural modifications are anticipated to facilitate installation while minimizing disruption to facility operations.

The partition would consist of structural steel framing with metal decking to create a secure barrier between recreation areas and prevent inmate interaction across the partition. The upper portion of the partition would incorporate security mesh panels to maintain the flow of light and ventilation throughout the space while preserving visibility for facility staff. The metal deck and mesh panels would be supported by a framework of vertical and horizontal steel tube members.

The vertical steel tubes would be laterally braced by connections to the existing beam above but would not be suspended from the existing structure. This approach limits the introduction of additional gravity loads into the overhead framing while providing the lateral stability necessary for the partition system. Gravity loads from the partition would be transferred directly to the structure below through vertical steel members anchored to the existing reinforced concrete beam and a new concrete curb. To support the partition loads, localized improvements to the existing concrete floor framing may be required. These improvements would provide an adequate load path for gravity forces while maintaining the integrity of the existing structural system. A typical partition section and connection detail are shown in the figures below.

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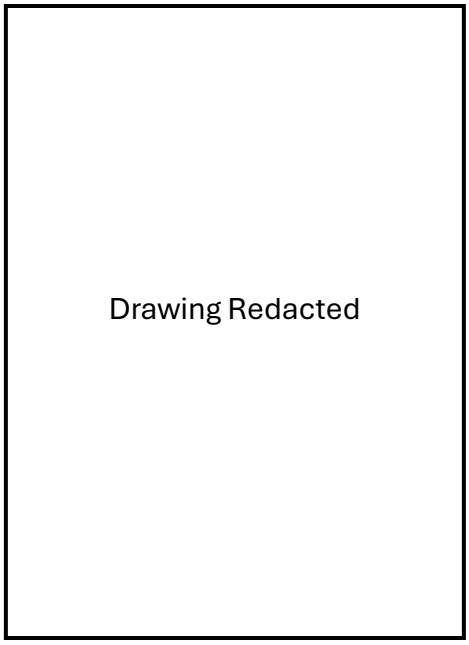


Figure 34: Partition section showing the metal deck spanning between steel tubes with gravity support from existing beam below and lateral support from the existing beam above.

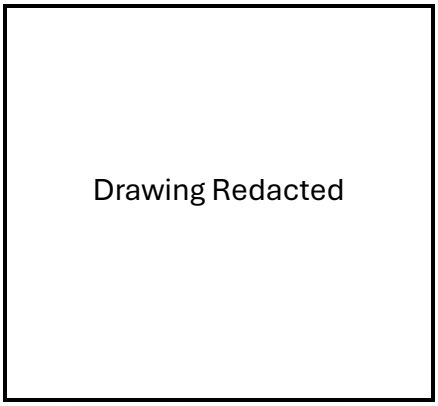


Figure 35: Detail highlighting new partition construction to new concrete curb attached to the existing concrete beam.

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E. IMPROVE VERTICAL CIRCULATION

Two proposed alternatives were evaluated to improve vertical circulation within the facility. The first alternative consists of constructing an additional elevator bay to accommodate two new elevators. The second alternative involves constructing bridge connections on the 2nd Floor, 3rd Floor, 3rd Floor Mezzanine, and 5th Floor levels to provide access between the Main Jail and a proposed Annex Building.

The addition of a new elevator bay for two elevators would require the construction of new vertical and lateral-force-resisting elements. The proposed structure would include a reinforced concrete masonry wall serving as both a shear wall and bearing wall extending the full height of the shaft. Steel framing members would support the sides of the elevator shaft and transfer loads to the existing structure through connections to existing concrete walls and beams.

Because the proposed elevator bay would not align directly over existing foundation elements, new foundation construction would be required. This would likely consist of extending existing grade beams and constructing a new shallow foundation system to support the proposed masonry shear and bearing wall. The new masonry wall would bear on a reinforced concrete grade beam that transfers loads into the extended foundation system.

The elevator addition is expected to have a relatively minor impact on the overall seismic demand of the existing structure due to its limited size and mass. However, localized strengthening of existing gravity-load-carrying elements may be required to accommodate the additional loads introduced by the elevator framing and associated structural components.

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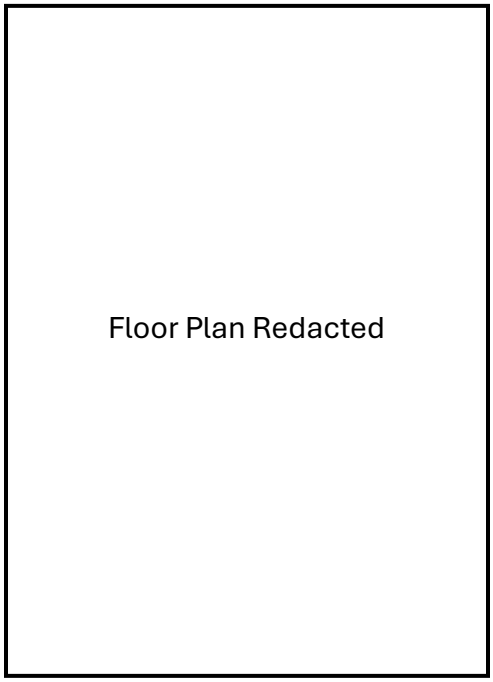


Figure 36: The location of the added structural elements to support additional vertical circulation.

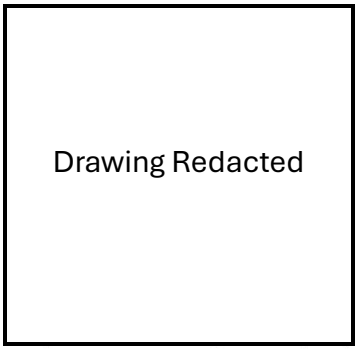


Figure 37: Proposed foundation changes using the record drawing as a reference.

The second vertical circulation improvement option, which includes multiple bridge structures, is more involved structural. The proposed connection would consist of a truss bridge spanning between the existing jail and annex buildings. The bridge would be seismically tied to the existing structures while being gravity-supported at both ends. Implementation of this option would require modifications to the existing gravity and lateral force-resisting systems to accommodate the additional loads introduced by the bridge. Structural upgrades may include strengthening existing framing elements, adding new columns or shear walls, and

potentially improving foundations to support the increased dead load and seismic demands associated with the truss bridge span. These modifications would be necessary to ensure adequate load transfer and overall structural performance. It is significant to note that the bridges must be seismically separated on one end to allow for differential movement between two buildings. The preferred option would be to include a seismic expansion joint within the interface between the new building. The proposed configuration is shown below.

Section Redacted

Figure 38: The proposed steel truss bridge connecting multiple floors.

Sincerely,

Patrick Crosby, PE, SE, F. DBIA
Principal

Matt Roloff, PE
Professional Engineer

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Prepared at the Direction of Counsel

APPENDIX A: CHECKLISTS

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ASCE 41-17 Tier 1 Checklists

FIRM:	Martin/Martin
PROJECT NAME:	Sacramento Jail Evaluation
SEISMICITY LEVEL:	High
PROJECT NUMBER:	26.0417.S.01
COMPLETED BY:	M.R.
DATE COMPLETED:	6.6.2026
REVIEWED BY:	M.P.
REVIEW DATE:	6.6.2026

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

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Project Name _____
Project Number _____

17.1.2IO Basic Configuration Checklist

Table 17-3. Immediate Occupancy Basic Configuration Checklist

Status				Evaluation Statement	Tier 2 Reference	Commentary Reference	Comments
Very Low Seismicity							
Building System—General							
C ☐	**NC** ☒	**N/A** ☐	**U** ☐	LOAD PATH: The structure contains a complete, well-defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation.	5.4.1.1	A.2.1.1	Transfer girders occur at the second floor at the transition from moment frames and shear wall system to a shear wall-only system. Further evaluation of the transfer girders is required.
C ☐	**NC** ☐	**N/A** ☒	**U** ☐	ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building is greater than 0.5% of the height of the shorter building in low seismicity, 1.0% in moderate seismicity, and 3.0% in high seismicity.	5.4.1.2	A.2.1.2	No adjacent buildings near by.
C ☒	**NC** ☐	**N/A** ☐	**U** ☐	MEZZANINES: Interior mezzanine levels are braced independently from the main structure or are anchored to the seismic-force-resisting elements of the main structure.	5.4.1.3	A.2.1.3	Mezzanines are positively attached at moment frames and/or shear walls.
Building System—Building Configuration							
C ☒	**NC** ☐	**N/A** ☐	**U** ☐	WEAK STORY: The sum of the shear strengths of the seismic-force-resisting system in any story in each direction is not less than 80% of the strength in the adjacent story above.	5.4.2.1	A.2.2.2	No weak story identified.
C ☒	**NC** ☐	**N/A** ☐	**U** ☐	SOFT STORY: The stiffness of the seismic-force-resisting system in any story is not less than 70% of the seismic-force-resisting system stiffness in an adjacent story above or less than 80% of the average seismic-force-resisting system stiffness of the three stories above.	5.4.2.2	A.2.2.3	No soft story identified.
C ☐	**NC** ☒	**N/A** ☐	**U** ☐	VERTICAL IRREGULARITIES: All vertical elements in the seismic-force-resisting system are continuous to the foundation.	5.4.2.3	A.2.2.4	RE: Load Path; Transfer girders occur at the second floor at the transition from moment frames and shear wall system to a shear wall-only system.

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

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C	NC	N/A	U				
☐	☒	☐	☐	GEOMETRY: There are no changes in the net horizontal dimension of the seismic-force-resisting system of more than 30% in a story relative to adjacent stories, excluding one-story penthouses and mezzanines.	5.4.2.4	A.2.2.5	RE: Weak Story and Soft Story.
☐	☒	☐	☐	MASS: There is no change in effective mass of more than 50% from one story to the next. Light roofs, penthouses, and mezzanines need not be considered.	5.4.2.5	A.2.2.6	It was determined that there is a significant difference in effective mass between the lower levels.
☒	☐	☐	☐	TORSION: The estimated distance between the story center of mass and the story center of rigidity is less than 20% of the building width in either plan dimension.	5.4.2.6	A.2.2.7	It was determined that the computed center of mass and story center of rigidity is within the limit of 20%. in both orthogonal directions.

Status				Evaluation Statement	Tier 2 Reference	Commentary Reference	Comments
Low Seismicity (Complete the Following Items in Addition to the Items for Very Low Seismicity)							
Geologic Site Hazards							
C	NC	N/A	U				
☐	☒	☐	☐	LIQUEFACTION: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance do not exist in the foundation soils at depths within 50 ft (15.2 m) under the building.	5.4.3.1	A.6.1.1	This building is located within a liquefaction zone per the California Geological Survey.
☒	☐	☐	☐	SLOPE FAILURE: The building site is located away from potential earthquake-induced slope failures or rockfalls so that it is unaffected by such failures or is capable of accommodating any predicted movements without failure.	5.4.3.1	A.6.1.2	No slope failure concern.
☒	☐	☐	☐	SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site are not anticipated.	5.4.3.1	A.6.1.3	Does not appear to be a concern based on the California Geological Survey Fault Activity Map.

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

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Project Number _____

Status				Evaluation Statement	Tier 2 Reference	Commentary Reference	Comments
Moderate and High Seismicity (Complete the Following Items in Addition to the Items for Low Seismicity)							
Foundation Configuration							
C ☐	**NC** ☒	**N/A** ☐	**U** ☐	OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$.	5.4.3.3	A.6.2.1	Not met by geometry of frames and shear walls.
C ☐	**NC** ☒	**N/A** ☐	**U** ☐	TIES BETWEEN FOUNDATION ELEMENTS: The foundation has ties adequate to resist seismic forces where footings, piles, and piers are not restrained by beams, slabs, or soils classified as Site Class A, B, or C.	5.4.3.4	A.6.2.2	Pile caps are not positively attached to the slab and there is no use of foundation ties.

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

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ASCE 41-17 Tier 1 Checklists

FIRM:	Martin/Martin
PROJECT NAME:	Sacramento Jail Evaluation
SEISMICITY LEVEL:	High
PROJECT NUMBER:	26.0417.S.01
COMPLETED BY:	M.R.
DATE COMPLETED:	6.6.2026
REVIEWED BY:	M.P.
REVIEW DATE:	6.6.2026

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

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17.11IO Structural Checklist for Building Type C1: Concrete Moment Frames

Table 17-23. Immediate Occupancy Structural Checklist for Building Type C1

Status				Evaluation Statement	Tier 2 Reference	Commentary Reference	Comments
Very 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 3.	5.5.1.1	A.3.1.1.1	There is an adequate number of frames in each direction. However, frame B and Frame X have less than 3 bays.
C ☐	NC ☐	N/A ☒	U ☐	INTERFERING WALLS: All concrete and masonry infill walls placed in moment frames are isolated from structural elements.	5.5.2.1.1	A.3.1.2.1	No infill walls inside of moment frames.
C ☒	NC ☐	N/A ☐	U ☐	COLUMN SHEAR STRESS CHECK: The shear stress in the concrete columns, calculated using the Quick Check procedure of Section 4.4.3.2, is less than the greater of 100 lb/in.^{2} (0.69 MPa) or $2\sqrt{f_c'}$.	5.5.2.1.4	A.3.1.4.1	Through the quick check, this is satisfied.
C ☒	NC ☐	N/A ☐	U ☐	COLUMN AXIAL STRESS CHECK: The axial stress caused by unfactored gravity loads in columns subjected to overturning demands is less than $0.13f_c'$. Alternatively, the axial stress caused by overturning forces alone, calculated using the Quick Check procedure of Section 4.4.3.6, is less than $0.30f_c'$.	5.5.2.1.3	A.3.1.4.2	Through the quick check, this is satisfied for the alternative analysis of 4.4.3.6.
Connections							
C ☐	NC ☒	N/A ☐	U ☐	CONCRETE COLUMNS: All concrete columns are doweled into the foundation, and the dowels are able to develop the tensile capacity of reinforcement in columns of the seismic-force-resisting system.	5.7.3.1	A.5.3.2	Concrete columns of moment frames do not continue to the foundations.

Status				Evaluation Statement	Tier 2 Reference	Commentary Reference	Comments
Low and Moderate Seismicity (Complete the Following Items in Addition to the Items for Very Low Seismicity)							
Seismic-Force-Resisting System							
C ☐	NC ☐	N/A ☒	U ☐	FLAT SLAB FRAMES: The seismic-force-resisting system is not a frame consisting of columns and a flat slab or plate without beams.	5.5.2.3.1	A.3.1.4.3	There are no flat slab frames in this building.

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

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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} (4.83 MPa) or $f_c'/6$ at potential hinge locations. The average prestress is calculated in accordance with the Quick Check procedure of Section 4.4.3.8.	5.5.2.3.2	A.3.1.4.4	There are no prestressed frame elements in this building.
C	NC	N/A	U				
☒	☐	☐	☐	CAPTIVE COLUMNS: There are no columns at a level with height/depth ratios less than 75% of the nominal height/depth ratio of the typical columns at that level.	5.5.2.3.3	A.3.1.4.5	All moment frame columns match in dimension with very few exceptions.
C	NC	N/A	U				
☐	☐	☐	☒	NO SHEAR FAILURES: The shear capacity of frame members is able to develop the moment capacity at the ends of the members.	5.5.2.3.4	A.3.1.4.6	Further analysis is required to observe actual behavior of frames.
C	NC	N/A	U				
☐	☒	☐	☐	STRONG COLUMN—WEAK BEAM: The sum of the moment capacity of the columns is 20% greater than that of the beams at frame joints.	5.5.2.1.5	A.3.1.4.7	This not met based on sectional analysis.
C	NC	N/A	U				
☒	☐	☐	☐	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.	5.5.2.3.5	A.3.1.4.8	All beam bars continue through the joint.
C	NC	N/A	U				
☒	☐	☐	☐	COLUMN-BAR SPLICES: All column-bar lap splice lengths are greater than $50d_b$ 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.	5.5.2.3.6	A.3.1.4.9	Welded or mechanical splices are used per drawings but unknown in as-built condition. Capacity is also unknown but assumed compliant.
C	NC	N/A	U				
☒	☐	☐	☐	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.	5.5.2.3.6	A.3.1.4.10	Beam splices are noted to occur within middle of bay.
C	NC	N/A	U				
☒	☐	☐	☐	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.	5.5.2.3.7	A.3.1.4.11	This is observed per typical frame details on 604_3-8B-3

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C	NC	N/A	U				
☒	☐	☐	☐	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 $d/4$.	5.5.2.3.7	A.3.1.4.12	This is observed per typical frame details on 604_3-8B-2
☒	☐	☐	☐	JOINT TRANSVERSE REINFORCING: Beam–column joints have ties spaced at or less than $8d_b$.	5.5.2.3.8	A.3.1.4.13	This is observed per typical frame details on 604_3-8B-3
☒	☐	☐	☐	JOINT ECCENTRICITY: There are no eccentricities larger than 20% of the smallest column plan dimension between girder and column centerlines.	5.5.2.3.9	A.3.1.4.14	Typically they are concentric and beam width matches column depth.
☒	☐	☐	☐	STIRRUP AND TIE HOOKS: The beam stirrups and column ties are anchored into the member cores with hooks of 135 degrees or more.	5.5.2.3.10	A.3.1.4.15	Compliant per typical frame details
☐	☐	☒	☐	DEFLECTION COMPATIBILITY: Secondary components have the shear capacity to develop the flexural strength of the components and are Compliant with the following items in this table: COLUMN-BAR SPLICES, BEAM-BAR SPLICES, COLUMN-TIE SPACING, STIRRUP SPACING, and STIRRUP AND TIE HOOKS.	5.5.2.5.2	A.3.1.6.2	There are no frames not part of the seismic force resisting system.
☒	☐	☐	☐	FLAT SLABS: Flat slabs or plates not part of the seismic-force-resisting system have continuous bottom steel through the column joints.	5.5.2.5.3	A.3.1.6.3	Flat slab bars pass through moment frame beams and column joints.
Diaphragms							
☒	☐	☐	☐	DIAPHRAGM CONTINUITY: The diaphragms are not composed of split-level floors and do not have expansion joints.	5.6.1.1	A.4.1.1	This is observed per the record drawings.
☒	☐	☐	☐	PLAN IRREGULARITIES: There is tensile capacity to develop the strength of the diaphragm at reentrant corners or other locations of plan irregularities.	5.6.1.4	A.4.1.7	Additional bars are added at reentrant corners.
☐	☐	☒	☐	DIAPHRAGM REINFORCEMENT AT OPENINGS: There is reinforcing around all diaphragm openings larger than 50% of the building width in either major plan dimension.	5.6.1.5	A.4.1.8	No openings larger than 50% of the building width in either direction.
Connections							
☐	☒	☐	☐	UPLIFT AT PILE CAPS: Pile caps have top reinforcement, and piles are anchored to the pile caps; the pile cap reinforcement and pile anchorage are able to develop the tensile capacity of the piles.	5.7.3.5	A.5.3.8	Pile caps have top reinforcement and piles are anchored to the pile caps. Further analysis is required to determine whether uplift occurs.

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

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ASCE 41-17 Tier 1 Checklists

FIRM:	Martin/Martin
PROJECT NAME:	Sacramento Jail Evaluation
SEISMICITY LEVEL:	High
PROJECT NUMBER:	26.0417.S.01
COMPLETED BY:	M.R.
DATE COMPLETED:	6.6.2026
REVIEWED BY:	M.P.
REVIEW DATE:	6.6.2026

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Project Name _____
Project Number _____

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

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

Status				Evaluation Statement	Tier 2 Reference	Commentary Reference	Comments
Very Low Seismicity							
Seismic-Force-Resisting System							
C ☐	NC ☒	N/A ☐	U ☐	COMPLETE FRAMES: Steel or concrete frames classified as secondary components form a complete vertical-load-carrying system.	5.5.2.5.1	A.3.1.6.1	Many of the shear walls also act as gravity supporting walls.
C ☒	NC ☐	N/A ☐	U ☐	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2.	5.5.1.1	A.3.2.1.1	This is compliant.
C ☒	NC ☐	N/A ☐	U ☐	SHEAR STRESS CHECK: The shear stress in the concrete shear walls, calculated using the Quick Check procedure of Section 4.4.3.3, is less than the greater of 100 lb/in.^{2} (0.69 MPa) or $2\sqrt{f_c}$.	5.5.3.1.1	A.3.2.2.1	From the quick check, this is satisfied.
C ☒	NC ☐	N/A ☐	U ☐	REINFORCING STEEL: The ratio of reinforcing steel area to gross concrete area is not less than 0.0012 in the vertical direction and 0.0020 in the horizontal direction. The spacing of reinforcing steel is equal to or less than 18 in. (457 mm).	5.5.3.1.3	A.3.2.2.2	This is compliant per the reinforcing noted on typical wall sections and elevations.
Connections							
C ☐	NC ☐	N/A ☒	U ☐	WALL ANCHORAGE AT FLEXIBLE DIAPHRAGMS: Exterior concrete or masonry walls that are dependent on flexible diaphragms for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections have strength to resist the connection force calculated in the Quick Check procedure of Section 4.4.3.7.	5.7.1.1	A.5.1.1	No major flexible diaphragms present for this building.
C ☒	NC ☐	N/A ☐	U ☐	TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of loads to the shear walls, and the connections are able to develop the lesser of the shear strength of the walls or diaphragms.	5.7.2	A.5.2.1	Diaphragms are connected to shear walls.

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

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Status				Evaluation Statement	Tier 2 Reference	Commentary Reference	Comments
C ☒	NC ☐	N/A ☐	U ☐	FOUNDATION DOWELS: Wall reinforcement is doweled into the foundation, and the dowels are able to develop the lesser of the strength of the walls or the uplift capacity of the foundation.	5.7.3.4	A.5.3.5	Walls are doweled to wall footings.
Foundation System							
C ☐	NC ☐	N/A ☐	U ☒	DEEP FOUNDATIONS: Piles and piers are capable of transferring the lateral forces between the structure and the soil.		A.6.2.3	This requires further analysis.
C ☒	NC ☐	N/A ☐	U ☐	SLOPING SITES: The difference in foundation embedment depth from one side of the building to another does not exceed one story.		A.6.2.4	Site slope does not exceed limits.
Low, Moderate, and High Seismicity (Complete the Following Items in Addition to the Items for Very Low Seismicity)							
Seismic-Force-Resisting System							
C ☒	NC ☐	N/A ☐	U ☐	DEFLECTION COMPATIBILITY: Secondary components have the shear capacity to develop the flexural strength of the components and are compliant with the following items in Table 17-23: COLUMN-BAR SPLICES, BEAM-BAR SPLICES, COLUMN-TIE SPACING, STIRRUP SPACING, and STIRRUP AND TIE HOOKS.	5.5.2.5.2	A.3.1.6.2	This appears to be met based on review of record drawings.
C ☒	NC ☐	N/A ☐	U ☐	FLAT SLABS: Flat slabs or plates not part of seismic-force-resisting system have continuous bottom steel through the column joints.	5.5.2.5.3	A.3.1.6.3	This is compliant.
C ☐	NC ☐	N/A ☒	U ☐	COUPLING BEAMS: The ends of both walls to which the coupling beam is attached are supported at each end to resist vertical loads caused by overturning. Coupling beams have the capacity in shear to develop the uplift capacity of the adjacent wall.	5.5.3.2.1	A.3.2.2.3	There are no coupling beams in this building.
C ☒	NC ☐	N/A ☐	U ☐	OVERTURNING: All shear walls have aspect ratios less than 4-to-1. Wall piers need not be considered.	5.5.3.1.4	A.3.2.2.4	Compliant per wall elevations 604_3-6-6.

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C	NC	N/A	U				
☒	☐	☐	☐	CONFINEMENT REINFORCING: For shear walls with aspect ratios greater than 2-to-1, the boundary elements are confined with spirals or ties with spacing less than $8d_b$.	5.5.3.2.2	A.3.2.2.5	Compliant per wall elevations 604_3-6-6.
☒	☐	☐	☐	WALL REINFORCING AT OPENINGS: There is added trim reinforcement around all wall openings with a dimension greater than three times the thickness of the wall.	5.5.3.1.5	A.3.2.2.6	This is compliant per typical details.
☒	☐	☐	☐	WALL THICKNESS: Thicknesses of bearing walls are not less than 1/25 the unsupported height or length, whichever is shorter, nor less than 4 in. (101 mm).	5.5.3.1.2	A.3.2.2.7	This is compliant per typical wall sections and details.
Diaphragms (Stiff or Flexible)							
☒	☐	☐	☐	DIAPHRAGM CONTINUITY: The diaphragms are not composed of split-level floors and do not have expansion joints.	5.6.1.1	A.4.1.1	No split-level floors or expansion joints.
☐	☒	☐	☐	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 15% of the wall length.	5.6.1.3	A.4.1.4	
☒	☐	☐	☐	PLAN IRREGULARITIES: There is tensile capacity to develop the strength of the diaphragm at reentrant corners or other locations of plan irregularities.	5.6.1.4	A.4.1.7	Reentrant corner reinforcing in diaphragms occurs per the record drawings.
☐	☐	☒	☐	DIAPHRAGM REINFORCEMENT AT OPENINGS: There is reinforcing around all diaphragm openings larger than 50% of the building width in either major plan dimension.	5.6.1.5	A.4.1.8	No openings larger than 50% of the building width in either direction.
Flexible Diaphragms							
☐	☐	☒	☐	CROSS TIES: There are continuous cross ties between diaphragm chords.	5.6.1.2	A.4.1.2	No significant flexible diaphragms.
☐	☐	☒	☐	STRAIGHT SHEATHING: All straight-sheathed diaphragms have aspect ratios less than 1-to-1 in the direction being considered.	5.6.2	A.4.2.1	No significant flexible diaphragms.
☐	☐	☒	☐	SPANS: All wood diaphragms with spans greater than 12 ft (3.6 m) consist of wood structural panels or diagonal sheathing.	5.6.2	A.4.2.2	No significant flexible diaphragms.

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

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C	NC	N/A	U				
☐	☐	☒	☐	DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 30 ft (9.2 m) and aspect ratios less than or equal to 3-to-1.	5.6.2	A.4.2.3	No significant flexible diaphragms.
☐	☐	☒	☐	NONCONCRETE FILLED DIAPHRAGMS: Untopped metal deck diaphragms or metal deck diaphragms with fill other than concrete consist of horizontal spans of less than 40 ft (12.2 m) and have aspect ratios less than 4-to-1.	5.6.3	A.4.3.1	No significant flexible diaphragms.
☐	☐	☒	☐	OTHER DIAPHRAGMS: Diaphragms do not consist of a system other than wood, metal deck, concrete, or horizontal bracing.	5.6.5	A.4.7.1	No significant flexible diaphragms.
Connections							
C	NC	N/A	U				
☐	☐	☐	☒	UPLIFT AT PILE CAPS: Pile caps have top reinforcement, and piles are anchored to the pile caps; the pile cap reinforcement and pile anchorage are able to develop the tensile capacity of the piles.	5.7.3.5	A.5.3.8	Piles are may be anchored to pile caps however it does not appear to be substantial or adequate. Net uplift must be evaluated.

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

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APPENDIX B: SUPPORTING CALCULATIONS

Attorney Client Privileged
Prepared at the Direction of Counsel

MARTIN/MARTIN
CONSULTING ENGINEERS

Title Date Job No.

Subject By Sheet

_ The concrete moment frames are evaluated to satisfy **Immediate Occupancy** Performance Objective under ASCE 41-17 for the **BSE-1E** Hazard Level.

_ The stiffness of Beam-Column moment frames is computed and modeled implicitly per 10.4.2.2

_ The shear strength of columns, Vcol, is computed per 10.4.2.3.1 unless detailing meets requirements of ACI 318 Ch. 18.

_The shear strength of beam-column joints, Vj, is computed per 10.4.2.3.2

_ The moment frames will be checked under **deformation-controlled** action. Per ASCE 41-17 Sec 7.5.2.2.1:
mkQCE > QUD
while:
m: component capacity modification factor for beam/column/joint (see below)
f'cE = 5000psi x 1.5 = 7500 (expected strength per Table 10.1)
fyE = 60000psi x 1.25 = 75000 (expected strength per Table 10.1)
Per Sec 6.2.4, k is taken as 1.0
QUD = QG ± QE
QG = 1.1(QD+QL) & QG = 0.9(QD)

_ The concrete moment frames are evaluated to satisfy **Life Safety** Performance Objective under ASCE 41-17 for the **BSE-2E** Hazard Level.

_ The stiffness of Beam-Column moment frames is computed and modeled implicitly per 10.4.2.2

_ The shear strength of columns, Vcol, is computed per 10.4.2.3.1 unless detailing meets requirements of ACI 318 Ch. 18.

_The shear strength of beam-column joints, Vj, is computed per 10.4.2.3.2

_ The moment frames will be checked under **deformation-controlled** action. Per ASCE 41-17 Sec 7.5.2.2.1:
mkQCE > QUD
while:
m: component capacity modification factor for beam/column/joint (see below)
f'cE = 5000psi x 1.5 = 7500 (expected strength per Table 10.1)
fyE = 60000psi x 1.25 = 75000 (expected strength per Table 10.1)
Per Sec 6.2.4, k is taken as 1.0
QUD = QG ± QE
QG = 1.1(QD+QL) & QG = 0.9(QD)

Table 10-10a. Numerical Acceptance Criteria for Linear Procedures—Reinforced Concrete Columns Other Than Circular with Spiral Reinforcement or Seismic Hoops as Defined in ACI 318

CONSERVATIVE

			m-Factors[a]				
			Performance Level				
				Component Type			
				Primary		Secondary	
$\left(\frac{N_{UD}}{A_g f'_{cE}}\right)$	ρ_t	V_{yE}/V_{ColOE}	IO	LS	CP	LS	CP
Columns not controlled by inadequate development or splicing along the clear height[b]							
≤ 0.1	≥ 0.0175	≥ 0.2 < 0.6	1.7	3.4	4.2	6.8	8.9
≥ 0.7	≥ 0.0175	≥ 0.2 < 0.6	1.2	1.4	1.7	1.4	1.7
≤ 0.1	≤ 0.0005	≥ 0.2 < 0.6	1.5	2.6	3.2	2.6	3.2
≥ 0.7	≤ 0.0005	≥ 0.2 < 0.6	1.0	1.0	1.0	1.0	1.0
≤ 0.1	≥ 0.0175	≥ 0.6 < 1.0	1.5	2.7	3.3	6.8	8.9
≥ 0.7	≥ 0.0175	≥ 0.6 < 1.0	1.0	1.0	1.0	1.0	1.0
≤ 0.1	≤ 0.0005	≥ 0.6 < 1.0	1.3	1.9	2.3	1.9	2.3
≥ 0.7	≤ 0.0005	≥ 0.6 < 1.0	1.0	1.0	1.0	1.0	1.0
≤ 0.1	≥ 0.0175	≥ 1.0	1.3	1.8	2.2	6.8	8.9
≥ 0.7	≥ 0.0175	≥ 1.0	1.0	1.0	1.0	1.0	1.0
≤ 0.1	≤ 0.0005	≥ 1.0	1.1	1.0	1.1	1.7	2.1
≥ 0.7	≤ 0.0005	≥ 1.0	1.0	1.0	1.0	1.0	1.0
Columns controlled by inadequate development or splicing along the clear height[b]							
≤ 0.1	≥ 0.0075		1.0	1.7	2.0	5.3	6.8
≥ 0.7	≥ 0.0075		1.0	1.0	1.0	2.8	3.5
≤ 0.1	≤ 0.0005		1.0	1.0	1.0	1.4	1.6
≥ 0.7	≤ 0.0005		1.0	1.0	1.0	1.0	1.0

[a] Values between those listed in the table shall be determined by linear interpolation.
[b] Columns are considered to be controlled by inadequate development or splicing where the calculated steel stress at the splice exceeds the steel stress specified by Eq. (10-1a) or (10-1b). Acceptance criteria for columns controlled by inadequate development or splicing shall never exceed those of columns not controlled by inadequate development or splicing.

Table 10-12. Values of γ for Joint Strength Calculation

	Value of γ				
	Condition i: Interior Joints[a]		Condition ii: Other Joints		
Transverse Reinforcement[b]	Interior Joint with Transverse Beams	Interior Joint without Transverse Beams	Exterior Joint with Transverse Beams	Exterior Joint without Transverse Beams	Knee Joint with or without Transverse Beams
C	20	15	15	12	8
NC	12	10	8	6	4

[a] For classification of joints, refer to Fig. 10-3.
[b] "C" and "NC" are abbreviations for conforming and nonconforming transverse reinforcement. Joint transverse reinforcement is conforming if hoops are spaced at $\leq h_c/2$ within the joint. Otherwise, the transverse reinforcement is considered nonconforming.

Table 10-13. Numerical Acceptance Criteria for Linear Procedures—Reinforced Concrete Beams

			m-Factors[a]				
			Performance Level				
			Component Type				
			Primary			Secondary	
Conditions			IO	LS	CP	LS	CP
Condition i. Beams controlled by flexure[b]							
$\frac{\rho - \rho'}{\rho_{bal}}$	Transverse reinforcement[c]	$\frac{V^d}{b_w d\sqrt{f'_{cE}}}$					
≤0.0	C	≤3 (0.25)	3	6	7	6	10
≤0.0	C	≥6 (0.5)	2	3	4	3	5
≥0.5	C	≤3 (0.25)	2	3	4	3	5
≥0.5	C	≥6 (0.5)	2	2	3	2	4
≤0.0	NC	≤3 (0.25)	2	3	4	3	5
≤0.0	NC	≥6 (0.5)	1.25	2	3	2	4
≥0.5	NC	≤3 (0.25)	2	3	3	3	4
≥0.5	NC	≥6 (0.5)	1.25	2	2	2	3
Condition ii. Beams controlled by shear[b]							
Stirrup spacing ≤ *d*/2			1.25	1.5	1.75	3	4
Stirrup spacing > *d*/2			1.25	1.5	1.75	2	3
Condition iii. Beams controlled by inadequate development or splicing along the span[b]							
Stirrup spacing ≤ *d*/2			1.25	1.5	1.75	3	4
Stirrup spacing > *d*/2			1.25	1.5	1.75	2	3
Condition iv. Beams controlled by inadequate embedment into beam–column joint[b]			2	2	3	3	4

Note: f'_{cE} in lb/in.^{2} (MPa) units.
[a] Values between those listed in the table shall be determined by linear interpolation.
[b] Where more than one of conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table.
[c] "C" and "NC" are abbreviations for conforming and nonconforming transverse reinforcement. Transverse reinforcement is conforming if, within the flexural plastic hinge region, hoops are spaced at $\leq d/3$, and if, for components of moderate and high ductility demand, the strength provided by the hoops (V_s) is at least 3/4 of the design shear. Otherwise, the transverse reinforcement is considered nonconforming.
[d] V is the shear force calculated using limit-state analysis procedures in accordance with Section 10.4.2.4.1.

Table 10-14. Numerical Acceptance Criteria for Linear Procedures—Reinforced Concrete Beam–Column Joints

			m-Factors[a]				
			Performance Level				
				Component Type			
				Primary		Secondary	
Conditions			IO	LS	CP	LS	CP
Condition i. Interior joints (for classification of joints, refer to Fig. 10-3)							
$\frac{P^b}{A_g f'_{cE}}$	Transverse reinforcement[c]	$\frac{V^d}{V_J}$					
≤0.1	C	≤1.2	1	1	1	3	4
≤0.1	C	≥1.5	1	1	1	2	3
≥0.4	C	≤1.2	1	1	1	3	4
≥0.4	C	≥1.5	1	1	1	2	3
≤0.1	NC	≤1.2	1	1	1	2	3
≤0.1	NC	≥1.5	1	1	1	2	3
≥0.4	NC	≤1.2	1	1	1	2	3
≥0.4	NC	≥1.5	1	1	1	2	3
Condition ii. Other joints (for classification of joints, refer to Fig. 10-3)							
$\frac{P^b}{A_g f'_{cE}}$	Transverse reinforcement[c]	$\frac{V^d}{V_J}$					
≤0.1	C	≤1.2	1	1	1	3	4
≤0.1	C	≥1.5	1	1	1	2	3
≥0.4	C	≤1.2	1	1	1	3	4
≥0.4	C	≥1.5	1	1	1	2	3
≤0.1	NC	≤1.2	1	1	1	2	3
≤0.1	NC	≥1.5	1	1	1	2	3
≥0.4	NC	≤1.2	1	1	1	1.5	2
≥0.4	NC	≥1.5	1	1	1	1.5	2

[a] Values between those listed in the table shall be determined by linear interpolation.
[b] P is the design axial force on the column above the joint calculated using limit-state analysis procedures in accordance with Section 10.4.2.4. A_g is the gross cross-sectional area of the joint.
[c] V is the shear force and V_J is the shear strength for the joint. The design shear force and shear strength should be calculated according to Section 10.4.2.4.1 and Section 10.4.2.3, respectively.
[d] "C" and "NC" are abbreviations for conforming and nonconforming transverse reinforcement, respectively. Transverse reinforcement is conforming if hoops are spaced at $\leq h_c/2$ within the joint. Otherwise, the transverse reinforcement is considered nonconforming.

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Title | Date | Job No.

Subject | By | Sheet

_ The concrete shear wall running continuously to the roof will be checked to satisfy **Immediate Occupancy** Performance Objective under ASCE 41-17 for the **BSE-1E** Hazard Level.
_ Concrete shear wall will be checked under deformation-controlled action. Per ASCE 41-17 Sec 7.5.2.2.1:
mkQ_{CE} > Q_{UD}
while:
m: component capacity modification factor for concrete shear wall (see below)
f'cE = 5000psi x 1.5 = 7500 (expected strength per Table 10.1)
fyE = 60000psi x 1.25 = 75000 (expected strength per Table 10.1)
Per Sec 6.2.4, k is taken as 1.0
$Q_{UD} = Q_G \pm Q_E$
$Q_G = 1.1(Q_D+Q_L)$ & $Q_G = 0.9(Q_D)$
_ The walls chosen are checked for both shear and flexural behavior. The stirrups at boundary elements appear to meet the ACI code requirement, thus walls are assumed to have confined boundary.
_ The m-factors at IO performance level for both flexural and shear action are 2 due to the axial loads are low per Tables 10-21 & 10-22. See attached for detail calculation.

Table 10-21. Numerical Acceptance Criteria for Linear Procedures—R/C Structural Walls and Associated Components Controlled by Flexure

			m-Factors[a]				
			Performance Level				
			Component Type				
			Primary			Secondary	
Conditions			IO	LS	CP	LS	CP
i. Structural walls and wall segments							
$\frac{(A_s - A'_s)f_{yE} + P}{t_w l_w f'_{cE}}$ [b]	$\frac{V^c}{t_w l_w \sqrt{f'_{cE}}}$	Confined Boundary[d]					
≤0.1	≤4	Yes	2	4	6	6	8
≤0.1	≥6	Yes	2	3	4	4	6
≥0.25	≤4	Yes	1.5	3	4	4	6
≥0.25	≥6	Yes	1.25	2	2.5	2.5	4

Table 10-22. Numerical Acceptance Criteria for Linear Procedures—R/C Structural Walls and Associated Components Controlled by Shear

	m-Factors				
	Performance Level				
	Component Type				
	Primary			Secondary	
Conditions	IO	LS	CP	LS	CP
i. Structural walls and wall segments[a]					
$\frac{(A_s - A'_s)f_{yE} + P}{t_w l_w f'_{cE}} \le 0.05$	2	2.5	3	4.5	6
$\frac{(A_s - A'_s)f_{yE} + P}{t_w l_w f'_{cE}} > 0.05$	1.5	2	3	3	4

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JOB TITLE SACRAMENTO JAIL SEISMIC EVALUATION

JOB NO. XX.XXXX.S.XX SHEET NO.

CALCULATED BY TN DATE 6/18/2026

DESIGN CRITERIA

PERFORMANCE OBJECTIVE	IO	
SEISMIC HAZARD LEVEL	BSE-1E	
COMPONENT ACTION	DC	Deformation-Controlled
EXPECTED CONCRETE COMPRESSIVE STRENGTH, F'CE	7500	psi
EXPECTED STEEL YIELD STRENGTH, FYE	75000	psi
KNOWLEDGE FACTOR, K	1	

m-FACTOR DETERMINATION

100 HIGHEST Pu

Pier	Length (in)	Thickness (in)	Load Combo	Pu (kips)	Vu (kips)	Mu (kip-ft)	P/(tlf'cE)	V/(tlVf'cE)	m_f-factor	m_s-factor
A7.2	1071	12	DC: 1.1(D+0.25L)+BSE-1EY-5	-8332.31	-63.86	126330	7.2036E-06	0.000	2	2
A7.2	1071	12	DC: 1.1(D+0.25L)+BSE-1EY-5	-8332.31	-63.86	126330	7.2036E-06	0.000	2	2
A7.2	1071	12	DC: 1.1(D+0.25L)+BSE-1EY-5	-8332.31	-63.86	126330	7.2036E-06	0.000	2	2
A7.2	1071	12	DC: 1.1(D+0.25L)+BSE-1EY-5	-8332.31	-63.86	126330	7.2036E-06	0.000	2	2
A7.2	1071	12	DC: 1.1(D+0.25L)+BSE-1EX-5	-8331.757	454.48	152575	7.2032E-06	0.000	2	2
A7.2	1071	12	DC: 1.1(D+0.25L)+BSE-1EX-5	-8331.757	454.48	152575	7.2032E-06	0.000	2	2
A7.2	1071	12	DC: 1.1(D+0.25L)+BSE-1EX-5	-8331.757	454.48	152575	7.2032E-06	0.000	2	2
A7.2	1071	12	DC: 1.1(D+0.25L)+BSE-1EX-5	-8331.757	454.48	152575	7.2032E-06	0.000	2	2
A7.2	1071	12	DC: 1.1(D+0.25L)+BSE-1EX-1	-7748.165	454.48	152575	6.6986E-06	0.000	2	2
A7.2	1071	12	DC: 1.1(D+0.25L)+BSE-1EX-1	-7748.165	454.48	152575	6.6986E-06	0.000	2	2
A7.2	1071	12	DC: 1.1(D+0.25L)+BSE-1EX-1	-7748.165	454.48	152575	6.6986E-06	0.000	2	2
A7.2	1071	12	DC: 1.1(D+0.25L)+BSE-1EX-1	-7748.165	454.48	152575	6.6986E-06	0.000	2	2
A7.2	1071	12	DC: 1.1(D+0.25L)+BSE-1EY-1	-7747.612	-63.86	126330	6.6981E-06	0.000	2	2
A7.2	1071	12	DC: 1.1(D+0.25L)+BSE-1EY-1	-7747.612	-63.86	126330	6.6981E-06	0.000	2	2
A7.2	1071	12	DC: 1.1(D+0.25L)+BSE-1EY-1	-7747.612	-63.86	126330	6.6981E-06	0.000	2	2
A7.2	1071	12	DC: 1.1(D+0.25L)+BSE-1EY-1	-7747.612	-63.86	126330	6.6981E-06	0.000	2	2
C.14	408	14	DC: 1.1(D+0.25L)+BSE-1EY-5	-7015.513	372.599	5378	1.3647E-05	0.000	2	2
C.14	408	14	DC: 1.1(D+0.25L)+BSE-1EY-5	-7015.513	372.599	5378	1.3647E-05	0.000	2	2
C.14	408	14	DC: 1.1(D+0.25L)+BSE-1EY-5	-7015.513	372.599	5378	1.3647E-05	0.000	2	2
C.14	408	14	DC: 1.1(D+0.25L)+BSE-1EY-5	-7015.513	372.599	5378	1.3647E-05	0.000	2	2
B7.5	654	12	DC: 1.1(D+0.25L)+BSE-1EY-5	-6768.211	708.242	40773	9.5824E-06	0.000	2	2
B7.5	654	12	DC: 1.1(D+0.25L)+BSE-1EY-5	-6768.211	708.242	40773	9.5824E-06	0.000	2	2
B7.5	654	12	DC: 1.1(D+0.25L)+BSE-1EY-5	-6768.211	708.242	40773	9.5824E-06	0.000	2	2
B7.5	654	12	DC: 1.1(D+0.25L)+BSE-1EY-5	-6768.211	708.242	40773	9.5824E-06	0.000	2	2
B7.7	546	12	DC: 1.1(D+0.25L)+BSE-1EY-5	-6581.022	73.558	10750	1.1160E-05	0.000	2	2
B7.7	546	12	DC: 1.1(D+0.25L)+BSE-1EY-5	-6581.022	73.558	10750	1.1160E-05	0.000	2	2
B7.7	546	12	DC: 1.1(D+0.25L)+BSE-1EY-5	-6581.022	73.558	10750	1.1160E-05	0.000	2	2
B7.7	546	12	DC: 1.1(D+0.25L)+BSE-1EY-5	-6581.022	73.558	10750	1.1160E-05	0.000	2	2
A7.2	1071	12	DC: 0.9D+BSE-1EY-5	-6575.178	9.106	102580	5.6845E-06	0.000	2	2
A7.2	1071	12	DC: 0.9D+BSE-1EY-5	-6575.178	9.106	102580	5.6845E-06	0.000	2	2
A7.2	1071	12	DC: 0.9D+BSE-1EY-5	-6575.178	9.106	102580	5.6845E-06	0.000	2	2
A7.2	1071	12	DC: 0.9D+BSE-1EY-5	-6575.178	9.106	102580	5.6845E-06	0.000	2	2
A7.2	1071	12	DC: 0.9D+BSE-1EX-5	-6574.625	527.446	128825	5.6840E-06	0.000	2	2
A7.2	1071	12	DC: 0.9D+BSE-1EX-5	-6574.625	527.446	128825	5.6840E-06	0.000	2	2
A7.2	1071	12	DC: 0.9D+BSE-1EX-5	-6574.625	527.446	128825	5.6840E-06	0.000	2	2
A7.2	1071	12	DC: 0.9D+BSE-1EX-5	-6574.625	527.446	128825	5.6840E-06	0.000	2	2
C.14	408	14	DC: 0.9D+BSE-1EY-5	-6489.359	363.093	5896	1.2623E-05	0.000	2	2
C.14	408	14	DC: 0.9D+BSE-1EY-5	-6489.359	363.093	5896	1.2623E-05	0.000	2	2
C.14	408	14	DC: 0.9D+BSE-1EY-5	-6489.359	363.093	5896	1.2623E-05	0.000	2	2
C.14	408	14	DC: 0.9D+BSE-1EY-5	-6489.359	363.093	5896	1.2623E-05	0.000	2	2
B7.5	654	12	DC: 1.1(D+0.25L)+BSE-1EY-5	-6429.001	835.103	49303	9.1021E-06	0.000	2	2
B7.5	654	12	DC: 1.1(D+0.25L)+BSE-1EY-5	-6429.001	835.103	49303	9.1021E-06	0.000	2	2
B7.5	654	12	DC: 1.1(D+0.25L)+BSE-1EY-5	-6429.001	835.103	49303	9.1021E-06	0.000	2	2
B7.5	654	12	DC: 1.1(D+0.25L)+BSE-1EY-5	-6429.001	835.103	49303	9.1021E-06	0.000	2	2
H.2	3570	14	DC: 1.1(D+0.25L)+BSE-1EY-5	-6303.078	257.743	53002	1.4012E-06	0.000	2	2
H.2	3570	14	DC: 1.1(D+0.25L)+BSE-1EY-5	-6303.078	257.743	53002	1.4012E-06	0.000	2	2
H.2	3570	14	DC: 1.1(D+0.25L)+BSE-1EY-5	-6303.078	257.743	53002	1.4012E-06	0.000	2	2
H.2	3570	14	DC: 1.1(D+0.25L)+BSE-1EY-5	-6303.078	257.743	53002	1.4012E-06	0.000	2	2
12.1.1	1095.96	12	DC: 1.1(D+0.25L)+BSE-1EX-5	-6214.023	310.554	50978	5.2499E-06	0.000	2	2
12.1.1	1095.96	12	DC: 1.1(D+0.25L)+BSE-1EX-5	-6214.023	310.554	50978	5.2499E-06	0.000	2	2
12.1.1	1095.96	12	DC: 1.1(D+0.25L)+BSE-1EX-5	-6214.023	310.554	50978	5.2499E-06	0.000	2	2
12.1.1	1095.96	12	DC: 1.1(D+0.25L)+BSE-1EX-5	-6214.023	310.554	50978	5.2499E-06	0.000	2	2
B7.5	654	12	DC: 1.1(D+0.25L)+BSE-1EX-5	-6167.816	1265.894	28860	8.7323E-06	0.000	2	2

B7.5	654	12	DC: 1.1(D+0.25L)+BSE-1EX-5	-6167.816	1265.894	28860	8.7323E-06	0.000	2	2
B7.5	654	12	DC: 1.1(D+0.25L)+BSE-1EX-5	-6167.816	1265.894	28860	8.7323E-06	0.000	2	2
B7.5	654	12	DC: 1.1(D+0.25L)+BSE-1EX-5	-6167.816	1265.894	28860	8.7323E-06	0.000	2	2
B7.7	546	12	DC: 0.9D+BSE-1EY-5	-6016.038	94.422	10434	1.0202E-05	0.000	2	2
B7.7	546	12	DC: 0.9D+BSE-1EY-5	-6016.038	94.422	10434	1.0202E-05	0.000	2	2
B7.7	546	12	DC: 0.9D+BSE-1EY-5	-6016.038	94.422	10434	1.0202E-05	0.000	2	2
B7.7	546	12	DC: 0.9D+BSE-1EY-5	-6016.038	94.422	10434	1.0202E-05	0.000	2	2
A7.2	1071	12	DC: 0.9D+BSE-1EX-1	-5991.033	527.446	128825	5.1795E-06	0.000	2	2
A7.2	1071	12	DC: 0.9D+BSE-1EX-1	-5991.033	527.446	128825	5.1795E-06	0.000	2	2
A7.2	1071	12	DC: 0.9D+BSE-1EX-1	-5991.033	527.446	128825	5.1795E-06	0.000	2	2
A7.2	1071	12	DC: 0.9D+BSE-1EX-1	-5991.033	527.446	128825	5.1795E-06	0.000	2	2
A7.2	1071	12	DC: 0.9D+BSE-1EY-1	-5990.48	9.106	102580	5.1790E-06	0.000	2	2
A7.2	1071	12	DC: 0.9D+BSE-1EY-1	-5990.48	9.106	102580	5.1790E-06	0.000	2	2
A7.2	1071	12	DC: 0.9D+BSE-1EY-1	-5990.48	9.106	102580	5.1790E-06	0.000	2	2
A7.2	1071	12	DC: 0.9D+BSE-1EY-1	-5990.48	9.106	102580	5.1790E-06	0.000	2	2
C.14	408	14	DC: 1.1(D+0.25L)+BSE-1EY-5	-5936.248	240.561	3950	1.1547E-05	0.000	2	2
C.14	408	14	DC: 1.1(D+0.25L)+BSE-1EY-5	-5936.248	240.561	3950	1.1547E-05	0.000	2	2
C.14	408	14	DC: 1.1(D+0.25L)+BSE-1EY-5	-5936.248	240.561	3950	1.1547E-05	0.000	2	2
C.14	408	14	DC: 1.1(D+0.25L)+BSE-1EY-5	-5936.248	240.561	3950	1.1547E-05	0.000	2	2
B7.5	654	12	DC: 0.9D+BSE-1EY-5	-5877.447	725.712	39896	8.3212E-06	0.000	2	2
B7.5	654	12	DC: 0.9D+BSE-1EY-5	-5877.447	725.712	39896	8.3212E-06	0.000	2	2
B7.5	654	12	DC: 0.9D+BSE-1EY-5	-5877.447	725.712	39896	8.3212E-06	0.000	2	2
B7.5	654	12	DC: 0.9D+BSE-1EY-5	-5877.447	725.712	39896	8.3212E-06	0.000	2	2
12.1.1	1095.96	12	DC: 1.1(D+0.25L)+BSE-1EX-5	-5875.305	306.271	56742	4.9638E-06	0.000	2	2
12.1.1	1095.96	12	DC: 1.1(D+0.25L)+BSE-1EX-5	-5875.305	306.271	56742	4.9638E-06	0.000	2	2
12.1.1	1095.96	12	DC: 1.1(D+0.25L)+BSE-1EX-5	-5875.305	306.271	56742	4.9638E-06	0.000	2	2
12.1.1	1095.96	12	DC: 1.1(D+0.25L)+BSE-1EX-5	-5875.305	306.271	56742	4.9638E-06	0.000	2	2
B7.7	546	12	DC: 1.1(D+0.25L)+BSE-1EY-5	-5841.059	178.089	6323	9.9055E-06	0.000	2	2
B7.7	546	12	DC: 1.1(D+0.25L)+BSE-1EY-5	-5841.059	178.089	6323	9.9055E-06	0.000	2	2
B7.7	546	12	DC: 1.1(D+0.25L)+BSE-1EY-5	-5841.059	178.089	6323	9.9055E-06	0.000	2	2
B7.7	546	12	DC: 1.1(D+0.25L)+BSE-1EY-5	-5841.059	178.089	6323	9.9055E-06	0.000	2	2
-A2.1	1548	12	DC: 1.1(D+0.25L)+BSE-1EY-5	-5834.158	415.735	44032	3.4897E-06	0.000	2	2
-A2.1	1548	12	DC: 1.1(D+0.25L)+BSE-1EY-5	-5834.158	415.735	44032	3.4897E-06	0.000	2	2
-A2.1	1548	12	DC: 1.1(D+0.25L)+BSE-1EY-5	-5834.158	415.735	44032	3.4897E-06	0.000	2	2
-A2.1	1548	12	DC: 1.1(D+0.25L)+BSE-1EY-5	-5834.158	415.735	44032	3.4897E-06	0.000	2	2
H.2	3570	14	DC: 0.9D+BSE-1EY-5	-5727.049	272.377	78387	1.2732E-06	0.000	2	2
H.2	3570	14	DC: 0.9D+BSE-1EY-5	-5727.049	272.377	78387	1.2732E-06	0.000	2	2
H.2	3570	14	DC: 0.9D+BSE-1EY-5	-5727.049	272.377	78387	1.2732E-06	0.000	2	2
H.2	3570	14	DC: 0.9D+BSE-1EY-5	-5727.049	272.377	78387	1.2732E-06	0.000	2	2
B7.5	654	12	DC: 0.9D+BSE-1EY-5	-5707.573	842.249	49586	8.0807E-06	0.000	2	2
B7.5	654	12	DC: 0.9D+BSE-1EY-5	-5707.573	842.249	49586	8.0807E-06	0.000	2	2
B7.5	654	12	DC: 0.9D+BSE-1EY-5	-5707.573	842.249	49586	8.0807E-06	0.000	2	2
B7.5	654	12	DC: 0.9D+BSE-1EY-5	-5707.573	842.249	49586	8.0807E-06	0.000	2	2
1.2	1812	14	DC: 1.1(D+0.25L)+BSE-1EX-5	-5683.282	516.939	127799	2.4893E-06	0.000	2	2
1.2	1812	14	DC: 1.1(D+0.25L)+BSE-1EX-5	-5683.282	516.939	127799	2.4893E-06	0.000	2	2
1.2	1812	14	DC: 1.1(D+0.25L)+BSE-1EX-5	-5683.282	516.939	127799	2.4893E-06	0.000	2	2
1.2	1812	14	DC: 1.1(D+0.25L)+BSE-1EX-5	-5683.282	516.939	127799	2.4893E-06	0.000	2	2

Floor Plan Redacted

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ETABS 23.1.1

SAMPLE CALC FOR PIER C.3 DESIGN

ETABS Shear Wall Design

ACI 318-19 Pier Design

Pier Details

Story ID	Pier ID	Centroid X (in)	Centroid Y (in)	Length (in)	Thickness (in)	Height (in)	Pier Ductility	LLRF
3rd Floor	C.3	1893.2382	1272	128.1769	14	162	Special Structural Wall	0.501

Material Properties

E_c (lb/in²)	f'_c (lb/in²)	Lt.Wt Factor (Unitless)	f_y (lb/in²)	f_{ys} (lb/in²)
3604996.5	7500	1	75000	75000

Design System Parameters

ρ	S_{DS}	Ω_0	SDC	Type
1	0.5	2.5	D	Special Structural Wall

Design Code Parameters

ϕ_T	ϕ_C	ϕ_v	ϕ_v (Seismic)	IP_{MAX}	IP_{MIN}	P_{MAX}
2	2	2	2	0.04	0.0025	0.8

Pier Leg Location, Length and Thickness

Station Location	ID	Left X_1 in	Left Y_1 in	Right X_2 in	Right Y_2 in	Length in	Thickness in	Pier A_g in²
Top	Leg 1	1829.1498	1272	1957.3267	1272	128.1769	14	1680
Bottom	Leg 1	1829.1498	1272	1957.3267	1272	128.1769	14	1680

Flexural Design for P, M_3 and M_2

Flexural is Ok

Station	D/C	Flexural	P_u kip	M_{u2} kip-ft	M_{u3} kip-ft
Top	0.137	DC: 0.9D+BSE-1EX	260.739	48.8797	-4762.5349
Bottom	0.384	DC: 0.9D+BSE-1EX	283.45	-45.5862	-12178.4765

Axial Force and Minor Moment Factors

Station Location	C_m Factor Unitless	δ_{ns} Factor Unitless	β_{dns} Unitless	EI_{eff} lb-ft²	P_c kip
Top Minor Bend(M2)	0.879024	1	1	146752296.99	7947.254
Bottom Minor Bend(M2)	0.723329	1	1	146752296.99	7947.254

Shear Design Capacity Parameters

Station Location	ID	Shear Combo	$M_{pr,major}$ kip-ft	Ω_v Unitless	ω_v Unitless	$\Omega_v * \omega_v$ Unitless
Top	Leg 1	DC: 1.1(D+0.25L)+BSE-1EX	0	1	1	1
Bottom	Leg 1	DC: 1.1(D+0.25L)+BSE-1EX	0	1	1	1

Shear Design (Part 1 of 2)

Station Location	ID	Rebar in²/ft	Shear Combo	P_u kip	M_u kip-ft	V_u kip	$V_{u,design}$ kip	h_w/l_w Unitless
Top	Leg 1	0.42	DC: 1.1(D+0.25L)+BSE-1EX	401.992	4607.769	564.257	564.257	11.374906
Bottom	Leg 1	0.42	DC: 1.1(D+0.25L)+BSE-1EX	429.751	11286.7212	564.257	564.257	11.374906

Shear Design (Part 2 of 2)

α_c Unitless	ϕV_c kip	ϕV_n kip
2	621.625	1294.553
2	621.625	1294.553

Shear is Ok

Boundary Element Check (ACI 18.10.6.3, 18.10.6.4) (Part 1 of 2)

Station Location	ID	Edge Length (in)	Governing Combo	P_u kip	M_u kip-ft	Stress Comp lb/in²	Stress Limit lb/in²
Top–Left	Leg 1	14.2315	DC: 1.1(D+0.25L)+BSE-1EX	795.648	-4785.9627	1941.53	1500
Top–Right	Leg 1	14.2315	DC: 1.1(D+0.25L)+BSE-1EX	795.648	4607.769	1885.75	1500
Bottom–Left	Leg 1	14.3783	DC: 1.1(D+0.25L)+BSE-1EX	823.407	-12275.9593	4301.59	1500
Botttom–Right	Leg 1	14.3783	DC: 1.1(D+0.25L)+BSE-1EX	823.407	11286.7212	3991.93	1500

Boundary Element Check (ACI 18.10.6.3, 18.10.6.4) (Part 2 of 2)

C Depth in	C Limit in
27.0492	16.3127
27.0492	16.3127
27.196	16.3127
27.196	16.3127

ETABS 23.1.1

SAMPLE CALC FOR PIER 6.1 DESIGN

ETABS Shear Wall Design

ACI 318-19 Pier Design

Pier Details

Story ID	Pier ID	Centroid X (in)	Centroid Y (in)	Length (in)	Thickness (in)	Height (in)	Pier Ductility	LLRF
3rd Floor	6.1	1698	1078.5	117	12	162	Special Structural Wall	0.458

Material Properties

E_c (lb/in²)	f'_c (lb/in²)	Lt.Wt Factor (Unitless)	f_y (lb/in²)	f_{ys} (lb/in²)
3604996.5	7500	1	75000	75000

Design System Parameters

ρ	S_{DS}	Ω_0	SDC	Type
1	0.5	2.5	D	Special Structural Wall

Design Code Parameters

ϕ_T	ϕ_C	ϕ_v	ϕ_v (Seismic)	IP_{MAX}	IP_{MIN}	P_{MAX}
2	2	2	2	0.04	0.0025	0.8

Pier Leg Location, Length and Thickness

Station Location	ID	Left X_1 in	Left Y_1 in	Right X_2 in	Right Y_2 in	Length in	Thickness in	Pier A_g in²
Top	Leg 1	1698	1020	1698	1137	117	12	1440
Bottom	Leg 1	1698	1020	1698	1137	117	12	1440

Flexural Design for P, M_3 and M_2

Flexural is Ok

Station	D/C	Flexural	P_u kip	M_{u2} kip-ft	M_{u3} kip-ft
Top	0.127	DC: 1.1(D+0.25L)+BSE-1EY	742.597	5.9903	3671.9421
Bottom	0.235	DC: 0.9D+BSE-1EY	477.552	6.7411	-6265.1599

Axial Force and Minor Moment Factors

Station Location	C_m Factor Unitless	δ_{ns} Factor Unitless	β_{dns} Unitless	EI_{eff} lb-ft²	P_c kip
Top Minor Bend(M2)	0.973744	1.217979	0.850375	91178203.49	4937.683
Bottom Minor Bend(M2)	0.992332	1.153046	1	84356918.1	4568.282

Shear Design Capacity Parameters

Station Location	ID	Shear Combo	$M_{pr,major}$ kip-ft	Ω_v Unitless	ω_v Unitless	$\Omega_v * \omega_v$ Unitless
Top	Leg 1	DC: 1.1(D+0.25L)+BSE-1EY	0	1	1	1
Bottom	Leg 1	DC: 1.1(D+0.25L)+BSE-1EY	0	1	1	1

Shear Design (Part 1 of 2)

Station Location	ID	Rebar in²/ft	Shear Combo	P_u kip	M_u kip-ft	V_u kip	$V_{u,design}$ kip	h_w/l_w Unitless	α_c Unitless
Top	Leg 1	0.36	DC: 1.1(D+0.25L)+BSE-1EY	628.823	3671.9421	267.966	267.966	14.615385	2
Bottom	Leg 1	0.36	DC: 1.1(D+0.25L)+BSE-1EY	650.541	6090.114	267.966	267.966	14.615385	2

Shear Design (Part 2 of 2)

ϕV_c kip	ϕV_n kip
486.36	1012.86
486.36	1012.86

Shear is Ok

Boundary Element Check (ACI 18.10.6.3, 18.10.6.4)

Station Location	ID	Edge Length (in)	Governing Combo	P_u kip	M_u kip-ft	Stress Comp lb/in²	Stress Limit lb/in²	C Depth in	C Limit in
Top–Left	Leg 1	13.2692	DC: 1.1(D+0.25L)+BSE-1EY	742.597	-3579.6134	2097.89	1500	24.9692	7.7316
Top–Right	Leg 1	13.2692	DC: 1.1(D+0.25L)+BSE-1EY	742.597	3671.9421	2138.36	1500	24.9692	7.7316
Bottom–Left	Leg 1	13.3705	DC: 1.1(D+0.25L)+BSE-1EY	764.315	-6280.4804	3297.17	1500	25.0705	7.7316
Botttom–Right	Leg 1	13.3705	DC: 1.1(D+0.25L)+BSE-1EY	764.315	6090.114	3213.73	1500	25.0705	7.7316

ETABS 23.1.1

SAMPLE CALC FOR PIER 7.1 DESIGN

ETABS Shear Wall Design

ACI 318-19 Pier Design

Pier Details

Story ID	Pier ID	Centroid X (in)	Centroid Y (in)	Length (in)	Thickness (in)	Height (in)	Pier Ductility	LLRF
3rd Floor	7.1	2082	1078.5	117	12	162	Special Structural Wall	0.526

Material Properties

E_c (lb/in²)	f'_c (lb/in²)	Lt.Wt Factor (Unitless)	f_y (lb/in²)	f_{ys} (lb/in²)
3604996.5	7500	1	75000	75000

Design System Parameters

ρ	S_{DS}	Ω_0	SDC	Type
1	0.5	2.5	D	Special Structural Wall

Design Code Parameters

ϕ_T	ϕ_C	ϕ_V	ϕ_V (Seismic)	IP_{MAX}	IP_{MIN}	P_{MAX}
2	2	2	2	0.04	0.0025	0.8

Pier Leg Location, Length and Thickness

Station Location	ID	Left X_1 in	Left Y_1 in	Right X_2 in	Right Y_2 in	Length in	Thickness in	Pier A_g in²
Top	Leg 1	2082	1020	2082	1137	117	12	1440
Bottom	Leg 1	2082	1020	2082	1137	117	12	1440

Flexural Design for P, M_3 and M_2

Flexural is Ok

Station	D/C	Flexural	P_u kip	M_{u2} kip-ft	M_{u3} kip-ft
Top	0.118	DC: 0.9D+BSE-1EY	360.302	0.5841	3447.9679
Bottom	0.194	DC: 0.9D+BSE-1EY	378.071	11.346	5155.3018

Axial Force and Minor Moment Factors

Station Location	C_m Factor Unitless	δ_{ns} Factor Unitless	β_{dns} Unitless	EI_{eff} lb-ft²	P_c kip
Top Minor Bend(M2)	0.620593	1	1	84356918.1	4568.282
Bottom Minor Bend(M2)	0.620593	1	1	84356918.1	4568.282

Shear Design Capacity Parameters

Station Location	ID	Shear Combo	$M_{pr,major}$ kip-ft	Ω_v Unitless	ω_v Unitless	$\Omega_v * \omega_v$ Unitless
Top	Leg 1	DC: 1.1(D+0.25L)+BSE-1EY	0	1	1	1
Bottom	Leg 1	DC: 1.1(D+0.25L)+BSE-1EY	0	1	1	1

Shear Design (Part 1 of 2)

Station Location	ID	Rebar in²/ft	Shear Combo	P_u kip	M_u kip-ft	V_u kip	$V_{u,design}$ kip	h_w/l_w Unitless	α_c Unitless
Top	Leg 1	0.36	DC: 1.1(D+0.25L)+BSE-1EY	468.487	3445.4445	177.858	177.858	14.615385	2
Bottom	Leg 1	0.36	DC: 1.1(D+0.25L)+BSE-1EY	490.205	5167.4998	177.858	177.858	14.615385	2

Shear Design (Part 2 of 2)

ϕV_c kip	ϕV_n kip
486.36	1012.86
486.36	1012.86

Shear is Ok

Boundary Element Check (ACI 18.10.6.3, 18.10.6.4)

Station Location	ID	Edge Length (in)	Governing Combo	P_u kip	M_u kip-ft	Stress Comp lb/in²	Stress Limit lb/in²	C Depth in	C Limit in
Top–Left	Leg 1	12.4262	DC: 1.1(D+0.25L)+BSE-1EY	557.166	-3410.3946	1891.65	1500	24.1262	8.6337
Top–Right	Leg 1	12.4262	DC: 1.1(D+0.25L)+BSE-1EY	557.166	3445.4445	1907.01	1500	24.1262	8.6337
Bottom–Left	Leg 1	12.5249	DC: 1.1(D+0.25L)+BSE-1EY	578.884	-5078.2269	2638.14	1500	24.2249	8.6337
Botttom–Right	Leg 1	12.5249	DC: 1.1(D+0.25L)+BSE-1EY	578.884	5167.4998	2677.27	1500	24.2249	8.6337

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CONSULTING ENGINEERS

Title | Date | Job No.

Subject | By | Sheet

_ The concrete transfer beam under the concrete moment frame without the vertical continuity to the foundation is recognized as critical structural condition. Several transfer beams will be checked to satisfy **Immediate Occupancy** Performance Objective under ASCE 41-17 for the **BSE-1E** Hazard Level.
_ Transfer beam will be checked under **force-controlled** action. Per ASCE 41-17 Sec 7.5.2.2.2:

$kQ_{CL} > Q_{UF}$

while:

f'cL = 4000psi (lower bound)
fyL = 60000psi (lower bound)
Per Sec 6.2.4, k is taken as 1.0
$Q_{UF} = Q_G \pm lamdaQ_E/(C_1C_2J)$
Lamda = 1.3 (for LS & IO)
C_1 = 1 (for period greater than 1.0s)
C_2 = 1 (for period greater than 0.7s)
J = 1 (for IO)
$Q_G = 1.1(Q_D+Q_L)$ & $Q_G = 0.9(Q_D)$

_ Along the transfer beam, all beam at second floor are checked also under Force-controlled action to meet Immediate Occupancy.
_ See attached for detail calculation.

_ The concrete transfer beam under the concrete moment frame without the vertical continuity to the foundation is recognized as critical structural condition. Several transfer beams will be checked to satisfy **Life Safety** Performance Objective under ASCE 41-17 for the **BSE-2E** Hazard Level.
_ Transfer beam will be checked under **force-controlled** action. Per ASCE 41-17 Sec 7.5.2.2.2:

$kQ_{CL} > Q_{UF}$

while:

f'cL = 4000psi (lower bound)
fyL = 60000psi (lower bound)
Per Sec 6.2.4, k is taken as 1.0
$Q_{UF} = Q_G \pm lamdaQ_E/(C_1C_2J)$
Lamda = 1.3 (for LS & IO)
C_1 = 1 (for period greater than 1.0s)
C_2 = 1 (for period greater than 0.7s)
J = 2 (high seismicity)
$Q_G = 1.1(Q_D+Q_L)$ & $Q_G = 0.9(Q_D)$

_ Along the transfer beam, all beam at second floor are checked also under Force-controlled action to meet Immediate Occupancy.
_ See attached for detail calculation.

Floor Plan Redacted

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JOB TITLE SACRAMENTO JAIL SEISMIC EVALUATION

JOB NO. XX.XXXX.S.XX SHEET NO.
CALCULATED BY MR DATE 6/10/2026

	BSE-1E (IO)							X = 1.3	J= 1.0	C1C2 = 1				BSE-2E (LS)		X = 1.3	J= 2.0	
		f'c	4000	fyt	60000			Demand		Capacity		DCR		Demand		Capacity		
Beam	BM Tag	bw	d	Av	s	Vc	Vs	Vu (kips)	Mu (kip-ft)	V (kips)	M (kips)	Shear	Flexural	Vu (kips)	Mu (kip-ft)	V (kips)	M (kips)	
B231	2B-8	54	81	2.12	12	553	859	-1043.6	-5079.5	1412	15390	0.57	0.33	-956.756	-4731.0893	1411.872	15390	
B232	2B-6	54	81	2.12	12	553	859	2539.7	-6587.6	1412	4326	1.80	1.52	2312.125	-6035.2113	1411.872	4326	
B227	2B-14	54	81	2.12	9	553	1145	-1048.5	8959.9	1698	15350	0.62	0.58	-909.698	8052.4946	1698.072	15350	
B247	2B-32A/2B-32B/2B-34	84	81	2.74	9	861	1480	-1805.6	10647.9	2340	8010	0.77	1.33	-1697.35	9804.5855	2340.245	8010	
B322	2B-164/2B-166/2B-168	84	81	1.86	6	861	1507	-2427.1	5783.5	2367	9839	1.03	0.59	-2233.12	-5380.5301	2367.245	9839	
B384	2B-186	72	81	1.86	10	738	904	1335.8	4335.7	1642	16700	0.81	0.26	1236.045	3932.2805	1641.656	16700	
B282	2B-147	48	81	1.5	9	492	810	-831.3	4281.7	1302	6700	0.64	0.64	-755.086	3951.2688	1301.797	6700	
B718	2B-153/2B-155	120	81	2.74	6	1229	2219	2204.3	9539.6	3449	15351	0.64	0.62	2065.691	8828.195	3448.894	15351	
B312	2B-97	72	81	2.07	8	738	1258	2348.2	5849.7	1995	7384	1.18	0.79	2148.051	5365.3414	1995.221	7384	
B308	2B-119	48	81	1.24	7	492	861	2483.8	5620.9	1353	4940	1.84	1.14	2236.418	5053.2234	1352.712	4940	
B319	2B-77	48	81	1.24	10	492	603	-1581.7	6305.1	1094	8557	1.45	0.74	-1450.23	5774.9654	1094.437	8557	
B388	2B-186	72	81	1.86	7	738	1291	1806.9	11034.3	2029	16686	0.89	0.66	1662.607	10144.3133	2029.068	16686	
B371	2B-127	56	81	1.5	9	574	810	1408.8	-3649.9	1384	5227	1.02	0.70	1271.649	-3210.5019	1383.764	5227	
B373	2B-133/2B-135	24	81	0.62	6	246	502	-449.856	3808.2964	748	1855	0.60	2.05	-390.878	3422.3315	748.0987	1855	
B385	2B-130	48	81	1.5	9	492	810	449.6	4968.4	1302	4917	0.35	1.01	399.292	4365.9056	1301.797	4917	
B386	2B-130	48	81	1.5	9	492	810	-477.2	4521.4	1302	4917	0.37	0.92	-420.252	3984.3335	1301.797	4917	
B360	2B-118	48	81	0.62	12	492	251	-577.5	3327.1	743	2492	0.78	1.34	-513.025	2881.6097	742.8974	2492	
B365	2B-161/163	56	81	1.5	6	574	1215	-1139.1	2430.4	1789	7355	0.64	0.33	-1025.58	2168.0933	1788.764	7355	
B345	2B-104/2B-106	56	81	1.5	9	574	810	-810.4	2411.2	1384	6152	0.59	0.39	-1025.58	2177.0442	1383.764	6152	
B306	2B-82	48	81	1.24	12	492	502	822.5	6819.7	994	7920	0.83	0.86	717.76	5903.9857	993.9974	7920	
B307	2B-86	48	81	1.5	9	492	810	798.8	5574.6	1302	7920	0.61	0.70	689.056	4792.8179	1301.797	7920	
B224	2B-111	48	81	1.24	7	492	861	747.8	4699.3	1353	6143	0.55	0.76	652.668	4106.1375	1352.712	6143	
B229	2B-62/2B-64/2B-66	72	81	2.12	8	738	1288	-1287.1	3322.6	2026	7371	0.64	0.45	-1201.96	3040.2093	2025.596	7371	
B278	2B-50	60	81	1.5	5	615	1458	922.5	-5300.3	2073	7376	0.45	0.72	807.29	-4604.2302	2072.747	7376	
B283	2B-81	60	81	0.88	12	615	356	-1972.7	-6762.7	971	4958	2.03	1.36	-1797.19	-5878.7652	971.1468	4958	
B286	2B-98	59	81	1.5	9	605	810	-854.9	3504.2	1415	6158	0.60	0.57	-779.938	3169.0943	1414.501	6158	
B275	2B-100	59	81	1.5	9	605	810	-1048.5	1989.5	1415	6158	0.74	0.32	-955.111	1771.4749	1414.501	6158	
B301	2B-56	60	81	1.5	12	615	608	-924.0	5913.2	1222	7958	0.76	0.74	-955.111	5102.1152	1222.247	7958	

C1C2 = 1			
DCR			
Shear	Flexural	Notes	
0.68	0.31	check negative moment	RELEASE MAYBE
1.64	**1.40**	check negative moment	RELEASE MAYBE
0.54	0.52		
0.73	**1.22**	2B-32B max moment	NS WALL SITTING ON BEAM CAUSING HIGH MOMENT
0.94	0.55	2B-168 max moment, check neg	
0.75	0.24		
0.58	0.59		
0.60	0.58		
1.08	0.73		RELEASE MAYBE
1.65	1.02		RELEASE MAYBE
1.33	0.67		SPLIT? INTERACTING W/ CROSS OVERS, ALSO WALL DIRECTLY ABV
0.82	0.61		
0.92	0.61	check negative moment	SPLIT? INTERACTING W/ CROSS OVERS
0.52	**1.84**		SPLIT? INTERACTING W/ CROSS OVERS
0.31	0.89		WALLS DIRECTLY ABOVE NS DIRECTION
0.32	0.81		WALLS DIRECTLY ABOVE NS DIRECTION
0.69	**1.16**		MODEL NOT ACCURATE TO DETAILING
0.57	0.29		
0.74	0.35		
0.72	0.75		WALLS DIRECTLY ABOVE NS DIRECTION
0.53	0.61		WALLS DIRECTLY ABOVE NS DIRECTION
0.48	0.67		WALLS DIRECTLY ABOVE EW DIRECTION
0.59	0.41		
0.39	0.62	check negative moment	WALL ON 2B-77 OVERTURNING ON 2B-50
1.85	**1.19**	check negative moment	RELEASE AT END
0.55	0.51		
0.68	0.29		
0.78	0.64		MODEL NOT ACCURATE TO DETAILING

_ The diaphragms are evaluated to satisfy **Immediate Occupancy** Performance Objective under ASCE 41-17 for the **BSE-1E** Hazard Level.

_ The diaphragms will be checked under **force-controlled** action. Per ASCE 41-17 Sec 7.5.2.2.2:
kQCL > QUF
while:
f'cL = 4000psi (lower bound)
fyL = 60000psi (lower bound)
Per Sec 6.2.4, k is taken as 1.0
QUF = QG ± lamdaQE/(C1C2J)
Lamda = 1.3 (for LS & IO)
C1 = 1 (for period greater than 1.0s)
C2 = 1 (for period greater than 0.7s)
J = 1 (for IO)
QG = 1.1(QD+QL) & QG = 0.9(QD)

_ The diaphragms are evaluated to satisfy **Life Safety** Performance Objective under ASCE 41-17 for the **BSE-2E** Hazard Level.

_ The diaphragms will be checked under **force-controlled** action. Per ASCE 41-17 Sec 7.5.2.2.2:
kQCL > QUF
while:
f'cL = 4000psi (lower bound)
fyL = 60000psi (lower bound)
Per Sec 6.2.4, k is taken as 1.0
QUF = QG ± lamdaQE/(C1C2J)
Lamda = 1.3 (for LS & IO)
C1 = 1 (for period greater than 1.0s)
C2 = 1 (for period greater than 0.7s)
J = 2 (for LS)
QG = 1.1(QD+QL) & QG = 0.9(QD)

DIAPHRAGM CAPACITY PER ACI 318:

Diaphragm Capacity:

			short span		long span		pt	Vn
		thk	**top**	**bot**	**top**	**bot**		
AA	2way	8	#4 16	#4 12OC	#4 24	#4 12	0.0031	30.143
BB	2way	8	#4 20	#4 24OC	#4 24	#4 18	0.0028	28.143
CC	2way	8	#4 24OC	#4 24OC	#4 24	#4 24	0.0021	24.143
DD	2way	8	#4 18OC	#4 24OC	#4 24	#4 24	0.0021	24.143
EE	2way	8	#4 24OC	#4 24OC	#4 24	#4 24	0.0021	24.143
FF	2way	8	#4 12OC	#4 6OC	#4 24	#4 12	0.0042	36.143
		thk	**top**	**bot**	**trasv**			
A	1way	8	#4 24OC	#4 12OC	#4 @ 18 TB		0.0028	28.143
B	1way	8	#4 24OC	#5 12OC	#4 @ 18 TB		0.0028	28.143
C	1way	8	#4 24OC	#5 8OC	#4 @ 18 TB		0.0028	28.143
D	1way	8	#4 24OC	#5 6OC	#4 @ 18 TB		0.0028	28.143
E	1way	8	#4 24OC	#54OC	#4 @ 18 TB		0.0028	28.143

Diaphragm Capacity:

			short span		long span		pt	Vn
		thk	**top**	**bot**	**top**	**bot**		
AA	2way	8	#4 16	#4 12OC	#4 24	#4 12	0.0031	30.143
BB	2way	8	#4 20	#4 24OC	#4 24	#4 18	0.0028	28.143
CC	2way	8	#4 24OC	#4 24OC	#4 24	#4 24	0.0021	24.143
DD	2way	8	#4 18OC	#4 24OC	#4 24	#4 24	0.0021	24.143
EE	2way	8	#4 24OC	#4 24OC	#4 24	#4 24	0.0021	24.143
FF	2way	8	#4 12OC	#4 6OC	#4 24	#4 12	0.0042	36.143
		thk	**top**	**bot**	**trasv**			
A	1way	8	#4 24OC	#4 12OC	#4 @ 18 TB		0.0028	28.143
B	1way	8	#4 24OC	#5 12OC	#4 @ 18 TB		0.0028	28.143
C	1way	8	#4 24OC	#5 8OC	#4 @ 18 TB		0.0028	28.143
D	1way	8	#4 24OC	#5 6OC	#4 @ 18 TB		0.0028	28.143
E	1way	8	#4 24OC	#54OC	#4 @ 18 TB		0.0028	28.143
3S		8						

Pier	**Width Bottom**	k	k/ft	BSE-1EX	k	k/ft	BSE-1EY		
	in	V2	VE	xQE/C1C2J	V2	VE	xQE/C1C2J	Max	DCR
C.6	120.00	305.13	30.51	39.67	171.83	17.18	11.17	39.67	1.41
C.7	192.00	447.84	27.99	36.39	346.39	21.65	14.07	36.39	1.29
C.8	414.00	1354.12	39.25	51.02	201.83	5.85	3.80	51.02	1.81
C.9	174.00	268.31	18.50	24.05	59.64	4.11	2.67	24.05	0.85
C.10	378.00	942.80	29.93	38.91	112.32	3.57	2.32	38.91	1.38
C.11	144.00	457.00	38.08	49.51	37.17	3.10	2.01	49.51	1.76
C.12	468.00	1201.88	30.82	40.06	300.74	7.71	5.01	40.06	1.42
C.13	114.00	412.27	43.40	56.42	127.57	13.43	8.73	56.42	2.00
C.14	408.00	451.89	13.29	17.28	325.28	9.57	6.22	17.28	0.61
C.15	72.00	119.01	19.84	25.79	19.49	3.25	2.11	25.79	0.92

B7.3	354.00	610.26	20.69	26.89	680.60	23.07	15.00	26.89	0.96
B7.4	120.00	323.62	32.36	42.07	162.63	16.26	10.57	42.07	1.49
B7.5	654.00	1151.84	21.13	27.48	837.68	15.37	9.99	27.48	0.98
B7.6	210.00	225.97	12.91	16.79	92.60	5.29	3.44	16.79	0.60
B7.7	546.00	794.11	17.45	22.69	178.52	3.92	2.55	22.69	0.81
B7.8	72.00	138.81	23.14	30.08	46.52	7.75	5.04	30.08	1.07
B7.9	270.00	400.98	17.82	23.17	99.20	4.41	2.87	23.17	0.82
A7.2	1071.00	850.89	9.53	12.39	286.22	3.21	2.08	12.39	0.44
A7.3	234.00	183.94	9.43	12.26	122.83	6.30	4.09	12.26	0.44
-A1.1	1176.00	304.19	3.10	4.04	245.61	2.51	1.63	4.04	0.14
-A2.1	1548.00	150.35	1.17	1.52	253.27	1.96	1.28	1.52	0.05
G1.1	600.00	1332.13	26.64	34.64	306.18	6.12	3.98	34.64	1.23
G1.2	468.00	722.07	18.51	24.07	195.24	5.01	3.25	24.07	0.86
G1.3	126.00	38.96	3.71	4.82	14.56	1.39	0.90	4.82	0.17
G8.1	498.00	1545.33	37.24	48.41	365.68	8.81	5.73	48.41	1.72
H.1	351.00	374.10	12.79	16.63	176.41	6.03	3.92	16.63	0.59
1.1	396.00	338.01	10.24	13.32	316.80	9.60	6.24	13.32	0.47
12.1	408.00	456.11	13.41	17.44	454.63	13.37	8.69	17.44	0.62
14.1	702.00	395.32	6.76	8.78	1147.35	19.61	12.75	12.75	0.45
16.1	396.00	476.93	14.45	18.79	588.77	17.84	11.60	18.79	0.67
39.2	744.00	479.08	7.73	10.05	2265.83	36.55	23.75	23.75	0.84
48.1	276.00	196.83	8.56	11.13	464.49	20.20	13.13	13.13	0.47
48.3	486.00	241.21	5.96	7.74	1318.62	32.56	21.16	21.16	0.75
55.1	210.00	408.91	23.37	30.38	316.27	18.07	11.75	30.38	1.08
55.3	486.00	246.08	6.08	7.90	961.83	23.75	15.44	15.44	0.55
74.1	27.00	13.98	6.21	8.08	6.72	2.99	1.94	8.08	0.29
74.2	186.00	34.30	2.21	2.88	131.95	8.51	5.53	5.53	0.20
74.3	114.00	155.57	16.38	21.29	93.94	9.89	6.43	21.29	0.76
8.2	27.96	14.73	6.32	8.22	38.26	16.42	10.67	10.67	0.38
8.3	186.00	110.54	7.13	9.27	164.90	10.64	6.92	9.27	0.33
8.4	430.00	344.41	9.61	12.49	1224.93	34.18	22.22	22.22	0.74
9.1	27.96	2.29	0.98	1.28	9.85	4.23	2.75	2.75	0.10
9.2	186.00	152.63	9.85	12.80	471.70	30.43	19.78	19.78	0.66
9.3	256.00	221.04	10.36	13.47	617.84	28.96	18.82	18.82	0.62

10.2	342.00	292.65	10.27	13.35	776.04	27.23	17.70	17.70	0.59
10.3	256.00	357.31	16.75	21.77	839.53	39.35	25.58	25.58	0.85
96.2	225.00	194.26	10.36	13.47	680.69	36.30	23.60	23.60	0.84
11.2	303.96	343.71	13.57	17.64	607.71	23.99	15.59	17.64	0.63
11.3	432.00	355.94	9.89	12.85	620.28	17.23	11.20	12.85	0.46
11.4	213.00	211.58	11.92	15.50	358.37	20.19	13.12	15.50	0.55
11.5	156.00	21.16	1.63	2.12	38.90	2.99	1.95	2.12	0.08
116.1	156.00	94.97	7.31	9.50	74.75	5.75	3.74	9.50	0.34
116.2	105.00	21.07	2.41	3.13	27.25	3.11	2.02	3.13	0.11
12.1.1	1095.96	351.00	3.84	5.00	466.53	5.11	3.32	5.00	0.18
52.1	240.00	135.52	6.78	8.81	434.40	21.72	14.12	14.12	0.50
B3.1	240.00	209.78	10.49	13.64	47.01	2.35	1.53	13.64	0.48
A2.1	786.00	529.82	8.09	10.52	205.04	3.13	2.03	10.52	0.37
CR.1	275.04	102.91	4.49	5.84	73.52	3.21	2.08	5.84	0.21
CR.2	312.19	62.21	2.39	3.11	67.67	2.60	1.69	3.11	0.11
CR.3	403.44	280.12	8.33	10.83	202.39	6.02	3.91	10.83	0.38
M13.3	126.00	484.80	46.17	60.02	179.99	17.14	11.14	60.02	2.13
M2.2	120.00	42.56	4.26	5.53	53.35	5.34	3.47	5.53	0.20
D3.2	144.00	445.55	37.13	48.27	107.84	8.99	5.84	48.27	1.72
G5.1	84.00	44.79	6.40	8.32	26.58	3.80	2.47	8.32	0.30
7.10	117.00	69.77	7.16	9.30	106.40	10.91	7.09	9.30	0.33
6.10	117.00	77.83	7.98	10.38	119.57	12.26	7.97	10.38	0.37
45.10	72.00	63.13	10.52	13.68	293.51	48.92	31.80	31.80	1.13
2.10	36.00	55.50	18.50	24.05	129.40	43.13	28.04	28.04	1.00
NG.1	144.00	241.52	20.13	26.16	730.41	60.87	39.56	39.56	1.41

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Title | Date | Job No.

Subject | By | Sheet

_ The piles are evaluated to satisfy **Immediate Occupancy** Performance Objective under ASCE 41-17 for the **BSE-1E** Hazard Level.

_ The piles will be checked under **deformation-controlled** action. Per ASCE 41-17 Sec 7.5.2.2.1:
mkQCE > QUD
while:
m: component capacity modification factor for beam/column/joint (see below)
f'cE = 5000psi x 1.5 = 7500 (expected strength per Table 10.1)
fyE = 60000psi x 1.25 = 75000 (expected strength per Table 10.1)
Per Sec 6.2.4, k is taken as 1.0
QUD = QG ± QE
QG = 1.1(QD+QL) & QG = 0.9(QD)

_m-factor = 2 per 8.4.5.1.1 fixed-base assumption

_ A liquefaction-affected model was not included in this study but recommended to be conducted.

_Perscriptive expected capacities for vertical load Qc =3*Qallow

_Retaining Walls are checked per section 8.6 with seismic loading of soil.

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Title Date Job No.

Subject By Sheet

Page 8
August 27, 1982
L & a No. 82-251

Loading Condition	Minimum Factor of Safety	Allowable Load per Pile-tons 12" square	14" square	16" square
Axial Downward-				
Dead Load	3	87	107	133
Dead plus Live Load	2	130	160	200
Total Load, including seismic or wind	1.5	174	214	266
Axial Uplift	2	40	50	60
Lateral	2	7	12	16

The indicated uplift capacities are based upon the assumption that the piles are properly reinforced to transfer pullout forces to the pile tips.

The lateral load capacities assume horizontal movement of the pile tops on the order of ¼ inch. Under that condition of lateral movement, an allowable equivalent fluid pressure of 150 pounds per cubic foot acting along one face of each pile cap and the connecting grade beams may be used in design computations. Pile fixity may be assumed at a depth of 4 feet below the top of each pile.

The above allowable pile capacities are recommended with the stipulation that a pile load testing program be performed immediately prior to the commencement of production pile driving and that a representative of our firm be present throughout production pile driving operations to record the driving resistance of each pile.

The weight of pile cap concrete extending below grade and the weight of each pile may be disregarded in determinations of the net compression load transmitted to the supporting soil.

Post-construction settlements of properly installed piles are anticipated to be negligible.

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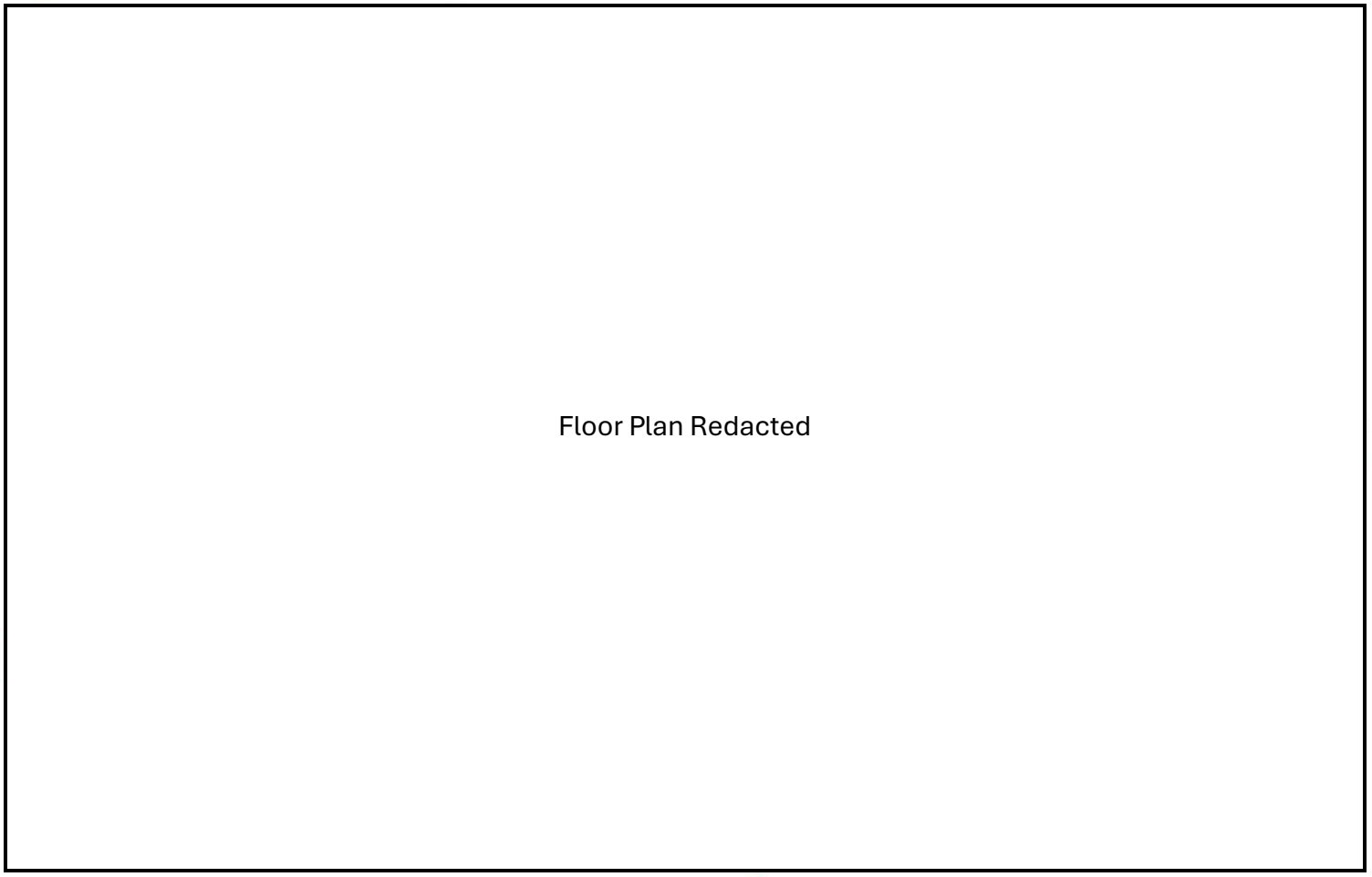
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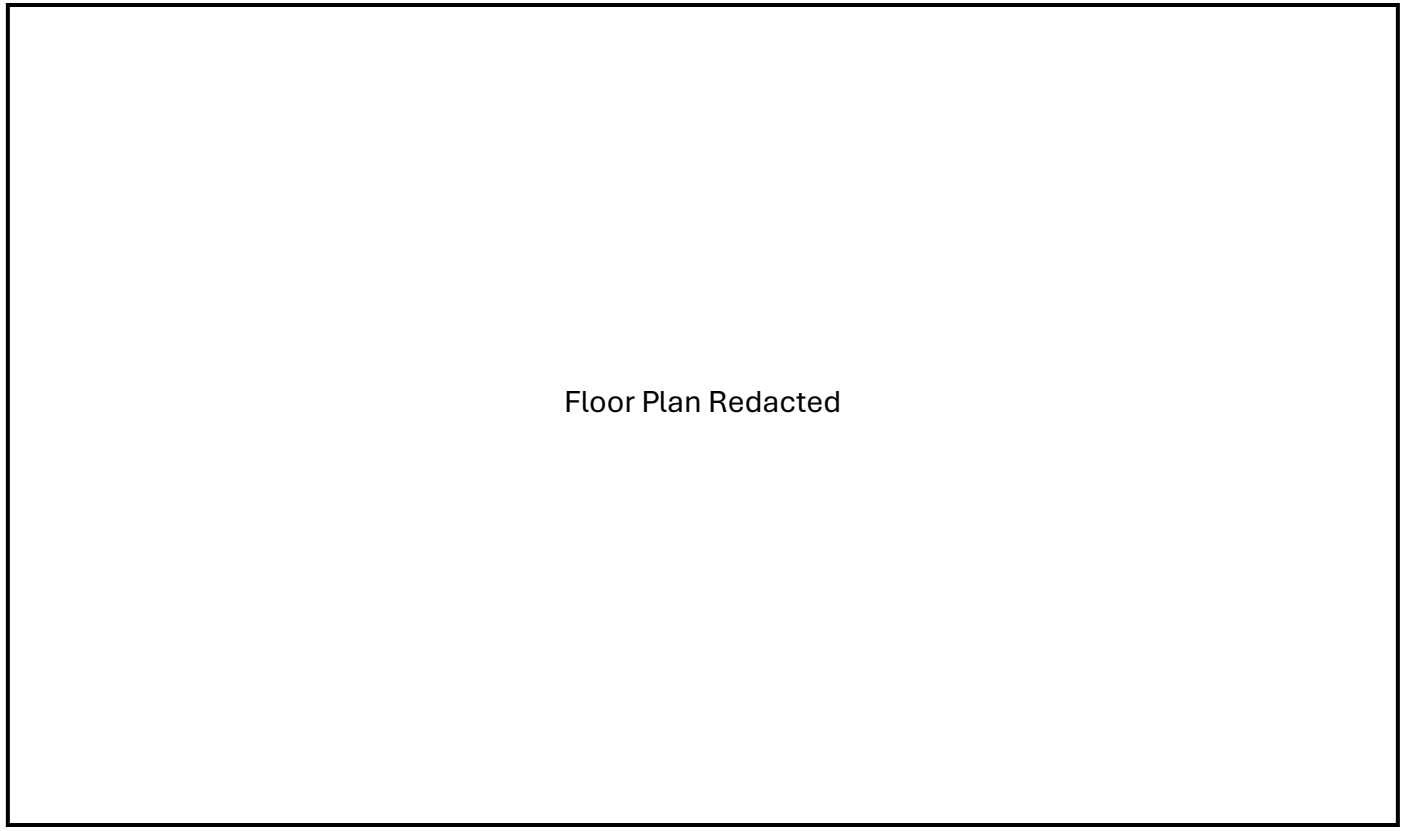
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