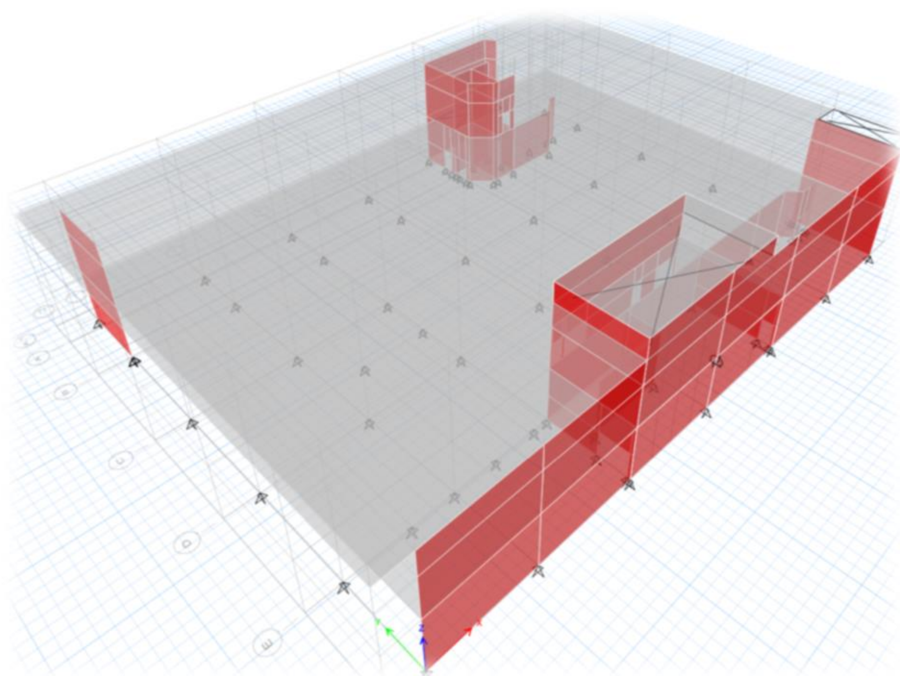
CIELO STRUCTURES, INC.

Seismic Assessment of the San Bernardino County Building Located at 303 West 5th Street San Bernardino, California

March 4, 2025 – DRAFT



Prepared for :
Dahl, Taylor & Associates, Inc.
2960 Daimler Street
Santa Ana, CA 92805

CIELO STRUCTURES, INC.

March 4, 2025

Mr. Stephen Vu
Project Manager
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Subject: ***Transmittal of Report Entitled: "Seismic Assessment of the San Bernardino County Building Located at 303 West 5th Street, San Bernardino, CA – March 4, 2025 - DRAFT"***

Dear Mr. Vu:

The attached report summarizes our findings and recommendations for the preliminary seismic analysis and seismic strengthening of the building located at 303 West 5th Street in San Bernardino, California.

Our services included administering a material testing program, which included destructive and non-destructive testing in the building, general survey of the existing building construction, preliminary seismic assessment and computer modeling and development of preliminary seismic strengthening measures for the building in order to reduce the risk of structural damage and potential building collapse in the event of a major earthquake near the site. A preliminary seismic structural retrofit construction cost opinion is also provided.

We appreciate the opportunity to work with you and the San Bernardino County on this project. If you should have any questions or require any further information, please do not hesitate to contact us.

Sincerely yours,
Cielo Structures, Inc.

Daniel J. Dopudja, P.E., S.E.
Principal Engineer

Enclosures: *As Noted*

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Table of Contents

Table of Contents (Continued)

Table of Contents (Continued)

Table of Contents (Continued)

Page

Tables

Figures

Table of Contents (Continued)

Executive Summary

A seismic evaluation and strengthening investigation were performed by Cielo Structures, Inc. (Cielo Structures) for the 303 West 5th Street Building in San Bernardino, California. The purpose of this project was to perform a seismic evaluation and analysis of the subject building and to develop retrofit concepts in order to meet the ASCE/SEI 41-17 Standard (Reference 1) for the "Basic Performance Objective for Existing Buildings" (BPOE) for both "Life-Safety" and "Collapse Prevention" performance levels.

E.1 Background

The 303 West 5th Street County Building is a two-story reinforced Concrete Masonry Unit (CMU) shear wall structure with parking located at ground level footprint and interior offices and lobby space located at first and second floors. Based upon real estate records found via public websites and San Bernardino County portal, the original building appears to have been constructed circa late 1970s or early 1980s and is currently unoccupied. Previously, the building was utilized as an office building for the State of California Department of Corrections and contained private offices along with meeting rooms and ground-level parking.

The building was likely designed utilizing the 1976 or 1979 edition of the Uniform Building Code (Reference 2), which is assumed based on the vintage of the building. Note that building codes have changed significantly since the original design, reflecting advancements in structural engineering and lessons learned from previous earthquakes. Important gravity and lateral load resisting features have changed for many of the structural and non-structural components in buildings similar to the subject building.

If the San Bernardino County intends to meet the “Basic Performance Objective for Existing Buildings” (BPOE) for both “Life-Safety” and “Collapse Prevention” performance levels, the 303 West 5th Street Building will require strengthening. Chapter 4 presents the results of our seismic evaluation. The proposed retrofit options are described in Chapter 5. A summary of these chapters is presented in the following sections.

E.2 Seismic Performance Objective

New buildings designed to the latest building codes are expected to provide some “standard” level of performance in order to protect life and/or minimize business interruption. Existing buildings, when compared to the latest building codes, often do not meet this “Life-Safety” level of performance. However, there are other industry standards that can be used to evaluate existing buildings to meet desired performance goals. One tool for selecting desired performance levels for a building is to utilize the various performance levels outlined in ASCE/SEI 41-17 *“Seismic Evaluation and Retrofit of Existing Buildings”* for the “Basic Performance Objective for Existing Buildings”.

Leadership for the San Bernardino County is interested in evaluating the 303 West 5th Street Building for the “Basic Performance Objective for Existing Buildings” which includes “Life-Safety” performance level (at 225-year-event) and “Collapse Prevention” performance level (at 975-year-event).

ASCE/SEI 41-17 defines Life-Safety as follows:

- **Life-Safety (LS):** The post-earthquake damage state in which significant damage to the structure has occurred, but some margin against either partial or total structural collapse remains. Some structural elements and components are severely damaged, but this has not resulted in large falling debris hazards, either within or outside the building. Injuries may occur during the earthquake; however, the overall risk to life-threatening injury as a result of structural damage is expected to be low. It should be possible to repair the structure; however, for economic reasons this may not be practical. While the damaged structure is not an imminent collapse risk, it would be prudent to implement structural repairs or install temporary bracing prior to re-occupancy.

- **Collapse Prevention (CP):** The post-earthquake damage state in which the building is on the verge of partial or total collapse. Substantial damage to the structure has occurred, potentially including significant degradation in the stiffness and strength of the lateral-force-resisting system, large permanent lateral deformation of the structure, and – to a more limited extent – degradation in vertical-load-carrying capacity. However, all significant components of the gravity-load-resisting system must continue to carry their gravity loads. A significant risk of injury caused by falling hazards from structural debris might exist. The structure might not be technically practical to repair and is not safe for re-occupancy because aftershock activity could induce collapse.

E.3 Analysis Results

Cielo Structure's three-dimensional computer analysis of the building shows that the existing lateral force-resisting system for the building is severely overstressed when compared to the requirements of the performance objectives (discussed in Section E.2 and Chapter 3). Most of the discovered deficiencies can be attributed to the use of older design and construction practices, which were based upon less stringent detailing requirements. Further information regarding the analysis results is presented in Chapter 4.

Based on the severe overstresses of the existing masonry shear walls, second floor and roof level diaphragms, it is expected that the building will experience significant damage during a large seismic event and the resulting damage may result in life-threatening injury and significant business interruption.

E.4 Retrofit Options

The proposed building strengthening was developed to meet the ASCE/SEI 41-17 Standard and current 2022 California Building Code (Reference 3) for detailing the new lateral-force-resisting systems. Cielo Structures evaluated two conventional concrete shear wall lateral-force-resisting systems. These systems were chosen due to the existing masonry construction of the building and the need to increase the stiffness of the building's new lateral-force-resisting system in order to reduce overstresses at existing masonry shear walls.

E.4.1 Retrofit Schemes

Multiple retrofit options were investigated using ETABS computer modeling software (Reference 10) before two options were selected. Use of new concrete shear walls and diaphragm strengthening systems were chosen due to the existing masonry and steel construction of the building and the need to significantly reduce masonry shear wall and diaphragm overstresses. Option No. 1 and Option No. 2 are similar in-nature; however, each option reflects slightly different configuration of shear walls and locations. The following potential strengthening concepts were evaluated and are being proposed:

- Addition of new reinforced concrete shear walls at both the interior and exterior of the building between existing masonry walls.
- Addition of new foundation system and concrete caissons below the new reinforced concrete shear walls.

- Addition of new reinforced concrete overlays on select interior masonry walls.
- New horizontal steel bracing elements and connections below the second floor and roof levels.
- Additional drag elements and tie connections at the second floor and roof level.
- New out-of-plane connections at select masonry walls at northwest and southeast corners of the building.
- New steel strong-back near the Lobby stairway.

Chapter 1 - Introduction

This report presents the results of Cielo Structures, Inc. (Cielo Structures) seismic evaluation of a two-story, masonry shear wall building located at 303 West 5th Street in San Bernardino, California. The currently unoccupied building was originally constructed circa late 1970s or early 1980s (based on recent real estate internet searches) and was used as the State of California Department of Corrections. No original record drawings of the building are available but only architectural, mechanical, electrical and plumbing remodel drawings from a 2001 tenant improvement (Reference 13).

1.1 Purpose

The purpose of this investigation is to seismically evaluate and analyze the subject building's vertical and horizontal seismic-lateral-resisting-systems and components in order to verify if these elements meet the ASCE/SEI 41-17 Standard *"Seismic Evaluation and Retrofit of Existing Buildings"* (Reference 1) for the "Basic Performance Objective for Existing Buildings" (BPOE) for both "Life-Safety" and "Collapse Prevention" performance levels. In order to meet the BPOE performance objective, Cielo Structures utilized the evaluation and analysis principles outlined in ASCE 41-17 Standard, which is further discussed in Chapter 3.

If the building's seismic-lateral-resisting-systems and components are found to be deficient, various potential seismic retrofit solutions, along with associated retrofit construction cost estimates are prepared to mitigate the building deficiencies. In addition, Cielo Structures also coordinated and participated in a building survey and material testing program for the structure in order to better understand the in-situ strengths of the building materials.

The scope of work for the structural/seismic evaluation and analyses and materials testing program for the building consisted of the following tasks:

1.2 Scope of Work

Task 1 Prepared Materials Testing Program

1. Conducted site visit to review interior and exterior elements including finishes and constraints associated with preparing materials testing program (Appendix A).

2. Prepared a Materials Testing Program (MTP) utilizing the general guidelines from ASCE/SEI Standard 41-17 (*Seismic Evaluation and Retrofit of Existing Buildings*) for all undocumented items and material strengths.

   Material Testing Program (MTP) included quantity, location, and type of destructive and non-destructive testing. MTP included 11x17 mark-up of plans documenting location of samples to be removed from the building for testing as well as other destructive and non-destructive tests.

   Items included in the Materials Testing Program were as follows:

   - *Column, beam, slab, and wall in-situ compressive strengths*
   - *Column, beam, slab, and wall reinforcement size and spacing*
   - *Floor, roof, and wall thicknesses and assembly*
   - *Masonry verification*
   - *Exterior architectural cladding verification for thickness*
   - *Steel framing sizes and spacing*
   - *Foundation sizes*

3. Provided patch and repair recommendations (if required for key areas) for the building elements and incorporated in the MTP where destructive testing was performed.

4. Issued the final Materials Testing Program to Dahl Taylor and the Inspection Company.

5. Provided minor pre-coordination services related to the site visit with the testing agency regarding the Materials Testing Program and field constraints.

6. Reviewed Material Testing Program and the associated patch and repair requirements for the building elements with the Testing and Inspection Company (MTGL) prior to destructive and non-destructive testing.

   Third-party testing company services was by others. Due to County budgetary constraints, the following scope of work was proposed for the material testing and investigation services:

- *Schmidt Hammer testing of columns, walls, slabs, and beams for compression strength*
- *Concrete slab and wall coring and compression testing and patching with non-shrink grout*
- *GPR Scanning of columns, walls, slabs, and beams*
- *GPR Scanning to confirm footing size*

Materials testing and investigation of existing building elements can result in inconclusive results or low material strengths due to variability of material composition and original construction.

Task 2 Conducted Building Survey for Key Architectural and Structural Elements and Components

1. Conducted site visits to review architectural and structural elements, locations and configuration.

2. Performed as-required field measuring of the original building, to collect measurements and dimensions of the building layout, column, beam sizes, cladding, roof overhangs, etc.

   Field measuring and verification of existing elements was limited in nature. Further extensive field measuring and verification will be required if building structure is retrofitted. As-built drawings are not available for the building.

3. Provided part-time on-site field supervision of field material testing work performed under Task 3.

Task 3 Reviewed Third-Party MTGL Destructive and Non-Destructive Materials Testing Report & Issued Consolidated Material Testing Report

1. Reviewed MTGL Material Testing Report (Appendix A).

2. Coordinated and discussed material testing findings with MTGL.

3. Prepared a brief internal data analysis and summary of Materials Testing and Building Investigation Report for the non-destructive and destructive investigation services including findings and consolidate internal data summary with MTGL report.

   Internal data compilation and summary are required for column, wall, slab, and beam compressive strengths for mean and average strength and standard deviation for the building analysis. Issued consolidated testing report to Dahl Taylor and Associates.

Note: Materials testing, and investigation of existing building elements can result in inconclusive results or low material strengths due to variability of material composition and original construction methods or materials.

Task 4 Performed Structural Evaluation & Seismic Analyses, Identified Problem Areas, Developed Conceptual Retrofit Schemes, Prepared Opinion of Preliminary Construction Cost & Prepared Bridging Document for the Subject Building.

1. Performed a detailed review of the survey notes and test findings from Task 2 and 3 in order to become familiar with the lateral-force-resisting systems and strengths.

2. Performed structural analyses based on the Basic Performance Objective for Existing Buildings (BPOE) for "Life-Safety" and "Collapse Prevention" performance levels per ASCE/SEI Standard 41-17 (*Seismic Evaluation and Retrofit of Existing Buildings*). *ASCE/SEI Standard 41-17 contains provisions for the latest performance-based seismic rehabilitation methodology. This national consensus standard was developed from FEMA 356 (Pre-Standard and Commentary for the Seismic Rehabilitation of Buildings). The ASCE/SEI Standard 41-17 represents state-of-the-art knowledge in earthquake engineering and is a tool for the structural engineering profession to improve building performance in future earthquakes. It includes significant improvements in the current understanding of building behavior in earthquakes. The California Existing Building Code References ASCE 41-17.*

   Analyses of the building were performed based on a Risk Category II structure for as defined by the building code. This results in an important factor of 1.0 effectively. We evaluated the building deficiencies based upon ASCE 41-17 utilizing Risk Category II and *a Basic Performance Objective for Existing Buildings (BPOE) as follows: (BSE-1E-20% probability of exceedance in 50 years) "Life-Safety" performance level,* ***and*** *(BSE-2E-5% probability of exceedance in 50 years) "Collapse Prevention" performance level.*

"Life-Safety" is defined as the post-earthquake damage state in which significant damage has occurred, but some margin against either partial or total structural collapse remains. Injuries may occur during the earthquake; however, the overall risk of life-threatening injury is expected to be low. It should be possible to repair the building; however, for economic reasons this may not be practical. While the damaged building is not an imminent collapse risk, it would be prudent to implement structural repairs or install temporary bracing prior to re-occupancy.

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

Evaluation of gravity only framing such as metal deck, concrete slabs, beams, and columns is excluded.

Non-Structural components (ceilings, partition walls, MEP systems, etc.) were not evaluated since the building might undergo an extensive tenant renovation including walls, ceiling, mechanical, etc. Bracing these components and elements would be the responsibility of the Design-Build team using the current code. Interior and exterior architectural concrete cladding was not explicitly evaluated nor assessed but will be discussed in the seismic report, if applicable.

3. Developed 3-D ETABS (Reference 10) computer model of the building and performed engineering calculations to quantitatively

identify specific areas of seismic deficiency relative to code criteria identified above. Assessed strengths, weaknesses and deficiencies in the building's lateral force (earthquake) resisting system, paying particular attention to critical and major members and connections.

4. Tabulated seismic stresses and deflections for key elements of the lateral-force-resisting system. This included members in the roof system, second floor, walls, and foundations.

5. Recommended conceptual concepts for practical and cost-effective retrofit schemes from the seismic evaluation and analysis. The proposed retrofit schemes were conveyed by usage of sketched plans, elevations, and selected details.

   Two strengthening concepts are included.

6. Compliance to selected seismic design provisions contained in the 2022 Edition of the California Building Code (CEBC) (Reference 3) for the performance levels was utilized for the conceptual strengthening of the deficient elements identified in the building analyses phase.

   Note that the intent of the building code provisions is to attain a "Life-Safety" level of performance; however, no explicit "Life-Safety" criterion is defined in the building code. Instead, the building code uses an importance factor to attain different levels of performance.

7. Performed field walk-downs and discussed with Dahl Taylor & Associates where additional or supplementary structural reinforcement could be placed without adversely impacting the operations of the building.

8. Revised the structural analyses to reflect the conceptual seismic strengthening measures determined during the analyses, walkdowns and discussions.

9. Field verified the constructability of the updated/finalized proposed upgrade schemes.

   Major visible interferences, obstructions and items which may require relocation are noted. Also, problem areas which may require special construction techniques or scheduling are identified.

10. Developed a rough-order-of-magnitude construction cost for the proposed building retrofit concept.

    Since the two proposed retrofit concepts are similar in nature, one retrofit cost is presented.

11. Developed a final report of results and recommendations, including sketches of the finalized retrofit concepts and projected construction costs for the building retrofit.

12. Attended two progress conference calls with Dahl, Taylor and Associates, Inc. at 50% and 100% Evaluation Levels.

13. Attended **one** in-person meeting with the San Bernardino County and Dahl, Taylor & Associates to discuss the report, findings, and the County's preferred strengthening option.

14. Prepared a bridging document based upon County's preferred strengthening option.

    Cielo extracted several key components from the seismic report including the County's preferred strengthening option (including *diagrams and narrative from the Strengthening Chapter of the Seismic Report) and include in a separate bridging report.*

This bridging report or bridging document includes strengthening diagrams and pdf mark-ups as well as strengthening narrative/discussion. The document does NOT include CAD drawings or any additional figures beyond the seismic report. The bridging document is not as comprehensive as the seismic report but provides the Design-Build Team with a proposed strengthening concept containing figures and strengthening narrative.

Please note that much of the analysis performed above can be used to finalize the retrofit concepts (in case building retrofits are required), and to develop construction documents for submittal to the local Building Department and solicitation of contractor bids. Preparation of construction documents for the subject building is not included in the scope of work above. Preparation of a geotechnical report is also excluded.

Note that the preliminary work performed and shown in the report relates only to seismic improvements. No other improvements, e.g., upgrades of outdated systems such as electrical, ventilation, energy, roofing, etc. are included. Improvements associated with existing structural damage or deterioration; or asbestos, lead paint or other hazardous material evaluation/mitigation are not included. Provisions of the Americans with Disabilities Act (ADA) and any other building department requirements not relating to structural issues (e.g., fire code upgrades, parking surveys, etc.) are not addressed as part of this project. Cielo Structures attempts to minimize the impact of structural retrofits on existing architectural, electrical, mechanical, plumbing, HVAC, etc., systems.

1.3 Report Outline

Chapter 2 presents a description of the building and its gravity and lateral force-resisting system. This includes descriptions of geometry, overall dimensions, gravity loading and the make-up of the overall structural members in the building.

Chapter 3 explains the analysis procedures, evaluation criteria and field testing used as a basis for the seismic analysis of the building. Included is a discussion on the seismic performance of similar buildings during earthquakes.

Chapter 4 presents the findings and results of the structural evaluation and analysis of the building.

Chapter 5 presents conceptual strengthening options to satisfy the seismic retrofit criteria.

Chapter 6 presents a rough-order-of-magnitude cost estimate for the proposed seismic strengthening schemes.

Chapter 7 provides a list of selected references utilized in the preparation of the seismic analysis of the building.

Appendix A includes a copy of the results of the materials testing that was conducted for the building.

1.4 Unknowns

Recent advances in seismology, engineering and technologies such as Geographic Information Systems now provide decision makers with the tools to track catastrophic exposures as well as predict future losses. This approach is superior to traditional extrapolation of previous experience, since large catastrophic losses tend to occur with low frequency and in regions where recent rapid urban development does not provide a true historical measure of the current loss potential. However, even with these new tools to assess loss on a prospective basis, as opposed to a retrospective basis, a number of limitations and unknowns still exist. These include:

- **Building Damage.** Even with extensive claims data available from the 1994 Northridge earthquake in California and other more recent earthquakes, the lack of significant claims data (due to the rareness of these events) precludes better definition of damage as a function of ground motion. Therefore, even for very similar structures located next to each other, it is not possible to fully explain the variation in damage, since some of the variation could be attributed to other factors such as ground motion.

- **Earthquake Ground Motion.** Each and every earthquake that occurs will produce a unique ground motion pattern. For example, the Northridge earthquake produced very high ground motions, while the 1989 Loma Prieta earthquake in California produced much stronger motion at large distances due to the reflection of seismic waves of a subsurface geologic layer. Future earthquakes will likely produce further variations.

- **Earthquake Recurrence Rates.** Substantial uncertainty still exists in predicting the recurrence rate of earthquakes on major fault systems, as well as the likelihood of when the next damaging event will take place. Recent research using subsurface trenching analysis and satellite based geodetic measuring has given greater insight into the movement of the earth's crust, but we still do not know enough to predict with certainty when each fault is likely to rupture.

- **Unknown Faults.** Although great strides have taken place in identifying potentially hazardous faults, damaging earthquakes have occurred on faults whose potential has been unrecognized or under predicted. Since the scientific community still does not have a full three-dimensional view of earth's subsurface, further surprises cannot be discounted.

1.5 Limitations

Our professional services have been performed using that degree of care and skill ordinarily exercised, under similar circumstances, by reputable engineers practicing in the structural field in this or similar localities at this time. No other warranty, expressed or implied, is made as to the professional advice included in this report. This report has been prepared for our client to be used solely in their evaluation of business issues related to use of the subject facility. The report has not been prepared for use by other parties and may not contain sufficient information for the purposes of other parties or other uses.

1.6 Qualifications

Cielo Structures, Inc. is a structural engineering and construction management firm providing seismic engineering, peer review, pre-construction, design, and construction management services to public-sector clients throughout Southern California. With nearly 30 years of experience in the seismic, structural engineering, and construction arena, Cielo Structures, Inc. is fully capable of handling any complex project from inception to close-out.

Cielo Structures, Inc. is state-certified as an MBE/WBE (Minority Business Enterprise/Women Business Enterprise), SBE (Small Business Enterprise) with the State of California and is recognized as an SBE (Small Business Enterprise), EBE (Emerging Business Enterprise), SBE-Micro, SBE-PW (Small Business - Public Works), and VSBE (Very Small Business Enterprise) with the City of Los Angeles.

Chapter 2 - Building Description

The information provided in this chapter provides a generalized overview of the construction and layout of the two-story, concrete masonry shear wall building located at 303 West 5th Street in San Bernardino, California.

2.1 General

The 303 West 5th Street County Building is a two-story reinforced Concrete Masonry Unit (CMU) shear wall structure (Figure 2-1 to Figure 2-6) with parking located at ground level footprint (Figure 2-7 to Figure 2-10) and interior offices and lobby space located at first and second floors (Figure 2-11 to Figure 2-12). Based upon real estate records found via public websites and San Bernardino County portal, the original building appears to have been constructed circa late 1970s or early 1980s and is currently unoccupied. Previously, the building was utilized as an office building for the State of California Department of Corrections and contained private offices along with meeting rooms and ground-level parking.

The planned occupancy and overall use of the building will remain as office space for the San Bernardino County administrative staff. There are currently no available as-built records drawings for the building; however, remodel (tenant improvement) drawings from 2001 are available (Reference 13). Comparing the available 2001 remodel drawings with the observed floor layout reflects one major structural change to the building. The installation of two new ground level columns and footings and a second-level floor beam on the west side of the building were completed recently. The new ground-level columns appear to have been added for gravity purposes only as the second-level floor beam appears to extend only between the two columns and does not extend extensively beyond the columns as a seismic drag beam would entail.

The building was likely designed utilizing the 1976 or 1979 edition of the Uniform Building Code (Reference 2), which is assumed based on the vintage of the building. Note that building codes have changed significantly since the original design, reflecting advancements in structural engineering and lessons learned from previous earthquakes. Important gravity and lateral load resisting features have changed for many of the structural and non-structural components in buildings similar to the subject building.

Prior to about 1970, similar CMU shear wall buildings were not adequately designed and detailed to properly dissipate energy during major earthquakes, resulting in significant damage to important load resisting elements, such as the boundary elements of the shear walls. CMU shear walls from the subject building era are termed “ordinary”-type shear walls. Extensive modifications of code requirements for design and detailing of shear wall construction for both in-plane forces and out-of-plane forces were made following the non-ductile behavior and failure of similar shear walls in the 1970s (for both in-plane, ie: diagonal cracking, and out-of-plane wall elements and diaphragm-to-wall connections) due to the 1971 Sylmar Earthquake in California. Subsequent editions of the UBC and the more recent California Building Code (CBC) have also included modifications (improvements) to the design and detailing of masonry shear wall boundary elements for in-plane loading and out-of-plane wall strength and connections for out-of-plane loading.

2.2 Building Description

The 303 West 5th Street County Building consists of a two-story CMU shear wall structure composed of fully grouted reinforced masonry blocks. An interior mechanical mezzanine is located between the second-floor level and roof level at the south side of the building. The building is rectangular in plan with overall dimensions of approximately 132 feet by 176 feet, resulting in an overall floor area of approximately 23,232 square feet at each level (Figure 2-13 and Figure 2-14). The interior mechanical mezzanine at the south side of the building is also rectangular in plan with overall dimensions of approximately 25 feet by 48 feet (Figure 2-15). The overall height of the building to the main roof is approximately 36 feet at the low end of the roof (north and south sides) and approximately 39 feet at the high roof ridgeline at Grid Line C (Figure 2-16 to Figure 2-20). At the south side of the building, the mechanical mezzanine roof extends approximately 5 feet above the main roof (Figure 2-20). Floor finishes vary from carpet in the office spaces to vinyl tile in the public hallways and common areas.

2.2.1 Vertical Load Carrying System

The main building roof is comprised of 2-1/2-inch-thick rigid insulation on 3-inch-thick insulating concrete (with chicken wire mesh) over 18 gage metal deck overlaid with ½ inch roofing membrane. The welded metal deck spans approximately 7 feet to east-west direction W14x steel wide flange purlin members that span 30-feet long and in turn are supported on north-south direction Wx steel wide flange girder members that are approximately 60-feet long. The steel roof girders are supported on 5-inch diameter steel pipe columns between second level and roof level. The parapet walls around the perimeter of the building at the roof level vary in height between 12 inches to 24 inches. The high roof at the mechanical mezzanine is composed of similar materials as the main roof, with the exception of the rigid insulation.

The typical second floor level is constructed of 3-1/2-inch thick lightly reinforced concrete (with welded wire mesh) over 16 gage metal deck. At the interior of the building, the welded metal deck spans to north-south direction W18x steel wide flange purlin members that span 30-foot to east-west direction W27x steel wide flange girders. At the perimeter of the building, the welded metal deck spans to reinforced concrete perimeter beams that span 30-feet to concrete supporting concrete piers. The metal deck is also welded to a steel ledger angle which is bolted to the interior masonry walls and exterior walls/concrete perimeter beams. The steel girders and purlins at the second level are supported on 24-inch-wide reinforced rectangular concrete piers at the perimeter of the building, 16-inch diameter reinforced concrete columns at the interior, and reinforced CMU bearing shear walls at both interior and exterior conditions.

The slab on grade was not tested; however, it is likely reinforced concrete between 4 inches and 6 inches thick with a single layer of reinforcing located in the center of the slab.

2.2.2 Vertical Lateral Force-Resisting Systems

The building's vertical lateral force-resisting system is composed of reinforced CMU shear walls supported on reinforced concrete continuous footings at the exterior and interior of the building. By today's standards, the building would best be defined as "ordinary" masonry shear walls as the original walls are not constructed with proper reinforcement detailing and boundary elements at the ends of the walls to resist flexural and shear stresses. Today's building codes do not allow for the use of "ordinary" reinforced masonry shear walls in high seismic regions such as San Bernardino.

The main exterior reinforced CMU shear walls at the perimeter of the building are composed of double wythe 10-inch-wide masonry block (with grout between wythes) located along Grid Lines 1 and 7 and single 10-inch-wide block at Grid Line F (Figure 2-1, Figure 2-2 and Figure 2-4). The 8-inch-wide exterior reinforced CMU shear wall at the north side of the building (Figure 2-3) is offset from the main building line along Grid Line "A" by approximately 4 feet due to the lobby stairwell. This offset creates a building irregularity which will be further discussed in Chapter 4. There are interior reinforced CMU bearing shear walls that vary in width between 8-inches and 12-inches along Grid Lines E, 3, 5, and near Grid Line B at first floor and second floor. All of the masonry shear walls are considered "ordinary" reinforced shear walls by code based on existing reinforcement bar layout and splicing. The interior and exterior CMU shear walls are also supported on reinforced concrete continuous foundations.

2.2.3 Horizontal Lateral Force-Resisting Systems

The horizontal lateral force-resisting systems (ie: diaphragm) at the roof level and second floor level are composed of 18 gage bare metal deck and concrete fill on 16 gage metal deck, respectively. In a seismic event, the roof level metal deck, welded to the supporting wide flange beams at the bottom flutes, spans horizontally to interior and exterior CMU shear walls and subsequent foundation systems. The roof metal deck is considered a flexible diaphragm system in the evaluation process, as compared to more rigid diaphragm systems, such as reinforced concrete slabs.

The second-floor diaphragm is composed of 2,000 pounds per square inch (psi) compressive strength light-weight concrete over metal deck, which in turn the metal deck is welded to the wide flange supporting members at the bottom deck flutes. In a seismic event, the second-floor diaphragm spans horizontally to interior and exterior CMU shear walls and subsequent foundation systems based on the rigidity of supporting CMU shear walls. The second-floor diaphragm is considered rigid in the evaluation process similar to other rigid systems as reinforced concrete slabs without metal deck.

2.2.4 Partition Walls

Interior partition walls appear to be constructed of light gauge metal studs covered by gypsum plaster on both sides of the studs. Partition walls heights vary from just above the ceilings to extending to the bottom of the floor or roof above.

2.2.5 Ceilings

At the ground-level, the ceiling is composed of ¾ to 1-inch-thick plaster on wire mesh hung directly below the wide flange framing. Typical ceilings in office areas at the second-floor consist of suspended T-bar ceiling systems with ceiling tiles and inset lights and air conditioning ducts. In common areas such as corridors, the ceiling consists of suspended T-bar ceiling systems with ceiling tiles directly below a gypsum "hard lid" ceiling. The upper gypsum board ceiling above the T-bar ceiling was likely used for fire rating purposes. Areas such as the mechanical rooms, janitor closets, and restrooms consist of "hard lid" drywall ceiling systems.

2.3 Building Gravity Loading

Based on the floor plans for the proposed renovation, the following additional floor loads (per the 2022 California Building Code [CBC]) (Reference 3) were applied to the building for the analysis:

- Roof Live Load = 20 psf
- Floor Live Load (offices) = 50 psf
- Partition Loads = 15 psf
- Mechanical Room = Actual equip. wt. + 20 psf LL

Figure 2-1: West Building Elevation

Figure 2-2: East Building Elevation

Figure 2-3: North Building Elevation

Figure 2-4: South Building Elevation

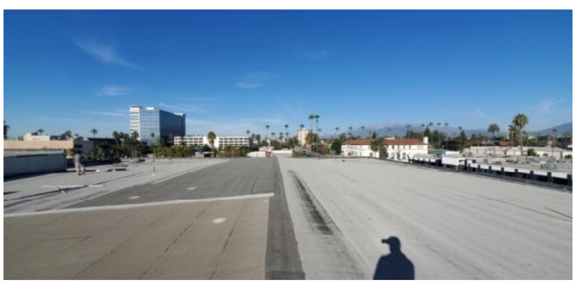


Figure 2-5: Main Roof Level at GL C

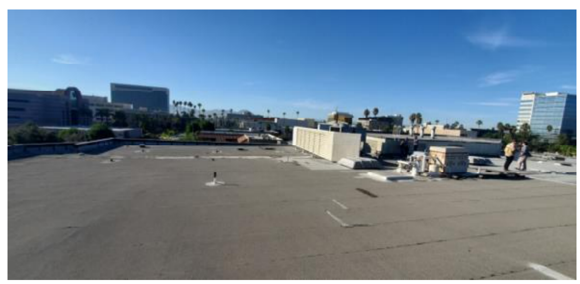
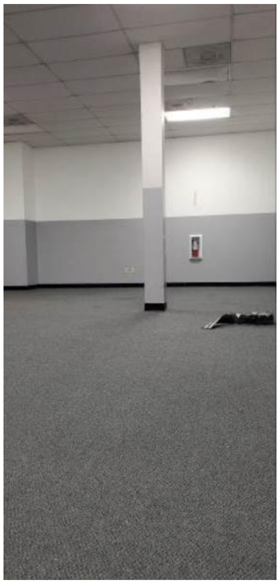
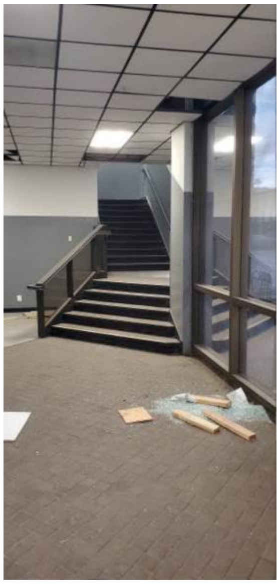


Figure 2-6: Main and High Roof Levels

Figure 2-7: Parking Level (Westward)

Figure 2-8: Parking Level (Eastward)

 Figure 2-9: First Level Concrete Pier	 Figure 2-10: First Level Concrete Col.
 Figure 2-11: Offices & Steel Column	 Figure 2-12: Lobby Space & Stairway

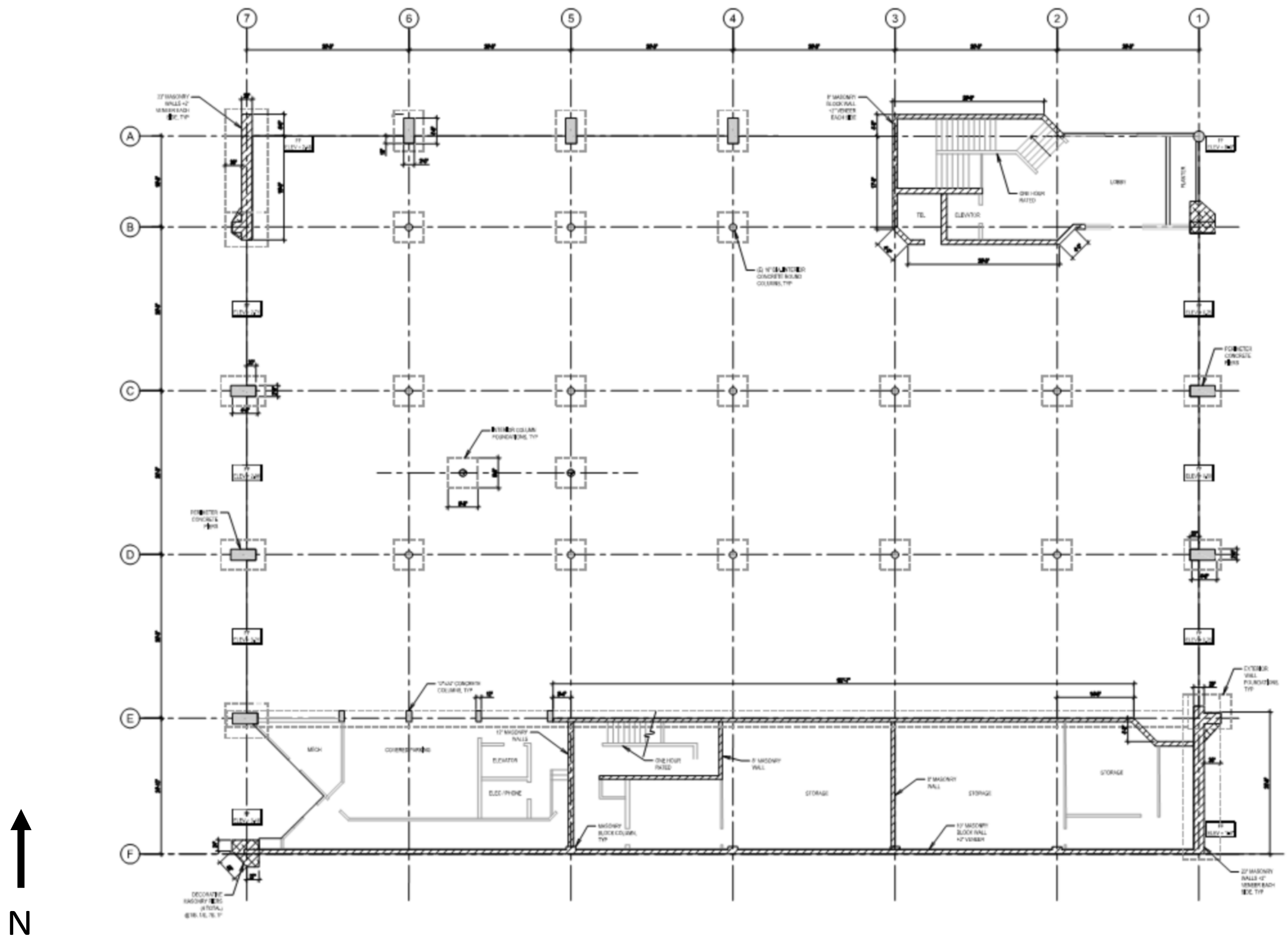


Figure 2-13: Foundation & Ground Level

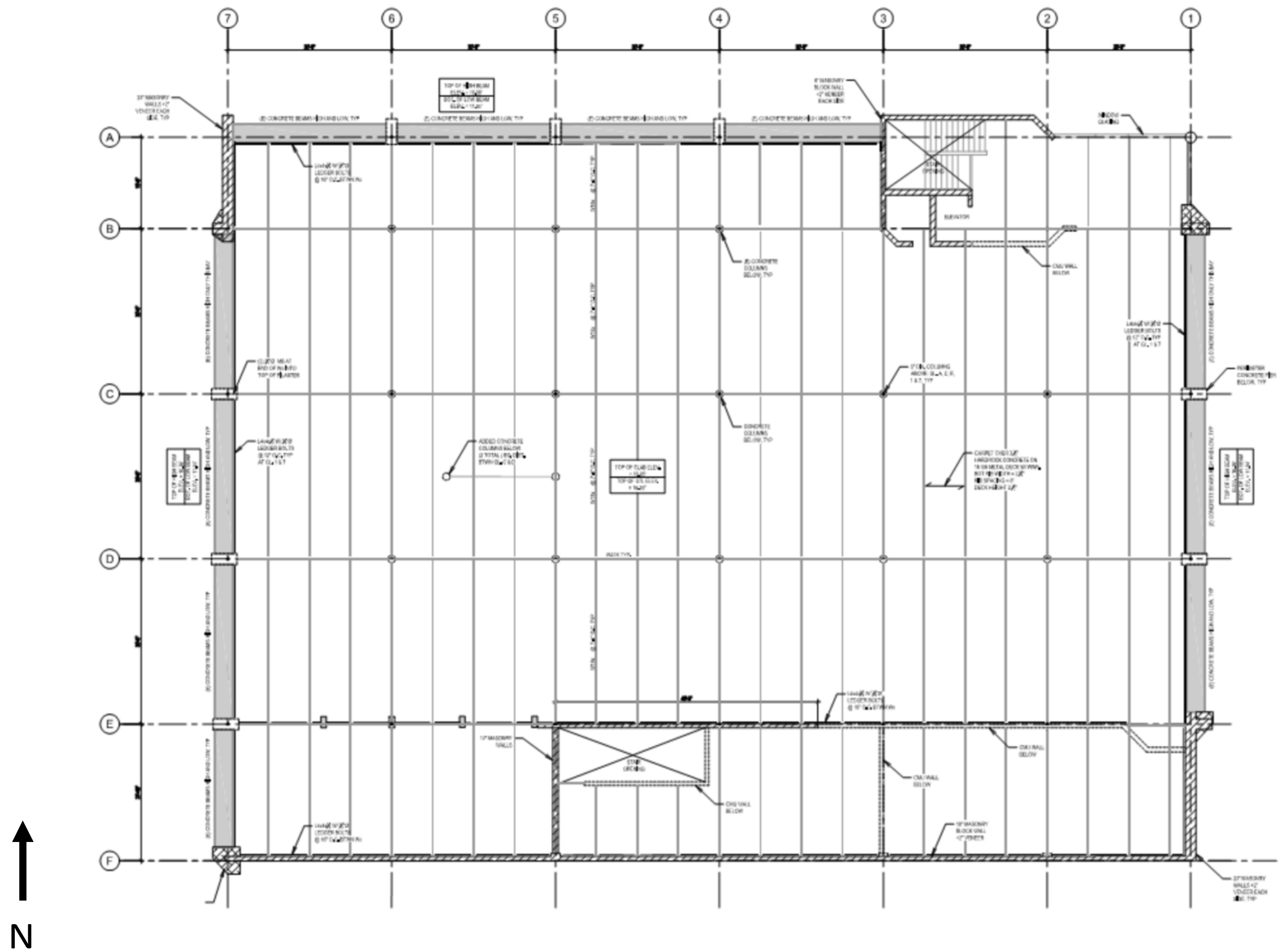


Figure 2-14: Second Floor Framing Plan

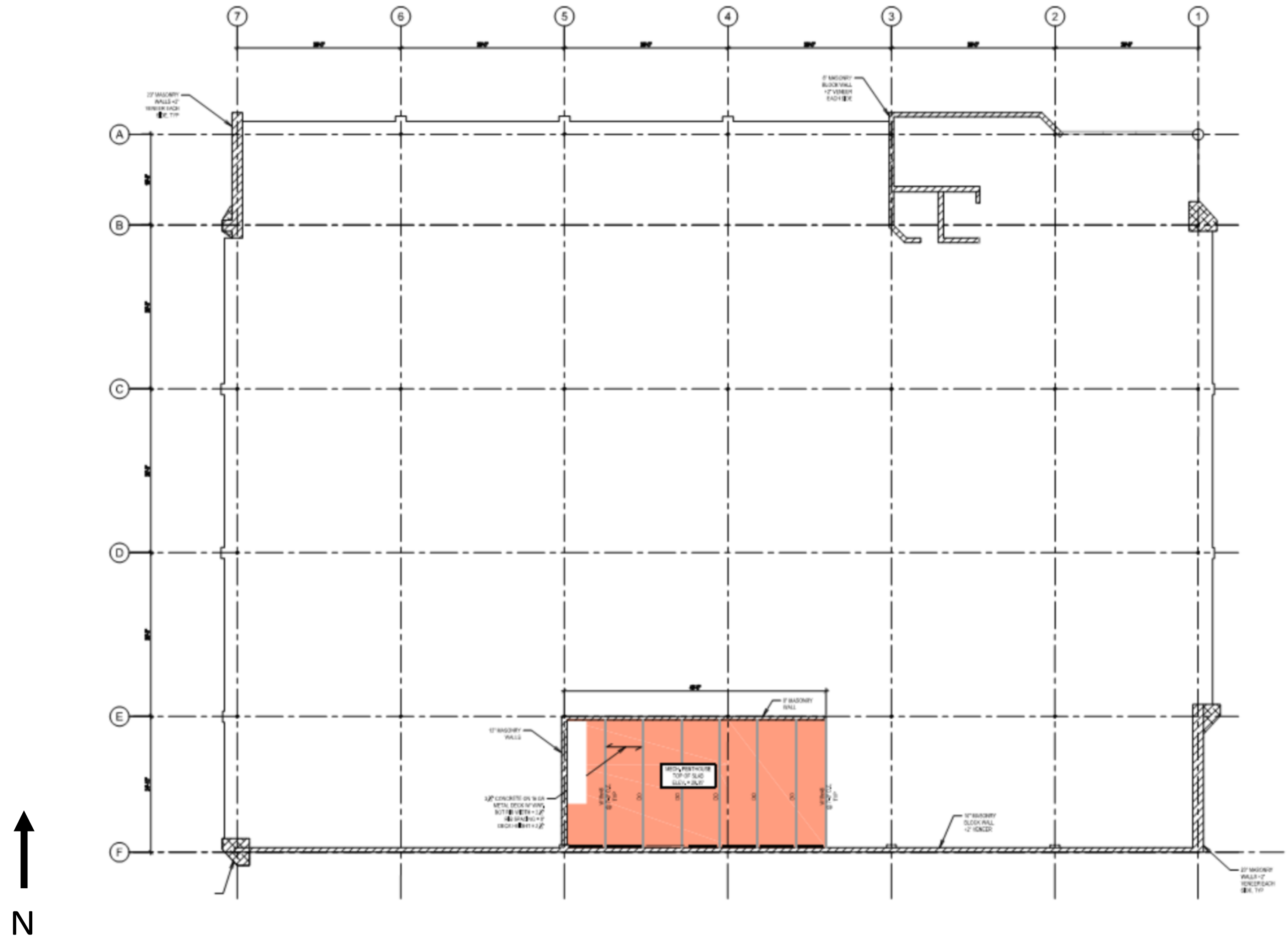


Figure 2-15: Mechanical Mezzanine Level Framing Plan

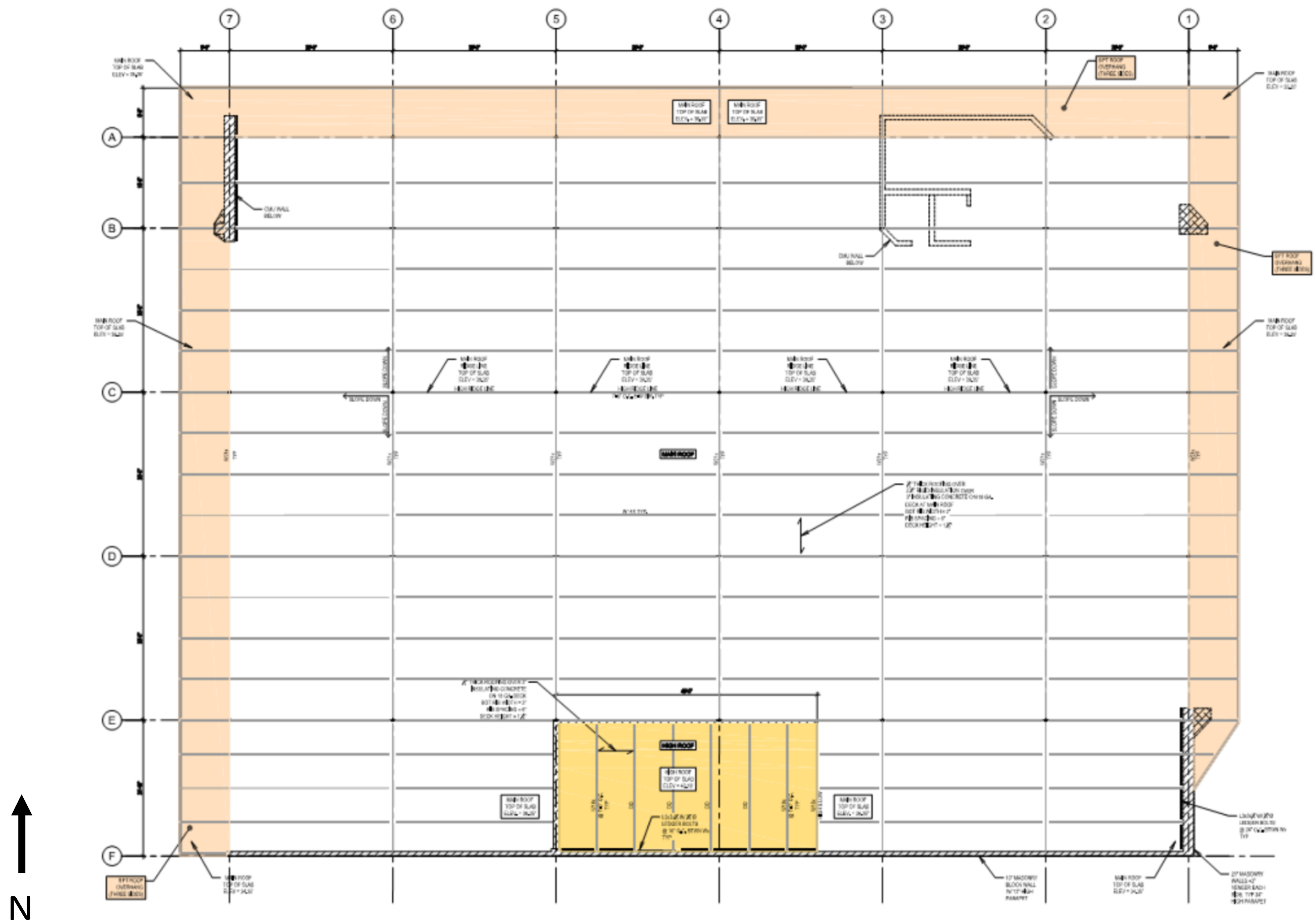


Figure 2-16: Main & High Roof Levels Framing Plan

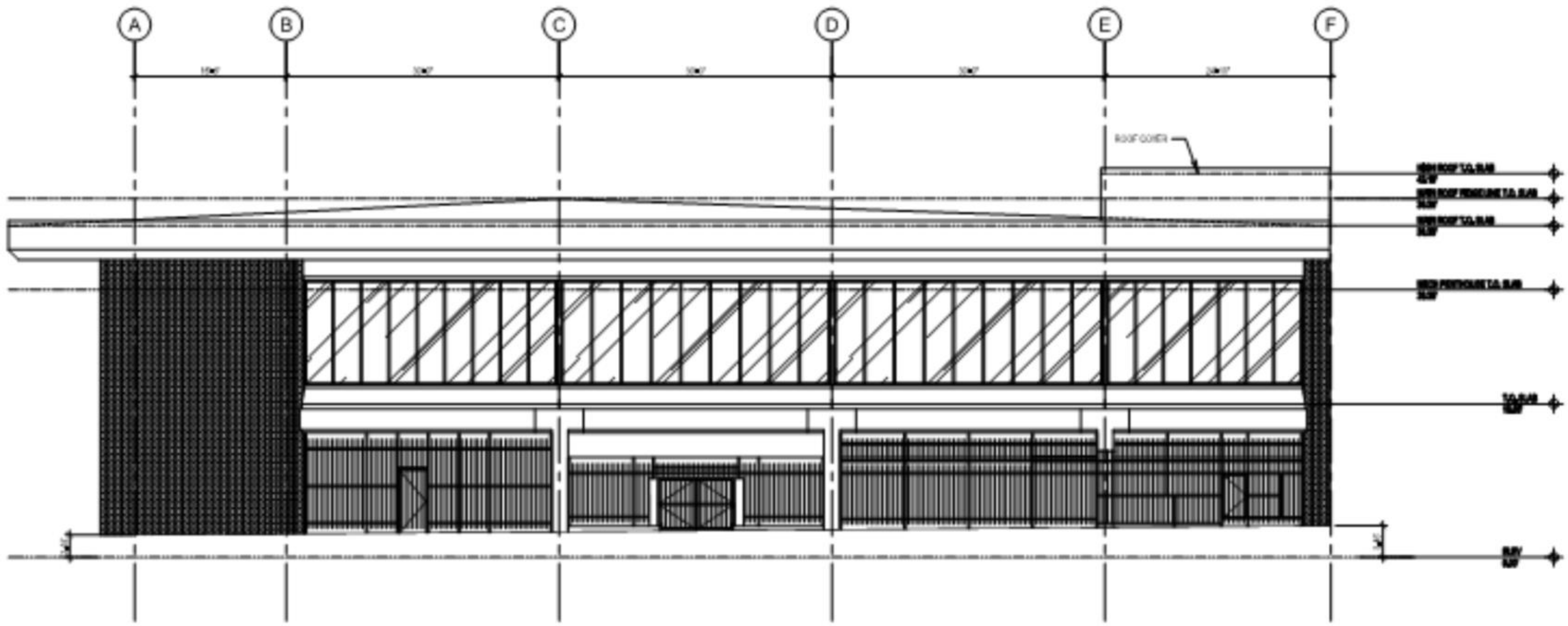


Figure 2-17: West Elevation Rendering

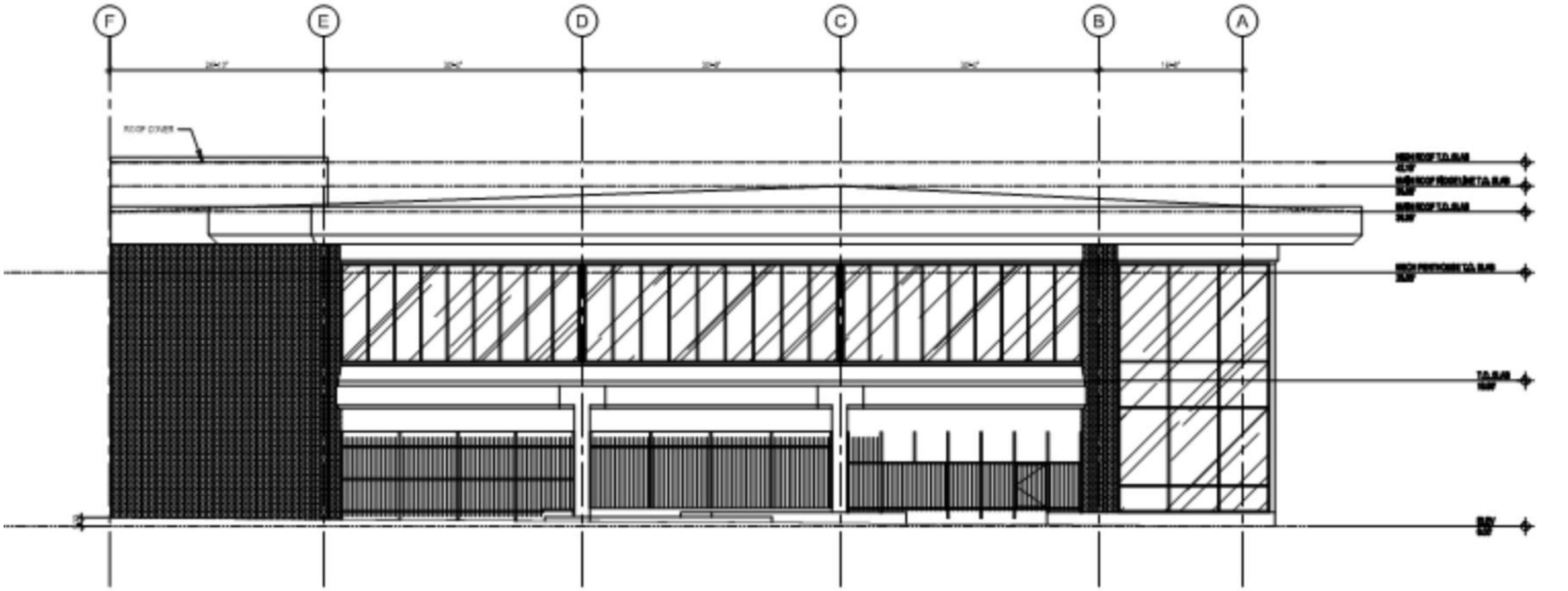


Figure 2-18: East Elevation Rendering

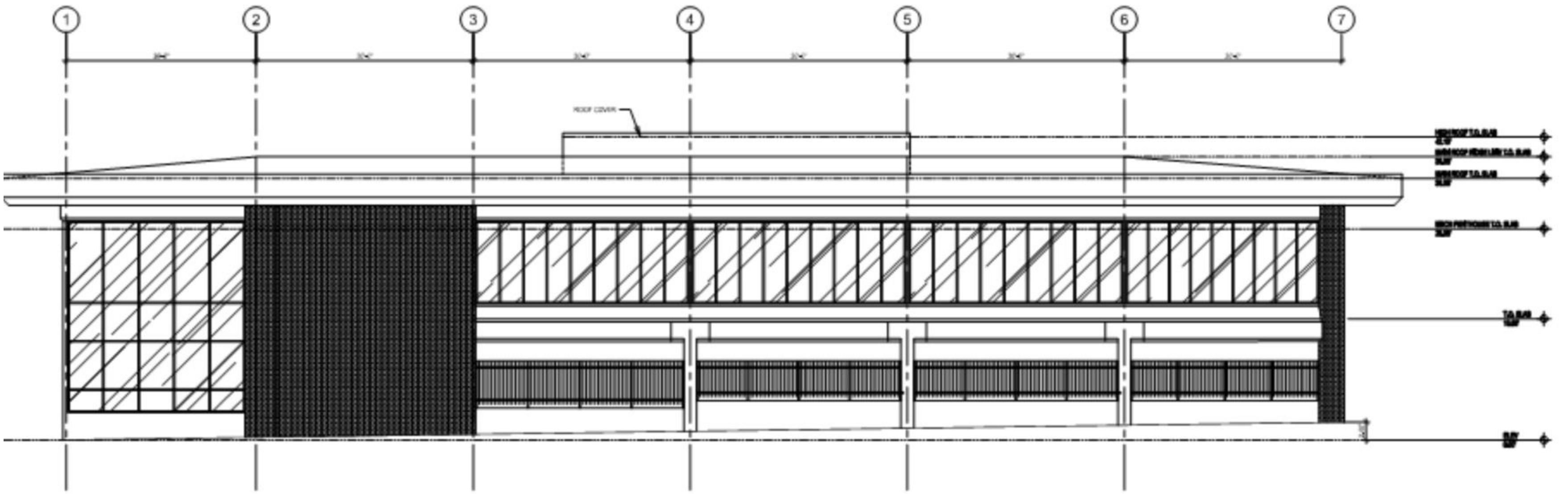


Figure 2-19: North Elevation Rendering

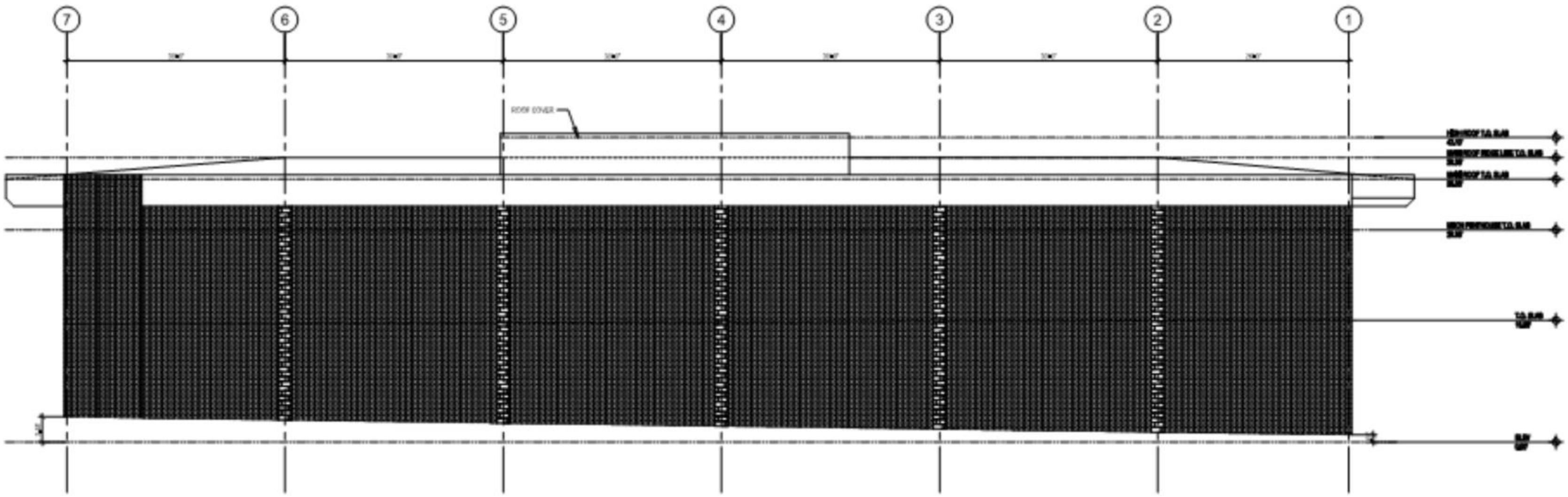


Figure 2-20: South Elevation Rendering

Chapter 3 - Evaluation Criteria and Analysis Procedures

The following sections present the evaluation criteria and analysis procedures used for the seismic analysis of the 303 West 5th Street County Building, in San Bernardino, California. This chapter also briefly documents our assumptions used to determine the capacities of the structural elements.

Please note that Cielo Structures only conducted a detailed review of the earthquake lateral-force-resisting system of the building and that evaluation of the gravity load resisting system (ie: non seismic elements) was not performed.

3.1 Seismic Performance Objective

During a major earthquake, a new building designed to the latest building code is expected to provide some “standard” level of performance in order to protect life and/or minimize business interruption. However, due to changes in the building code based on lessons learned from past earthquakes and information learned from research into building behavior during simulated earthquakes, the requirements to meet a “Life-Safety” level of performance has become stricter over the years.

As an alternative, a building owner can during the design of a building (if they so choose) request that a building be designed using a performance-based design methodology that incorporates design specifications that are stricter than the requirements of the local governing building code and stipulates a desired level of performance (damage control) and business interruption duration.

If desired, existing deficient buildings can also be upgraded to meet desired performance goals. One tool for selecting a desired performance level for a building is to utilize the various performance levels outlined in ASCE/SEI 41-17 *"Seismic Evaluation and Retrofit of Existing Buildings"* (Reference 1). In summary, the performance levels discussed in ASCE/SEI 41-17 are defined as follows:

- **Immediate Occupancy (IO):** The post-earthquake damage state in which only very limited structural damage has occurred. The basic vertical-and lateral-force-resisting systems of the building retain almost all of their pre-earthquake strength and stiffness. The risk of life-threatening injury as a result of structural damage is very low, and although some minor structural repairs might be appropriate, these repairs would generally not be required before re-occupancy. Continued use of the building is not limited by its structural condition but might be limited by damage or disruption to nonstructural elements of the building, furnishing, or equipment and availability of external utility services.
- **Life-Safety (LS):** The post-earthquake damage state in which significant damage to the structure has occurred, but some margin against either partial or total structural collapse remains. Some structural elements and components are severely damaged, but this has not resulted in large falling debris hazards, either within or outside the building. Injuries may occur during the earthquake; however, the overall risk to life-threatening injury as a result of structural damage is expected to be low. It should be possible to repair the structure; however, for economic reasons this may not be practical. While the damaged structure is not an imminent collapse risk, it would be prudent to implement structural repairs or install temporary bracing prior to re-occupancy.
- **Collapse Prevention (CP):** The post-earthquake damage state in which the building is on the verge of partial or total collapse. Substantial damage to the structure has occurred, potentially including significant degradation in the stiffness and strength of the lateral-force-resisting

system, large permanent lateral deformation of the structure, and – to a more limited extent – degradation in vertical-load-carrying capacity. However, all significant components of the gravity-load-resisting system must continue to carry their gravity loads. A significant risk of injury caused by falling hazards from structural debris might exist. The structure might not be technically practical to repair and is not safe for re-occupancy because aftershock activity could induce collapse.

- **Intermediate Categories:** Performance categories between the three performance levels described above can also be chosen for rehabilitation. For instance, one such intermediate performance level is the Reduced Damage Performance Level, which falls in between the Immediate Occupancy and the Life-Safety Performance Levels. Another performance level is Continued Operations, which is stricter than the Immediate Occupancy Performance Level.

In regards to the subject building, leadership for the San Bernardino County is interested in evaluating the 303 West 5th Street County Building to meet ASCE 41-17 Basic Performance Objective for Existing Buildings (BPOE) which includes evaluation and analysis for both the “Life-Safety” and “Collapse Prevention” performance levels.

3.2 Seismic Force Level for Analysis and Retrofit

The 303 West 5th Street County Building was evaluated for both “Life-Safety” and “Collapse Prevention” performance criteria utilizing the current building code for new buildings and nationally recognized criteria specific for existing buildings. These criteria are as follows:

- “Seismic Rehabilitation of Existing Buildings” – ASCE/SEI 41-17 (Reference 1).
- 2022 California Building Code (CBC) (Reference 3).

Both the 1979 Uniform Building Code (UBC) (Reference 2), to which the building was likely originally constructed, and the current 2022 CBC code include design requirements that are intended to represent a “Life-Safety” seismic performance level, meaning the level of risk for life-threatening injury and entrapment directly following a major earthquake should be low. However, the design intent of the building code allows for significant damage to the building in a major earthquake, though some margin against either partial or total structural collapse should remain.

It is important to remember that the basic building code equations and regional seismic shaking intensity maps (seismic demand) have changed significantly since the building was originally constructed based on knowledge gained from research and/or past performance of similar buildings during major earthquakes. In particular, reinforcing requirements and detailing of masonry shear walls has become more stringent in recent years in order to address the inadequacy of shear wall systems. Proper detailed shear walls include adequate lap splicing, reinforcement development, and boundary elements, in order to dissipate energy from major earthquakes without failure (non-ductile framing).

In conducting this study, conceptual strengthening measures were developed where existing lateral force-resisting components were found to be critical and/or significantly deficient. As discussed with and requested by the San Bernardino County, the proposed conceptual strengthening measures are limited to mitigations of major seismic deficiencies and do not include all the building deficiencies noted in Chapter 4. In evaluating possible strengthening measures, several factors were considered, including cost-effectiveness, maintaining building appearance/architectural features and impact of the strengthening work on floor plans and long-term operations.

The criteria in our evaluation of the building are further discussed in the following sections:

3.2.1 Original Design - 1979 Uniform Building Code

No specific seismic design information is available since original drawings are not available. However, the original structural design of the building was likely based on the 1979 Uniform Building Code (UBC). The 1979 UBC utilizes a force-based design methodology and defines the horizontal earthquake force for building design (working stress level, ie: Allowable Stress Design-ASD) using the following equation:

$$V = ZIKCSW$$

where,

Z = Seismic Zone Factor
I = Importance Factor
K = Horizontal Force Factor
C = Factor Based on the Period of the Building
S = Numerical Coefficient for Site-Structure Resonance
W = Building Weight Including Partitions

Since the building utilizes a masonry shear wall system to resist seismic forces and is located in Seismic Zone 4 (based on the 1979 UBC), the design working stress base shear would be approximately V = 0.14W (14% of the building weight). This seismic force level was likely used for the original design of the building's lateral-force-resisting system, including the masonry shear walls, foundations and the floor and roof diaphragms.

Based upon the original 1979 UBC design code, the calculated allowable stress level base shear corresponds to an ultimate strength level base shear coefficient (V/W or Cs) of approximately twenty 20 % (percent), based upon a strength level procedure used by the current code:

C_s = 1.4 V/W = 1.4 x 0.14 = **0.196** (1979 UBC Life-Safety Equivalent Strength Level Design)

3.2.2 NEHRP Mapped Seismic Hazard

The NEHRP mapped response spectrum accelerations, used in the 2022 CBC and ASCE/SEI 41-17, are provided by the United States Geological Survey (USGS) for building locations based on site latitude and longitude. The mapped design values are defined for site class 'D' Default (no soils report). The mapped design response accelerations for short (0.2 second) and long (1.0 second) building periods are summarized below for the ground motion per NEHRP definitions for both the "Life Safety" performance level for BSE-1E (20% occurrence in 50 years – 225 year mean return period) and "Collapse Prevention" performance level BSE-2E (5% occurrence in 50 years - 975 year mean return period):

ASCE 41-17 Life Safety Performance Level (BSE-1E – 225 year):

Mapped Spectral Acceleration Values for rock (Site Class D-Default)

S_S = 1.054; $\mathbf{S_{XS} = 1.265}$

S_1 = 0.353; S_{X1} = 0.688

ASCE 41-17 Collapse Prevention Performance Level (BSE-2E - 975 year):

Mapped Spectral Acceleration Values for rock (Site Class D-Default)

S_S = 2.104; $\mathbf{S_{XS} = 2.525}$

S_1 = 0.824; S_{X1} = 1.4

Current Edition of the California Building Code (BSE-2N - 2,475 year):

MCE Spectral Response Accelerations (Site Class D: F_a = 1.2, F_v = 1.7)

S_S = 2.167; S_{DS} = 2.60

S_1 = 0.863; S_{D1} = 1.467

Current Edition of the California Building Code (BSE-1N – 475 year) :

DBE Spectral Response Accelerations (Site Class D: F_a = 1.2, F_v = 1.7)

$\mathbf{S_{DS} = 1.733}$

S_{D1} = 0.978

3.2.3 2022 California Building Code - Base Shear

The 2022 California Building Code (CBC) utilizes a force-based seismic design methodology and defines the horizontal earthquake force for building design at the ultimate strength level. The 2022 CBC defines the earthquake force for the building using the following equations:

$$V = \frac{S_{DS}}{\left(R/I\right)} W = C_s W$$

but not greater than

$$V_{\max} = \frac{S_{D1}}{T\left(R/I\right)} W = C_s W$$

and not less than

$$V_{\min} = \frac{0.5 S_1}{\left(R/I\right)} . W = C_s W \text{ or } V_{\min} = 0.01W$$

where,

S_{DS} = Design basis spectral acceleration at short period (0.2 second)

S_{D1} = Design basis spectral acceleration at long period (1.0 second)

S_1 = Mapped maximum considered earthquake spectral acceleration on rock at 1.0 second period

I = Importance factor

R = System response modification coefficient

W = Building weight including partitions

T = Fundamental period of the building, not to exceed $1.4T_a$

The fundamental building period in the building as calculated using the current code is 0.31 seconds. The seismic response coefficient (C_s) is multiplied by the effective seismic mass to determine the design lateral force, or horizontal base shear. The seismic response coefficient for an ordinary masonry shear wall system (R=2), calculated at the ultimate strength level (LRFD), is:

C_s = **0.867** (2022 CBC, Life Safety Design, Strength Level)

Note that the seismic response coefficient (C_s) for a life-safety level utilizing the current code (2022 CBC) is nearly **four times larger** than the ultimate strength force level used in the original design of the building (ie: 0.867 / 0.196). Therefore, deficiencies in the building are expected at the life-safety performance level. A comparison of seismic force levels for the codes used in the design and analysis of the building is presented in Table 3-1.

3.2.4 ASCE/SEI 41-17 Building Evaluation

ASCE/SEI 41-17 uses a displacement-based methodology to assess the displacement demands for existing elements in the building's structural system. The methodology is based on research in the 1960s that indicated that actual measured lateral building displacements are larger than the lateral displacements predicted using conventional elastic force-based methodologies. This additional building displacement is accounted for when using the ASCE/SEI 41-17 displacement-based methodology.

Using the actual elastic building displacement with **unreduced** seismic forces, additional displacements of the building are determined and added to the computer model of the building. The capacity of the elements (columns, beams, walls, etc.) to meet the required displacement is assessed based on the nature and quality of the existing elements using force-based techniques. While the displacement-based response spectrums for the 225 year event (BSE-1E) and 975 year event (BSE-2E) can be much larger than the reduced (code) spectra, the magnitudes of the final design force divided by the applicable ductility factors (R factor for current code and m factor for ASCE 41 standard) used in both methodologies tend to be reasonably close in most cases.

3.2.4.1 Linear Dynamic Analysis Procedure

Based on the building type and lack of available material information, the Linear-Dynamic (Response Spectrum) Procedure was selected as the most appropriate method to analyze both the existing and strengthened building configurations. The analysis procedure consisted of the following steps:

- A detailed three-dimensional computer model representing the geometry and cracked stiffness of the building was developed using ETABS (Reference 10). ETABS is a commercially available finite element software package.

- The building mass and the applied gravity loads (self-weight) for explicitly modeled building components (columns, beams, etc.) were calculated by ETABS, while additional mass and gravity loads (plaster, wall cladding, etc.) were applied to the computer model at nodes and structural beam elements.

- The building configurations, element geometries and masses used in the computer models were verified in the field for general appropriateness and accuracy prior to the final analysis.

- NEHRP mapped acceleration values for short and long periods from were used to develop the design spectrum for soil profile “D” Default as summarized above to be used with the ASCE/SEI 41-17 evaluation criteria.

- The pseudo lateral load (V) in a given horizontal direction corresponding to the actual expected building displacements was determined using ASCE/SEI 41-17 for Linear Dynamic Procedure (LDP) as follows with 5% building damping:

$$V=C_1C_2S_aW$$

where,

C_1 = Modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response.

C_2 = Modification factor to represent the effect of hysteretic pinching on the maximum displacement response.

S_a = Response spectrum acceleration at the fundamental building period and damping ratio (based on modal analysis).

W = Building weight including partitions.

As a result, the following parameters apply:

$C_1C_2 = 1.1$

The base shear results from the dynamic modal analysis are multiplied by seismic response coefficients C_1C_2 to determine the design lateral base shear forces and design displacements. For this building, the BSE-1E (ie: 225 year) and BSE-2E (ie: 975 year) seismic base shears from the modal analysis (for life safety and collapse prevention performance levels) are:

ASCE 41-17 Life Safety Performance Level (BSE-1E – 225 year):

V = 1.265 x 1.1 / 1.002 = **1.388** W (ASCE/SEI 41-17 BSE-1E – Life Safety Unreduced Spectrum).

With an average shear-controlled ductility factor of 2 for masonry walls at a Life Safety performance level, and an expected material strength factor of 1.25 and material knowledge factor of 0.75 being considered, the strength level demand divided by appropriate material factors is as follows:

V = 1.388 / (2 x 0.75 x 1.25) = **0.740** W (Strength Level Design)

ASCE 41-17 Collapse Prevention Performance Level (BSE-2E - 975 year):

V = 2.525 x 1.1 / 1.002 = **2.778** W (ASCE/SEI 41-17 BSE-1E – Collapse Prevention Unreduced Spectrum)

With an average shear-controlled ductility factor of 3 for masonry walls at a Collapse Prevention performance level, and expected material strength factors of 1.25 and material knowledge factor of 0.75 being considered, the strength level demand divided by appropriate material factors is as follows:

V = 2.778 / (3 x 0.75 x 1.25) = **0.988** W (Strength Level Design)

3.3 Code Comparison

The overall building seismic forces calculated using ASCE/SEI 41-17 and the 2022 CBC are larger than the seismic forces used in the original design of the building. The variance in code forces can be attributed to knowledge gained from performance of buildings and their components during past major earthquakes, in particular the 1994 Northridge earthquake in California.

Please note that ASCE/SEI 41-17 defines two types of building components, deformation-controlled components and force-controlled components. Deformation controlled components utilize m-factors to represent the expected ductility of the component. Also, a k-factor is utilized to represent the current knowledge of the building component material properties. Individual building component capacities are multiplied by and thereby modified by their corresponding m- and k-factors. Force controlled components utilize a J-factor to reduce earthquake forces to a component based on the ability of the structural system to transfer the full seismic force to the component.

Therefore as indicated in Section 3.2.4.1 of this report, the ASCE/SEI 41-17 seismic base shear can be divided by the m-, k-, and J-factors for the individual components to arrive at a base shear for comparison purposes with the original design code and the 2022 CBC. In addition, ASCE/SEI 41-17 uses expected material strengths, while the original and current codes use specified material strengths. Therefore, the base shear must also be multiplied by the ratio of specified to expected material strengths for a proper comparison of the capacity of the building to resist major earthquakes. Section 3.2.4.1 indicates the equivalent ASCE 41-17 Strength Level Design modified by these factors for comparison purposes.

According to ASCE/SEI 41-17, the knowledge factor should be 0.75 if no testing of the materials has been completed, while a knowledge factor of 1.00 is warranted if material testing is completed. Therefore, a k-factor of 1.0 is taken for the concrete masonry units while a k-factor of 0.75 is utilized for reinforcement steel and wide flanges. Further testing in the implementation (ie: retrofit) phase will benefit the retrofit design for both the reinforcement steel and wide flange steel beams.

Based on the comparison of seismic force levels for the individual building components, the analysis of the building and design of any required strengthening measures is based on ASCE/SEI 41-17 (the BSE-1E and BSE-2E levels). While the overall forces and general design of any strengthening measures can be accomplished with the ASCE/SEI 41-17 Standard, the 2022 CBC (and supporting material standards) must be used for any detailing or layout requirements if the building is retrofitted during the implementation phase. These additional detailing requirements may modify, increase and/or decrease our proposed strengthening concepts.

3.4 Existing Material Strengths

Since original building drawings were **not** available, a material testing program was developed based on the County's material testing budget in order to gather information required to conduct the structural evaluation.

Material test results are reflected in Appendix A; however, a brief summary is provided below for the tested compressive strengths for the structural concrete (f'c) and masonry units (f'm) as noted below for the evaluation:

Ground-level concrete foundations: : f'_c = 3,000 psi.

First-floor concrete columns and piers: f'_c = 4,000 psi.

Second-floor concrete slab over metal deck: f'_c = 1,950 psi

Second-floor concrete perimeter beams: f'_c = 6,500 psi

First and Second floor 8" Concrete Masonry Unit (CMU) walls: f'_m = 1,500 psi

First and Second floor 10" Concrete Masonry Unit (CMU) walls: f'_m = 3,250 psi

Due to the excessively destructive nature of removing reinforcement bar samples and wide flange coupons (and with consideration of the County's budget), no reinforcement bar samples, and steel coupons were tested. In lieu of testing, Grade 60 reinforcement bars were used in the evaluation based on the historical default material properties shown in Table 10-3 in ASCE/SEI 41-17 for a building constructed in the late 1970 or early 1980s. A knowledge factor of k=0.75 was used for all reinforcement bars and wide flange steel beams.

A more detailed explanation of the materials testing and condition assessment program is presented in Appendix "A". The unprocessed raw test data for each concrete core sample location and MTGL's material report are included Appendix A.

3.5 Computer Analysis Model

A three-dimensional finite element computer analysis model was developed for the building using ETABS (Reference 10), a commercially available finite element program. The ETABS model is used to evaluate the adequacy of the existing design based on ASCE/SEI 41-17 and perform stress checks of selected structural members.

The model contains the gravity and lateral force resisting systems, including floor and roof slabs, concrete columns, concrete piers, and interior/exterior masonry walls. The model was developed to satisfy the requirements for building simulation as defined by ASCE/SEI 41-17. Major features of the ETABS computer model shown in Figure 3-2 and Figure 3-3 and used in this study are summarized as:

- Building roof overhang weight and mass is included in the model.
- Building slab, column, beam and wall member sections matching the as-built condition are included.
- Concrete properties are set at specified strengths based on the findings of the material testing program (see Appendix " A").
- Uniformly distributed dead and live gravity loads were applied to the floor and roof slabs.
- Weights of the perimeter masonry walls are included in the model.
- Floor and roof diaphragms are modeled as rigid and flexible diaphragms, respectively.
- P-delta effects are considered for the sustained dead load.
- The spread footing foundations are modeled as rigid pinned nodes.
- Seismic load combinations for gravity plus BSE-1E and BSE-2E response spectra plus accidental torsion.

3.6 Seismic Behavior of Similar Buildings

The subject building primarily utilizes reinforced masonry shear walls to resist lateral seismic forces. The following sections describe the performance of similar buildings in past earthquakes.

3.6.1 Reinforced Concrete Masonry Shear Wall Buildings

Many modern buildings are constructed with reinforced masonry bearing walls. The walls are most typically constructed of hollow concrete blocks, with reinforcing steel inserted within the cavities in the block, which are then grouted solidly. The walls serve as architectural elements, vertical-load-bearing structural elements, and lateral-load-resisting shear walls.

The floors and roofs of masonry buildings can comprise a number of different systems. Many one- and two-story structures are provided with floors and roofs of timber construction; however, metal deck and concrete-filled metal deck supported by steel framing are also common in such structures such as the subject building.

The performance of reinforced concrete masonry buildings with steel floors and roofs is strongly related to the capacity of the wall anchorage to the roof and floor diaphragms. When provided with good wall anchorage details, these buildings often perform well. However, proper wall anchorage was not required by codes until after the 1971 San Fernando earthquake, when many masonry walls separated from their roof diaphragms and failed due to out-of-plane forces on the walls. A series of code changes requiring improved anchorage of masonry walls to diaphragms were enacted following that earthquake to avoid these failures. The subject building was constructed after the 1971 San Fernando earthquake and implemented tightly spaced ledger bolts for in-plane and out-of-plane forces. Over the years, these provisions have been modified several times, as additional earthquakes and research demonstrated that previous requirements were inadequate.

The configuration and detailing of the walls themselves is extremely important to the building's earthquake performance. Walls with extensive openings are often subject to large cracking and spalling of the masonry units around the openings. The 1994 UBC adopted special detailing requirements for such masonry wall frames, intended to provide better performance. However, the use of these improved requirements is optional. The pattern in which the masonry is laid up is also important. Most concrete masonry is laid up in a running bond pattern, in which the joints of the masonry units in each layer are staggered relative to the layers above and below. This is a preferred form of construction. Some buildings incorporate a masonry pattern known as stack bond, in which the joints between units align vertically from the top of the wall to the bottom.

Another important factor in the performance of masonry buildings is the degree of quality control exercised during construction. Some collapses of masonry buildings in past earthquakes have been attributed to failure of ungrouted reinforced cells of masonry, poor quality mortar, and similar construction deficiencies. Continuous inspection during the construction process is an effective method of avoiding such problems; however, such inspection is not required in all cases. Instead, the building codes require greatly reduced design stresses in masonry constructed without special inspection. Many believe that continuous special inspection should be provided, regardless of the design stresses.

As with other types of construction, masonry buildings with substantial plan irregularities have experienced damage in past earthquakes. Storefront or corner buildings with walls on only two or three sides have proven very susceptible to damage.

Table 3-1

Overall Seismic Force Level Comparison [1]

Performance Level	Seismic Base Shear (Ordinary Masonry Shear Walls)			
	1979 UBC (Existing) [2]	2022 CBC (BSE-1N)	ASCE 41-17 (BSE-1E)[3]	ASCE 41-17 (BSE-2E) [3]
Design Seismic Base Shear (V), Strength Level Life Safety	0.196 W	0.867 W	0.740 W	---
Design Seismic Base Shear (V), Strength Level Collapse Prevention	---	---	---	0.988W

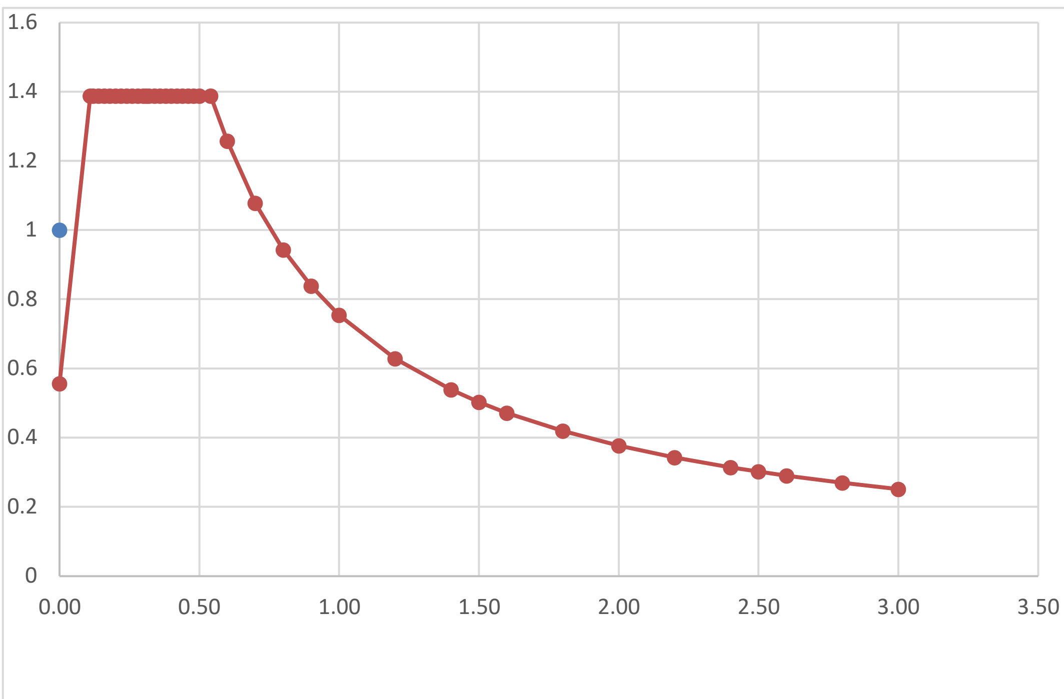


Figure 3-1: Raw Response Spectrum (BSE-1E)

X-axis (T Period) vs Y-axis (Sa acceleration)

[1] Original and current code base shear comparisons.
[2] Strength level base shear calculated as 1.4 times the allowable stress level base shear.
[3] Raw ASCE 41-17 spectrum divided by m factor, k factor, and expected strength factor

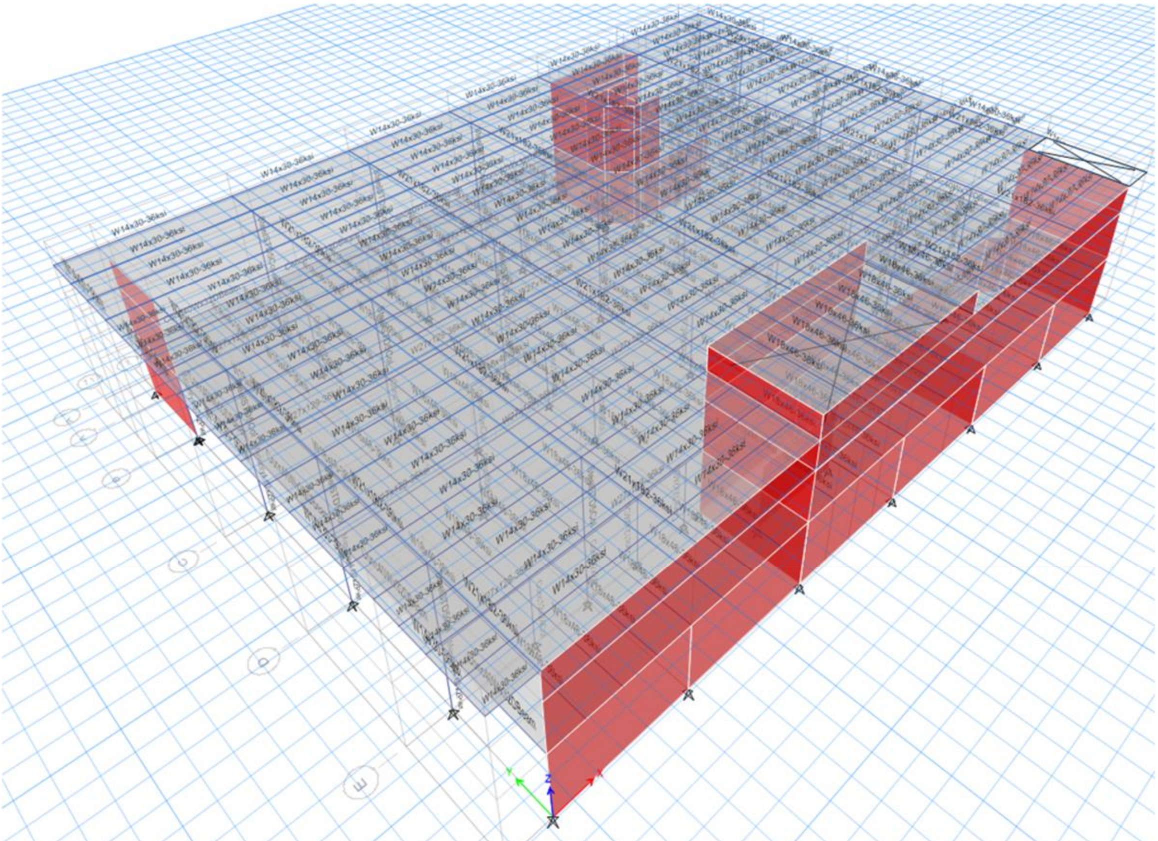


Figure 3-2: ETABS 3-D Finite Element Model

(With Framing Beams Shown)

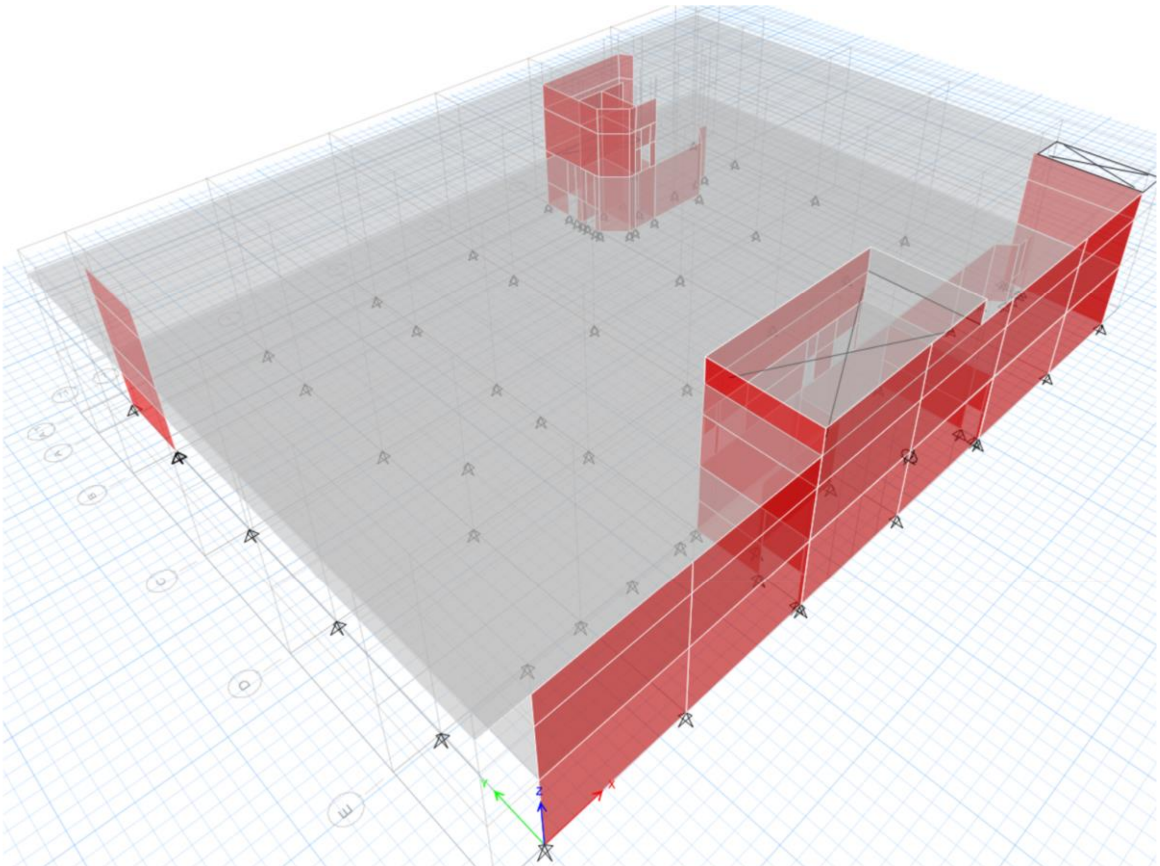


Figure 3-3: ETABS 3-D Finite Element Model

(Without Framing Beams Shown)

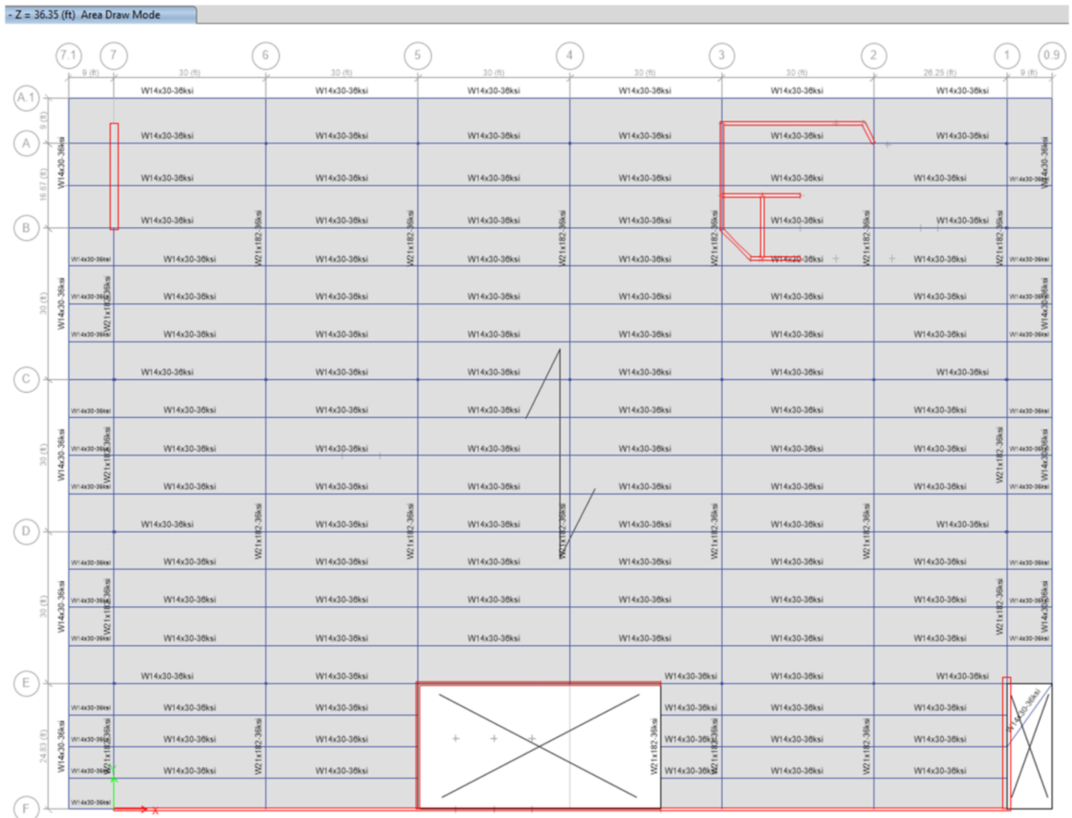


Figure 3-4: ETABS Finite Element Model, Roof Level

Chapter 4 - Structural Evaluation

This chapter presents the findings of Cielo Structure's seismic evaluation of the 303 West 5th Street Building in San Bernardino, California. Potential conceptual strengthening options to correct the major seismic deficiencies are presented in Chapter 5.

The structural analysis is based on the "Life-Safety" (LS) performance level and "Collapse Prevention" (CP) performance level per ASCE/SEI 41-17, *"Seismic Evaluation and Retrofit of Existing Buildings"* (ASCE/SEI 41-17) (Reference 1). A detailed explanation of the evaluation criteria and analysis procedures is presented in Chapter 3.

The findings and seismic strengthening concepts, discussed in the following sections and Chapter 5, are based on an evaluation and assessment of the structural calculations, three-dimensional computer modeling, and field review. A site walkdown was performed by Cielo Structures personnel in October and November of 2024 in order to provide a general survey of the building (since original drawings were not available), provide material testing guidance, and to assess potential constructability issues for proposed strengthening concepts. The site visits generally verified the building configuration, structural elements, general connections, and materials, but it is important to note that hidden conditions exist (especially at the west/east side roof level framing due to current fireproofing conditions) and that some differences are likely to occur during the "Implementation Phase" of the proposed strengthening concepts.

4.1 Structural Analysis Findings

The result of the structural analysis for the building shows that the existing vertical (ie: shear walls) and horizontal (ie: diaphragms) lateral force-resisting systems are severely overstressed when compared to the requirements of the performance objectives (discussed in Chapter 3). Most of the discovered deficiencies can be highly attributed to the following:

- Higher seismic forces between the older and newer design codes due to better understanding of building performance and proximity of nearby earthquake faults

- Use of ASCE/SEI 41 Basic Performance Objective of Existing Buildings (BPOE) for both the "Life-Safety" and "Collapse Prevention" performance levels where "Collapse Prevention" performance level is twice (2x) the seismic demand as compared to the "Life-Safety" performance level, with nominal increase in material lower-bound and expected strength capacities (30% to 50%) at the "Collapse Prevention" level

- Design practices, which were based upon less stringent detailing requirements for reinforced masonry shear walls and drag connection elements

- Marginal original design of roof level diaphragm.

4.1.1 Demand/Capacity Ratio

To give an idea of the extent of the deficiencies, the use of the "Demand/Capacity (D/C) ratio" is used in the following discussions. The D/C Ratio is a comparison of design force to design capacity based on our preliminary calculations and computer modeling. Ratios of 1.0 and less meet the lateral loading criteria, while D/C ratios greater than 1.0 indicate that the seismic forces exceed the allowed capacity of an element. Some components with D/C ratios greater than 1.0 are deemed acceptable due to inherent overstrength and/or ductility of their materials and their relative risk to life-safety and business interruption. A summary of maximum D/C ratios for the masonry shear walls, drag elements, diaphragms, beams, connections, and footings is presented in Table 4-1. Refer to Figure 4-1 through Figure 4-3 for floor plans showing the layout and location of overstressed structural elements.

4.2 Comparison of Current Code Criteria to Original Design Criteria

There are currently no available as-built records drawings for the building; however, remodel (tenant improvement) drawings from 2001 are available (Reference 13). Based upon real estate records found via public websites and San Bernardino County portal, the original building appears to have been constructed circa late 1970s or early 1980s.The 1979 Uniform Building Code (Reference 2) (UBC), or a slightly earlier edition, would have been utilized for the original building design.

The primary vertical lateral-force-resisting system of the building is comprised of ordinary reinforced masonry shear walls. The horizontal lateral-force-resisting system for the building is composed of bare metal deck diaphragm at the roof level and concrete fill over metal deck diaphragm at the second floor level. Concrete and steel drag beams are located at the second floor and roof levels. Based on our evaluation, the design-basis horizontal force (base shear) used in the **original** design of the lateral-force-resisting systems is nearly **four (4)** to **five (5) times** lower in the 1979 UBC, when compared to the current 2022 California Building Code (CBC) (Reference 3) and ASCE/SEI 41-17 *"Seismic Evaluation and Retrofit of Existing Buildings"* Standard. Therefore, the building in its current state does not meet the minimum "Life-Safety" nor "Collapse Prevention" performance objectives, as specified in the 2022 CBC and ASCE 41-17 Standard.

Based on our analysis, the following sections present the most significant findings pertaining to critical elements in the building's existing lateral-force-resisting systems.

4.2.1 Concrete Masonry Unit (CMU) Shear Walls

There are three types of existing masonry block widths that were used for the reinforced masonry shear walls (8-inch, 10-inch and 12-inch) as indicated in Chapter 2. The exterior reinforced masonry shear walls at the northwest and southeast corners of the building are composed of double wythe 10-inch block, with grout joint between blocks, and two layers of No. 5 vertical reinforcement bar at each block at 12 inches on center and two layers of No. 5 horizontal reinforcement bar at each block at 16 inches on center.

The south exterior full-length masonry wall is composed of 10-inch-wide blocks with two layers of No. 5 vertical reinforcement bar at 14 inches on center and two layers of No. 5 horizontal reinforcement bar at 24 inches on center. The interior masonry shear walls, and north side exterior masonry shear wall, are composed of 8-inch and 12-inch-wide blocks with No. 5 vertical reinforcement bars that vary in spacing from 12 inches to 16 inches on center and No. 5 horizontal reinforcement bars that vary in spacing from 12 inches to 24 inches on center.

The masonry walls were evaluated for both in-plane and out-of-plane seismic loading at both the first floor and second floor levels. The evaluation indicates that the masonry walls are overstressed for in-plane seismic loads based on current design requirements and it is expected that they will experience significant damage during the repeated cyclic loading of an earthquake. D/C ratios in some locations were over 2.0 for “Life-Safety” and 5.0 for “Collapse Prevention”.

In addition, the north masonry shear wall near the lobby stairway is offset from the collector/drag elements along the north side of the building at Grid Line A, resulting in limited seismic forces being delivered to this shear wall due to the horizontal irregularity. **This is a major structural deficiency** that can lead to localized failure and partial collapse and non-use of the critical stairway near the lobby after a seismic event.

The evaluation also indicates that the masonry walls are overstressed for out-plane seismic loads at the second-floor level near the lobby stairway with D/C ratios of 1.4 and 1.8 for the “Life-Safety” and “Collapse Prevention”, respectively. See Table 4-1 for D/C ratios.

4.2.2 Masonry Wall Ledger Bolt Connections

The existing masonry wall ledger bolt connections consist of 3/4-inch diameter bolts connected directly between the existing steel ledger angle and embedded into the grouted masonry wall. The spacing of the ledger bolts at the second floor vary from 12-inch to 16-inch on center while the spacing of the ledger bolts at the roof level are spaced at approximately 24 inches on center. Embedment length into the masonry wall is unknown (destructive testing was not performed); however, a 4-inch embedment (or longer) is typical.

The D/C ratios for in-plane seismic loading on the ledger bolts at the second floor and roof level are 2.4 and 2.7, respectively, for “Life-Safety” level. The D/C ratios for in-plane seismic loading on the ledger bolts at the second floor and roof level are 3.8 and 4.2, respectively, for “Collapse Prevention” level.

The D/C ratios for out-of-plane seismic loading on the ledger bolts at the second floor and roof level are 1.0 and 0.6, respectively, for “Life-Safety” level. The D/C ratios for out-of-plane seismic loading on the ledger bolts at the second floor and roof level are 1.3 and 0.9, respectively, for “Collapse Prevention” level. See Table 4-1 for D/C ratios.

4.2.3 Roof and Second Floor Diaphragms

The roof level and second floor level diaphragms are composed of bare metal deck and concrete-fill on metal deck, respectively. Similar to the masonry shear walls, the current seismic forces on these diaphragms, based on ASCE 41-17 LS and CP performance levels are four (4) to five (5) times higher than the original 1979 UBC code, hence the overstresses for these diaphragms is expected.

The D/C ratios for second floor and roof diaphragms are 2.1 and 3.5, respectively, for “Life-Safety” level while the D/C ratios for in-plane seismic loading on the ledger bolts at the second floor and roof level are 2.7 and 4.7, respectively, for “Collapse Prevention” level. See Table 4-1 for D/C ratios.

4.2.4 Drag Elements and Connections

The second-floor perimeter concrete beams at the west, north, and east elevations consist of an upper and lower beam system that are composed of lightly reinforced longitudinal bars based upon the recent material testing program conducted by MTGL (Appendix A). The lower concrete perimeter beams, measuring 24 inches wide by 32 inches deep, appear to be architectural beams only as these elements on the on the west and east elevations terminate between Grid Lines B and C. The upper concrete perimeter beams, measuring 40 inches wide by 32 inches deep without the architectural chamfer, are located for the full length and width of the building (less masonry shear wall bay) and appear to have been designed as collector/drag elements for the second floor. The D/C ratio for the second-floor concrete drag element directly adjacent to the masonry shear wall at Grid Lines E/1 and B/7 are 4.0 and 6.0 for “Life-Safety” and “Collapse Prevention” performance levels, respectively.

The second floor and roof level steel wide flange framing members (and associated connections) that align with interior and exterior masonry walls will behave as collector/drag elements and deliver seismic loads into the reinforced masonry walls to the extent of their capacities to act as tension/compression members (or shear elements for connections). Based on the seismic evaluation, second floor and roof level steel members are overstressed with D/C ratios of 1.2 and 1.6, respectively for the “Life-Safety” performance level. D/C ratios of 1.8 and 2.5 for second floor and roof level steel members, respectively, for the “Collapse Prevention” performance level are noted in Table 4-1. The drag connection at each end of the steel wide flange member was not visible but likely consists of at least four (4) or 5 (five) A307 bolts into the adjacent wide flange member. The drag connection at the end of the steel wide flange member at the masonry shear wall, the largest magnitude collector load, was not visible but likely consists of at least two (2) A307 embedded bolts into the masonry wall. These connections would need to be verified in the Implementation Phase as the D/C ratio for the connection near the masonry wall is the largest in magnitude for the entire building at 46.0 for "Life Safety" performance level.

4.2.5 Soil Bearing

Based upon the severity of the reinforced masonry shear walls, a brief evaluation was conducted for the soil bearing at the existing foundation system at Grid Line 1 between Grid Lines E and F. The soil bearing pressure due to seismic and gravity loading exceeds the ultimate bearing capacity of 6,000 pounds per square foot by over 200% (D/C of 3.6) for "Life-Safety" performance level (Table 3-1).

4.2.6 Drift and Adjacent Building Separation

The 1979 UBC does not contain any specific requirements for drift, except to state that limits should be based on accepted engineering practice. In contrast, the 2022 CBC provides a method for calculating the inelastic drift based on the elastic drift, an amplification factor and the importance factor. These drift limits are provided to help limit building damage. The inelastic drift limit specified in ASCE 7-16 (Reference 4) for cantilevered masonry shear wall structures is 1.0 percent for occupancy categories I and II (Life-Safety). However, these inelastic drift limits are applicable to structures designed to the current code (i.e., with ductile detailing).

Based on our ETABS computer modeling analysis (Reference 10), the lateral displacement at the roof level for the building is 3.8 inches (0.87 percent drift) in the east/west direction and 3.5 inches (0.80 percent drift) in the north/south direction.

Overall, displacement in the east/west and north/south direction are less than the permissible 2022 CBC drift limits of 1%.

The existing adjacent building at the south side of the property near Grid Line F (owned by others) is approximately 10 feet tall and is located 3 inches clear from the subject County building. The adjacent building is also a masonry shear wall building. If the adjacent building drift is established as 1% per 2022 CBC limits this equates to an approximate lateral displacement of 1.2 inches. Based on the ETABS computer model analysis, the lateral displacement at the second-floor level for the County building in the north/south direction is approximately 1.3 inches. The required building separation based upon 2022 CBC using Square Root Sum of the Square method (SRSS) is 1.8 inches. Since a 3-inch separation is provided between the two existing buildings, the buildings have adequate separation and should not “pound” against each other.

4.3 Non-Structural Items

Survey and analysis of non-structural components including partition walls, ceilings and MEP equipment is outside the scope of work for this report and was not conducted.

Table 4-1

D/C Ratio Summary[1, 2]

Structural Component	Level	Max D/C Ratio (LS[3])	Max D/C Ratio (CP[4])
CMU Masonry Walls (In-Plane)	First Floor	2.3	4.7
	Second Floor	2.4	5.1
CMU Masonry Walls (Out-of-Plane)	First Floor	0.7	1.0
	Second Floor	1.4	1.8
Masonry Connections (In-Plane)	Second Floor	2.4	3.8
	Roof Level	2.7	4.2
Masonry Connections (Out-of-Plane)	Second Floor	1.0	1.3
	Roof Level	0.6	0.9
Diaphragms	Second Floor	2.1	2.7
	Roof Level	3.5	4.7
Concrete Drag Beams	Second Floor	4.0	6.0
Steel Drag Beams	Second Floor	1.2	1.8
	Roof Level	1.6	2.5
Drag Connections	All Levels	0.6 - 46.0[5]	0.9 - 71.0[5]
Foundation at GL 1	Ground Level	3.6	5.4

1 D/C ratios < 1.0 indicate that the structural components meet the requirements for the indicated performance level. D/C ratios >1.0 indicate non-compliant structural components.

2 The reported D/C ratios represent the maximum ratio for the given structural component. The D/C ratios will vary from member to member for each structural component.

3 LS: Life Safety Performance Level

4 CP: Collapse Prevenon Level

5 Assumes (2)-A307 bolts at end of steel beam to masonry wall

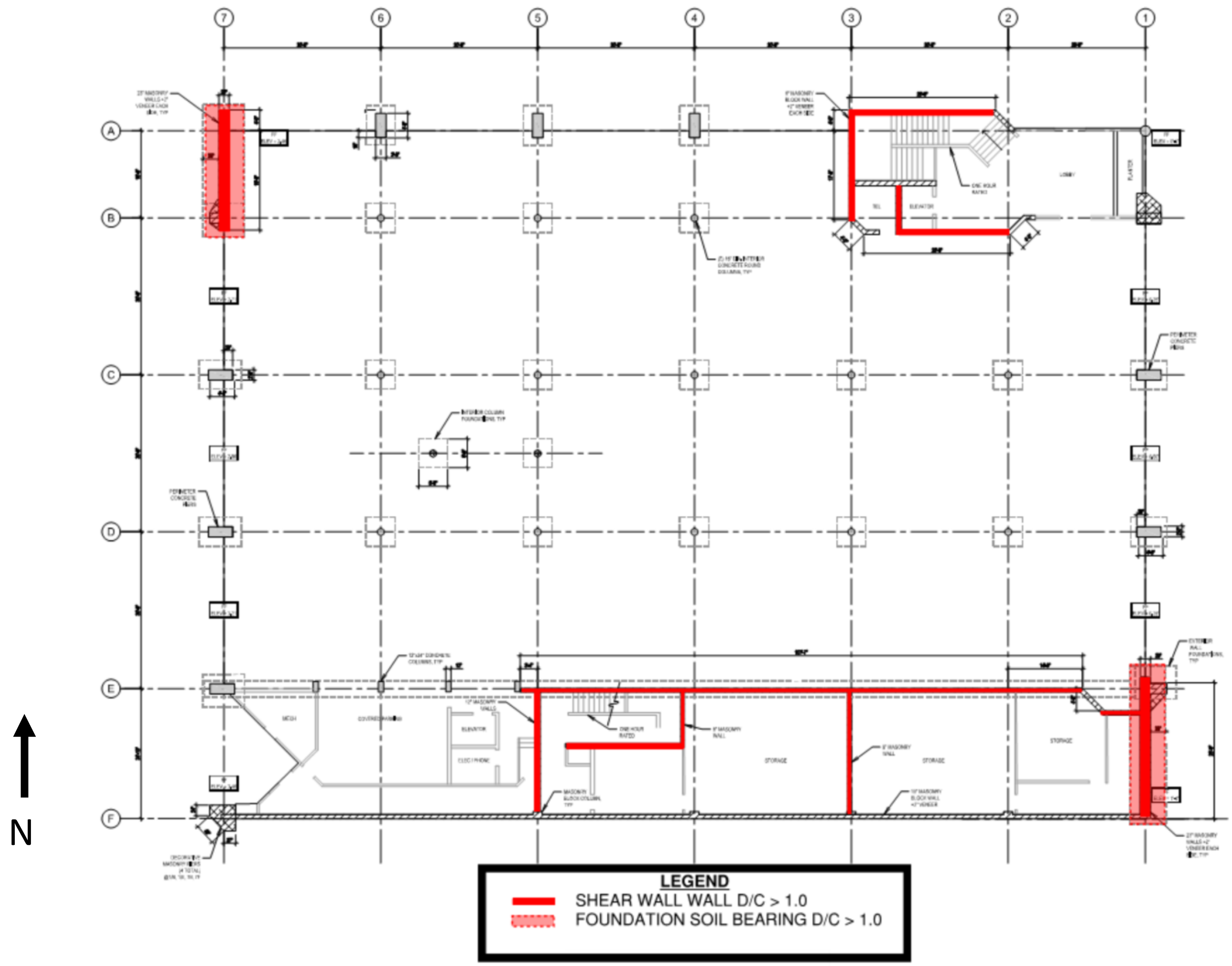


Figure 4-1: Foundation and Ground Level D/C Ratio Map

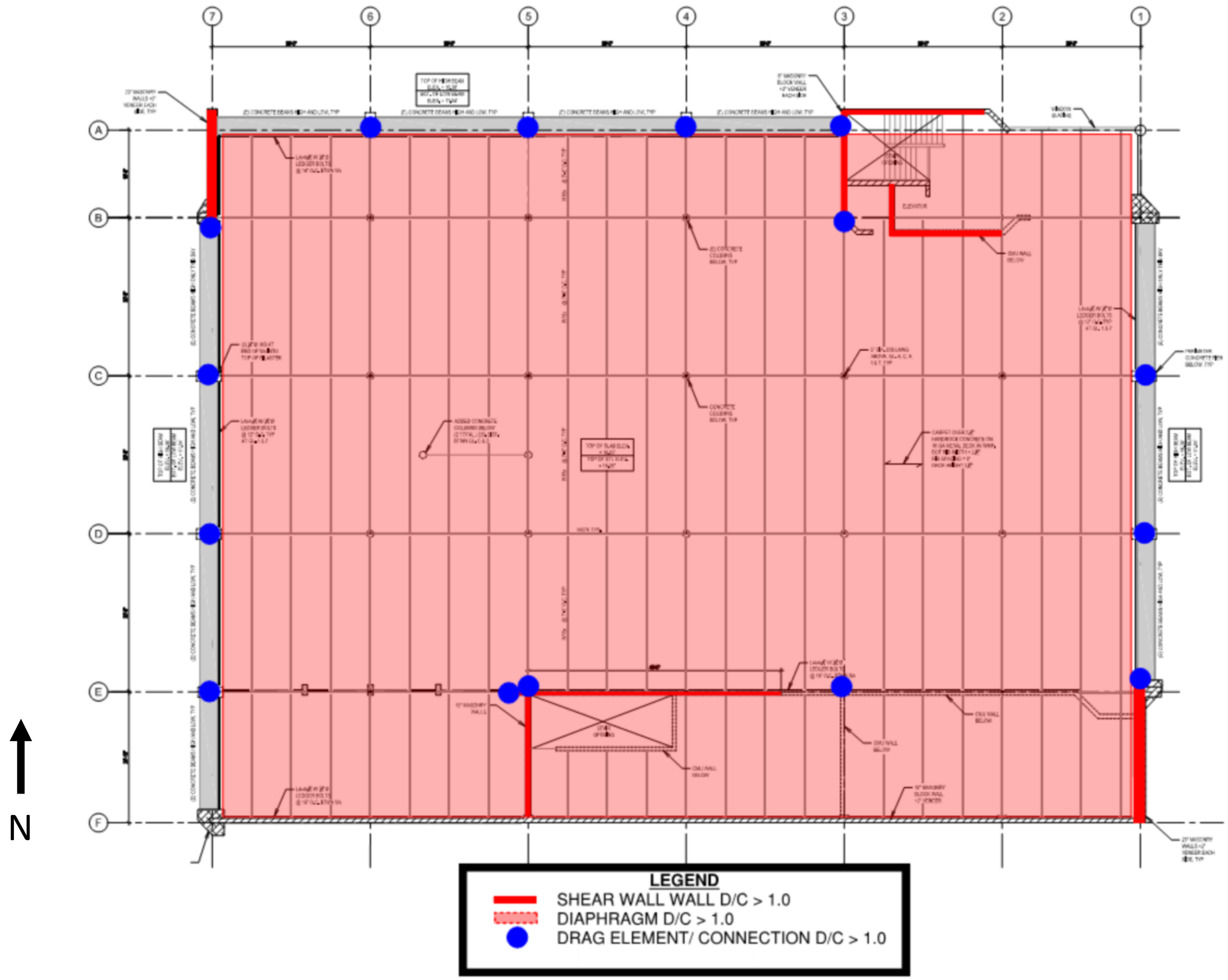


Figure 4-2: Second Floor D/C Ratio Map

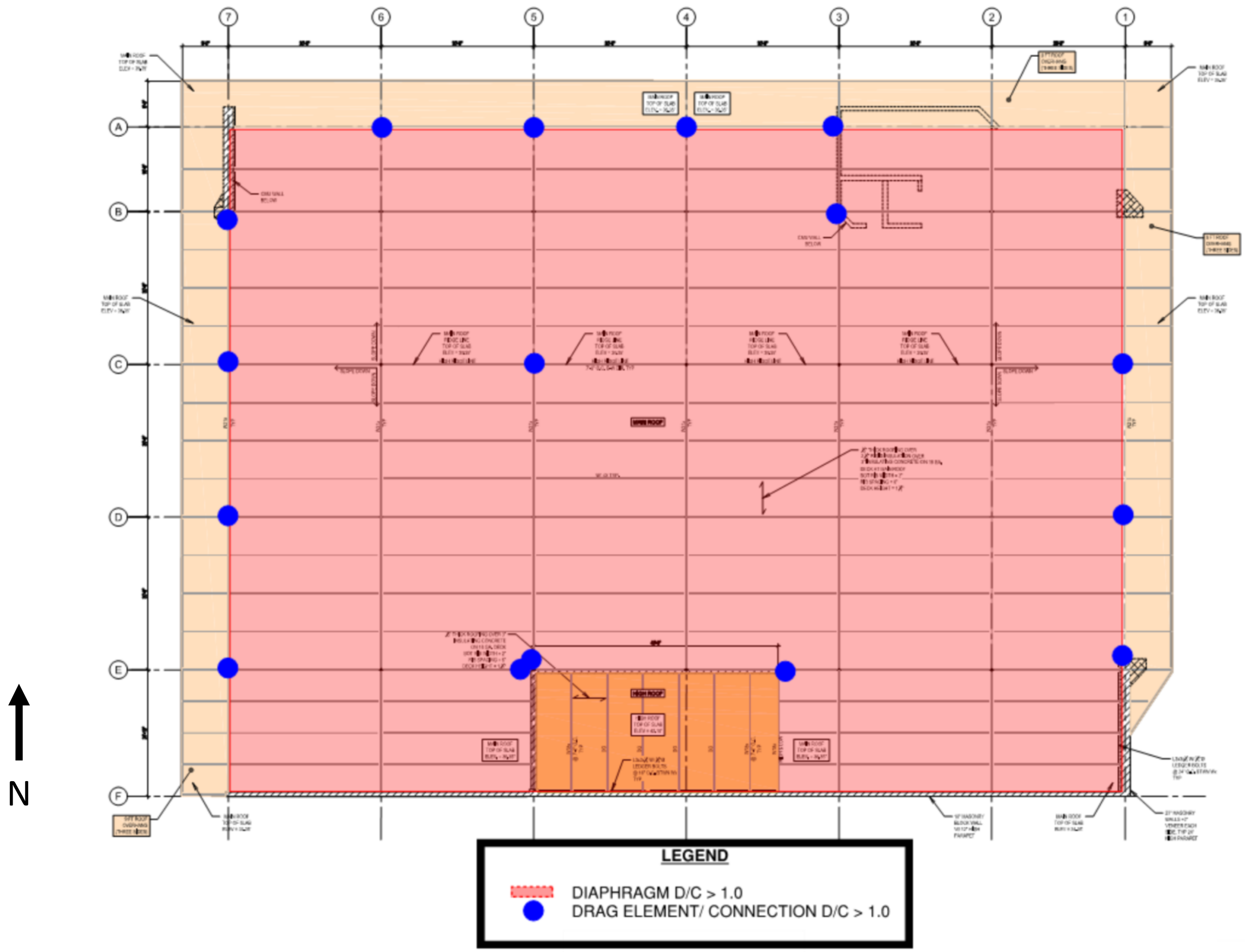


Figure 4-3: Roof Level D/C Ratio Map

Chapter 5 - Seismic Strengthening Concepts

This chapter presents our recommendations, including possible conceptual strengthening schemes developed to mitigate the deficiencies discussed in previous chapters in this report. The retrofit schemes presented in this chapter are only some of the schemes that could be used to improve the performance of the building during a significant seismic event.

5.1 Seismic Performance Design Basis

Cielo Structures performed a structural/seismic analysis to identify problem areas relating to the original design. Conceptual retrofit schemes were developed for upgrading the building for the "Life-Safety" and "Collapse Prevention" performance levels.

The conceptual strengthening schemes were developed in enough detail to demonstrate the general concepts that could be used. The details provided in this chapter as a possible solution can be utilized to finalize and implement the retrofit of the building to a selected level of strengthening based on overall risk reduction objectives.

The proposed strengthening schemes attempt to take into consideration some of the current functions within the existing building and were selected based upon discussions with client's personnel.

5.1.1 Secondary Considerations

Depending on the usage of the building at the time of retrofit implementation, the proposed retrofit concepts may be modified to reflect revised conditions.

No original as-built drawings were available for review during the time of this analysis.

5.2 Strengthening Concepts

Multiple retrofit options were investigated using ETABS computer modeling software (Reference 10) before two options, based on conventional concrete shear wall lateral force resisting systems, were chosen. These two systems were chosen due to the existing masonry and steel construction of the building and the need to significantly reduce masonry shear wall and diaphragm overstresses. The following potential strengthening concepts were evaluated and are being proposed:

- Addition of new reinforced concrete shear walls at both the interior and exterior of the building between existing masonry walls.
- Addition of new foundation system and concrete caissons below the new reinforced concrete shear walls.
- Addition of new reinforced concrete overlays on select interior masonry walls.
- New horizontal steel bracing elements and connections below the second floor and roof levels.
- Additional drag elements and tie connections at the second floor and roof level.
- New out-of-plane connections at select masonry walls at northwest and southeast corners of the building.
- New steel strong-back near the Lobby stairway.

The proposed systems will increase the building stiffness and, while further decreasing the lateral drift of the building, increase the base shear forces. Foundation work and excavation for the new shear walls will be required.

The proposed building strengthening was developed using ASCE/SEI 41-17 (Reference 1) to meet the current 2022 California Building Code (CBC) (Reference 3) strength requirements for new lateral force resisting systems. Should the design proceed to a final construction phase, the 2022 CBC (and supporting material standards) must be used for any detailing requirements and ASCE/SEI 41-17 Standard can be utilized for the actual retrofit design upgrades at the Authority Having Jurisdiction approval. These additional detailed requirements may modify the proposed strengthening concepts.

5.3 Seismic Strengthening Options to Meet Basic Performance Objective for Existing Buildings (BPOE) for Life-Safety and Collapse Prevention

Leadership for the San Bernardino County is interested in strengthening the 303 West 5th Street County Building to meet ASCE/SEI 41-17 Basic Performance Objective for Existing Buildings (BPOE) which includes upgrades to meet both the "Life-Safety" and "Collapse Prevention" performance levels. The proposed strengthening concepts that meet the BPOE are briefly discussed in the following sections.

5.3.1 Building Strengthening Option No. 1

- **Reinforced Concrete Shear Walls and Foundations:** New reinforced 10-inch-thick concrete shear walls are proposed to upgrade the building to meet the ASCE/SEI 41-17 BPOE from ground level up to the roof level. The concrete shear walls are proposed at the ground level (rather than a brace frame system) in order to maintain some form of building deflection-compatibility with the existing masonry shear wall system at the ground level. A steel brace frame system was not proposed at the ground level since stiffness compatibility is essential to reduce masonry wall overstresses with rigid diaphragms (second floor level). The proposed new concrete shear wall system extends up to the roof level for consistency purposes, even though the roof is considered a flexible diaphragm.

An alternative vertical-lateral-resisting system can be explored during the Implementation Phase between second floor and roof level, since seismic loads to lateral systems are based upon diaphragm tributary spans, rather than vertical-lateral-system stiffness.

The new concrete shear walls are located on the north, east, and west exterior sides of the building walls at Grid Lines A, 1, and 7 respectively, and at the interior of the building between Grid Lines C and D and Grid Lines 2 and 6 (Figure 5-1 through Figure 5-6). All proposed new concrete shear walls will require new foundation work including drilled concrete caissons as indicated in Figure 5-3.

The new exterior shear wall and foundation systems are located directly between Grid Lines C and D (west and east walls) and Grid Lines 4 and 5 (north wall) resulting in new concrete caissons being in close proximity with the existing concrete pier foundation system. The new west and east walls would not be located in any drive-aisle access bay at the ground level parking. The new concrete shear walls will have their own integrated boundary elements at the ends of the walls and will not rely on the existing large rectangular piers as boundary elements. The shear walls will be slightly shorter in length than proposed Option 2 and will require deep concrete caissons to resist the seismic overturning forces.

The new interior shear wall and foundation systems are located along the main east-west gridlines and are offset from north-south gridlines, resulting in new concrete caissons being located further away from any existing foundation system. The walls are “L”-shaped in configuration and will have their own integrated boundary elements at the ends of the walls, thus, not relying on the existing round concrete columns as boundary elements. The shear walls will require deep concrete caissons to resist the large seismic overturning forces.

Temporary shoring will be required to install these new shear walls as decking must be cut and new ledger angles and bolts installed on each side of the new shear walls. The new shear walls will “flare-out” around the existing steel wide flange beams below the second floor and rood deck (for ledger angle support) and new weldable reinforcement bars and thru-flange bars added at the beams.

- **Reinforced Concrete Wall Overlays:** New reinforced concrete wall overlays are proposed to upgrade the existing masonry walls near the north and south sides of the building at ground level and second floor (Figures 5-3 and 5-5). The concrete overlays address residual overstresses in the masonry walls even with the added new concrete shear walls.

- **Diagonal Steel Diaphragm Bracing:** As indicated in Chapter 4, the second floor and roof level diaphragms are grossly overstressed based on BPOE level forces and inadequate diaphragm capacities. Additional new concrete shear walls are being added to reduce the diaphragm (and masonry wall) overstress; however, in order to reduce the diaphragm stresses to meet the ASCE/SEI 41-17 BPOE level, horizontal steel diaphragm bracing (tube or pipe) below the second floor and roof level is required (Figure 5-4 and Figure 5-6). The horizontal steel diaphragm bracing would be installed mid-depth of the existing wide flange members, with epoxy-anchor connections to masonry walls/concrete piers and welded/bolted connections to steel framing, at each end of the bracing.

  All plaster/gypsum board/acoustical ceilings and mechanical systems would have to be demolished or relocated to install the new horizontal diaphragm bracing. Other alternative diaphragm strengthening measures such as new slabs, FRP, etc. were explored; however, none of these are practical approaches based upon diaphragm overstress levels, additional weight to building, and diaphragm composition.

- **Collector/Drag Elements:** New and upgraded diaphragm collector/drag elements and connections are required in order to bolster the collector lines in-line with existing and new shear walls. Figure 5-4 and Figure 5-6 reflect the new and upgraded members and connection locations.
- **Out-of-Plane Wall-to-Diaphragm Connections:** The exterior double-wythe masonry shear walls at the northwest and southeast corners of the building (27-inch thick walls with cladding) require new out-of-plane connections and steel members (where noted) at the second floor and roof levels in order to prevent the exterior walls from separating from the diaphragm during a seismic event (Figure 5-4 and Figure 5-6). In addition, it appears based on recent photographs provided by the County that the north masonry wall near the Lobby stairway may not be braced laterally. New lateral bracing and connections are required for this wall.
- **Out-of-Plane Wall Strength Connections:** The existing tall masonry wall at the north side of the lobby stairway (unrestrained height between 25 feet to 30 feet directly adjacent to the landing and stairs) is an 8-inch reinforced masonry block with 2 curtains of No. 5 vertical reinforcement bar at 12 inches on center. Due to the excessive wall height at this location, several new vertical steel strong-backs are required at the lobby stairway wall to provide increased flexural capacity of the wall in the out-of-plane direction. These steel strong-backs would be anchored to the existing masonry wall with epoxy anchors at four to five feet on center along the height of the member and would include connections to the diaphragm above and below.

Refer to Figure 5-1 through Figure 5-10 for floor plans and elevations for Option No. 1 building strengthening.

Typical concrete details are included in Figure 5-20 through Figure 5-23.

5.3.2 Building Strengthening Option No. 2

Building Strengthening Option No. 2 is similar to Option No. 1 with the exception of the locations of the proposed interior and exterior concrete shear walls (Figure 5-11). The 10-inch-thick exterior shear walls are located such that the walls are slightly longer in length than Option No. 1 shear walls and “straddle” between two building bays along the west, east, and north sides. The exterior shear wall locations in this option are more favorable structurally since caissons are not in close proximity with existing pier footings and the walls can be slightly longer to minimize overturning issues. However, the proposed shear wall locations in this option impact drive-aisle access at the west and east sides of the building along Grid Lines 1 and 7.

The 10-inch-thick interior shear walls, similar to Option No. 1, are “L”-shaped in configuration and are located between Grid Lines C and D and Grid Lines 2.5 and 5.5 (Figure 5-12 through Figure 5-19). The east-west direction shear walls in this option are located at Grid Lines C and D.

All other proposed strengthening concepts in Option No. 2 are identical to Option No. 1, as shown in Figure 5-12 through Figure 5-19.

Typical concrete details are included in Figure 5-20 through Figure 5-23.

5.4 Non-Structural Items

Survey and analysis of non-structural components including partition walls, ceilings, and MEP equipment was outside the scope of work for this report and was not conducted since the County will completely renovate the interior space of the building. However, the following are examples of typical recommendations for strengthening non-structural items to a “Life Safety” and “Collapse Prevention” performance levels if the renovation work results in select elements remaining.

5.4.1 Partition Walls

For any existing partition walls that will remain during the tenant improvement project, install additional diagonal bracing at the top of the partition walls to prevent them from toppling and/or excessively leaning after a significant seismic event. Pay particular attention to partition walls near exits.

5.4.2 Ceiling Systems

For any existing ceilings that will remain during the tenant improvement project, install required vertical compression strut and diagonal bracing in the ceiling system to minimize potential ceiling damage. Install missing wire hangers for all ceiling mounted lights and air vents.

5.4.3 Non-Structural Equipment

Perform an engineering review of piping, ducting and equipment anchorage and install anchoring/bracing systems for these components.

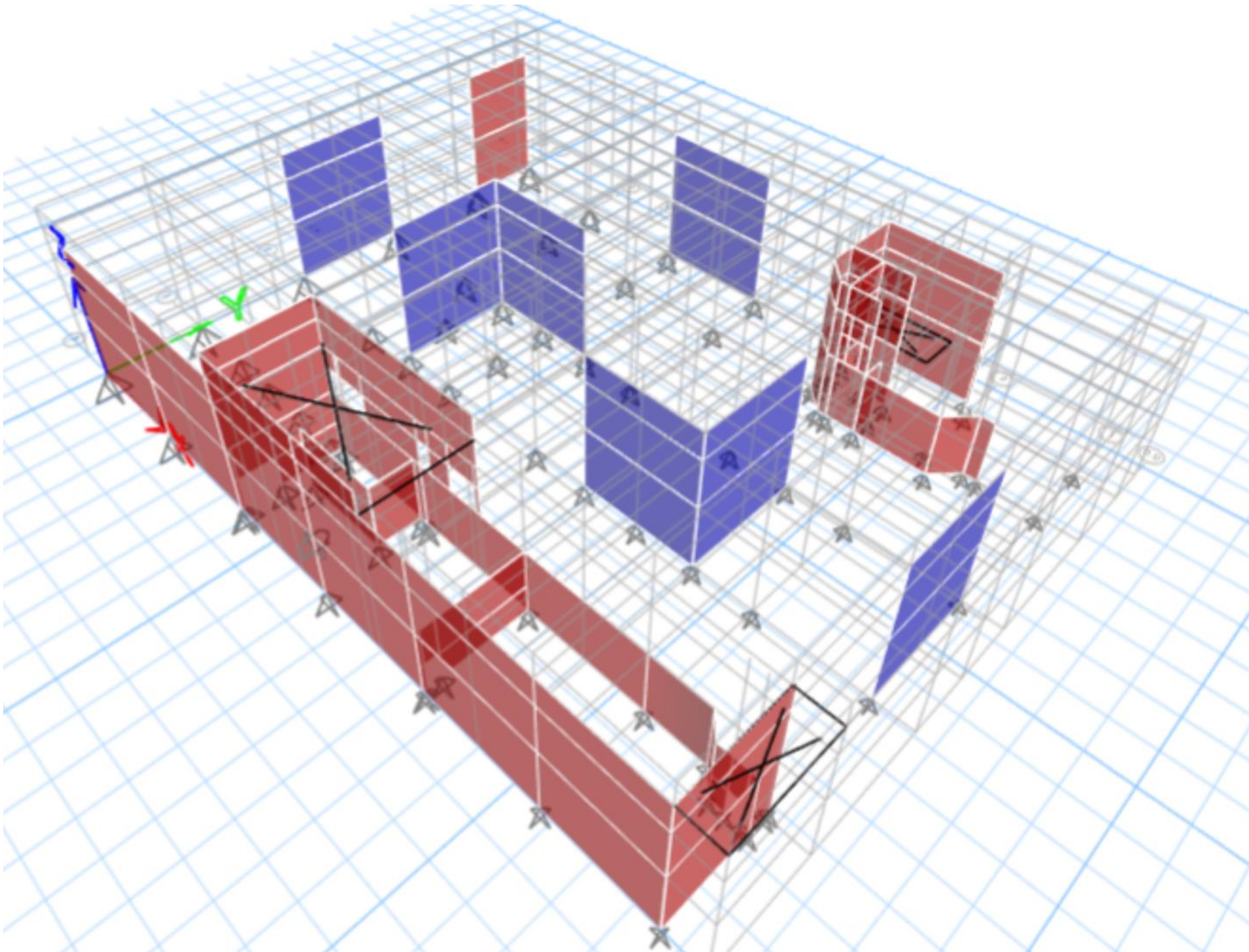


Figure 5-1: Option No. 1 – ETABS Model – Looking Northwest

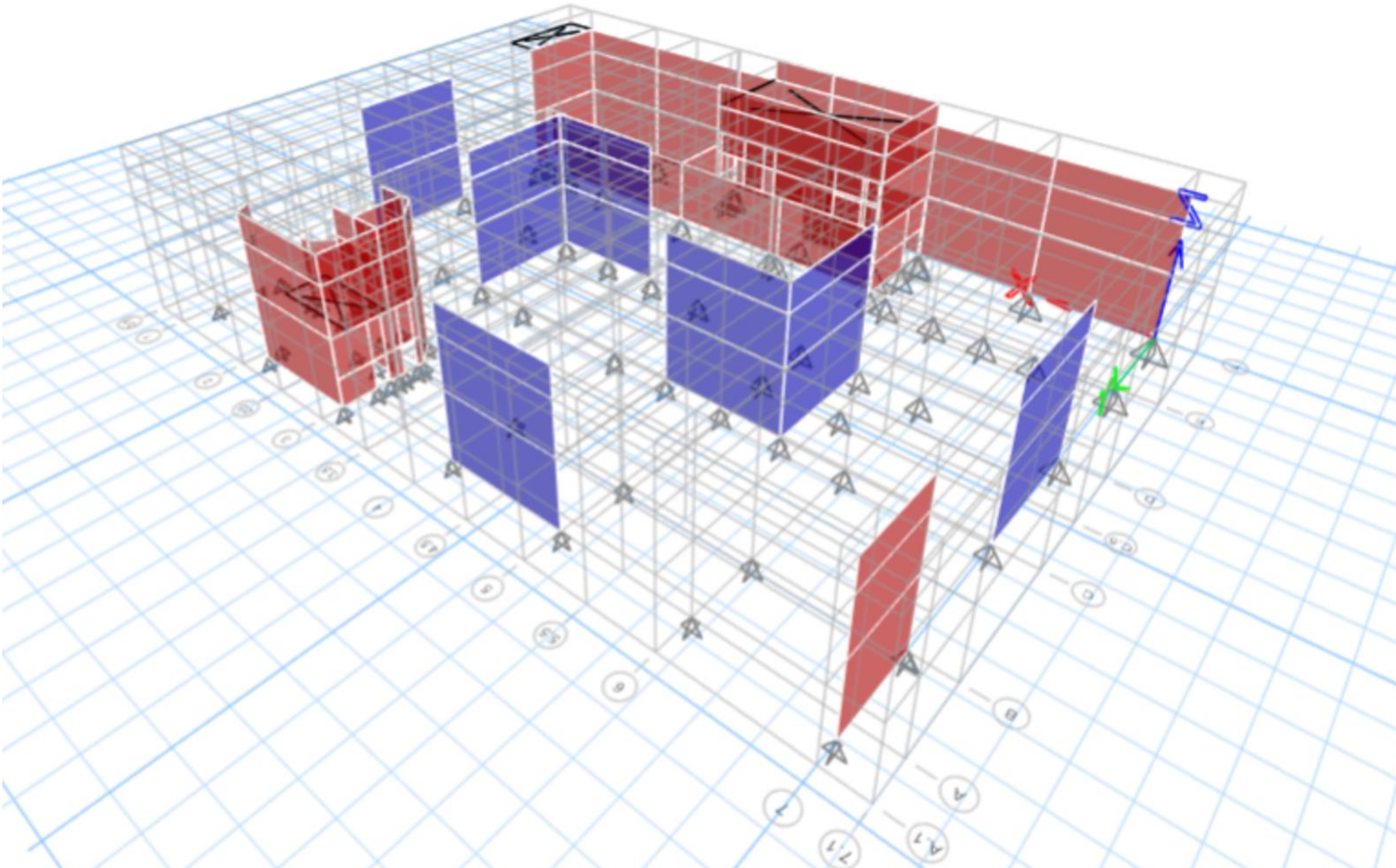


Figure 5-2: Option No. 1 – ETABS Model – Looking Southeast

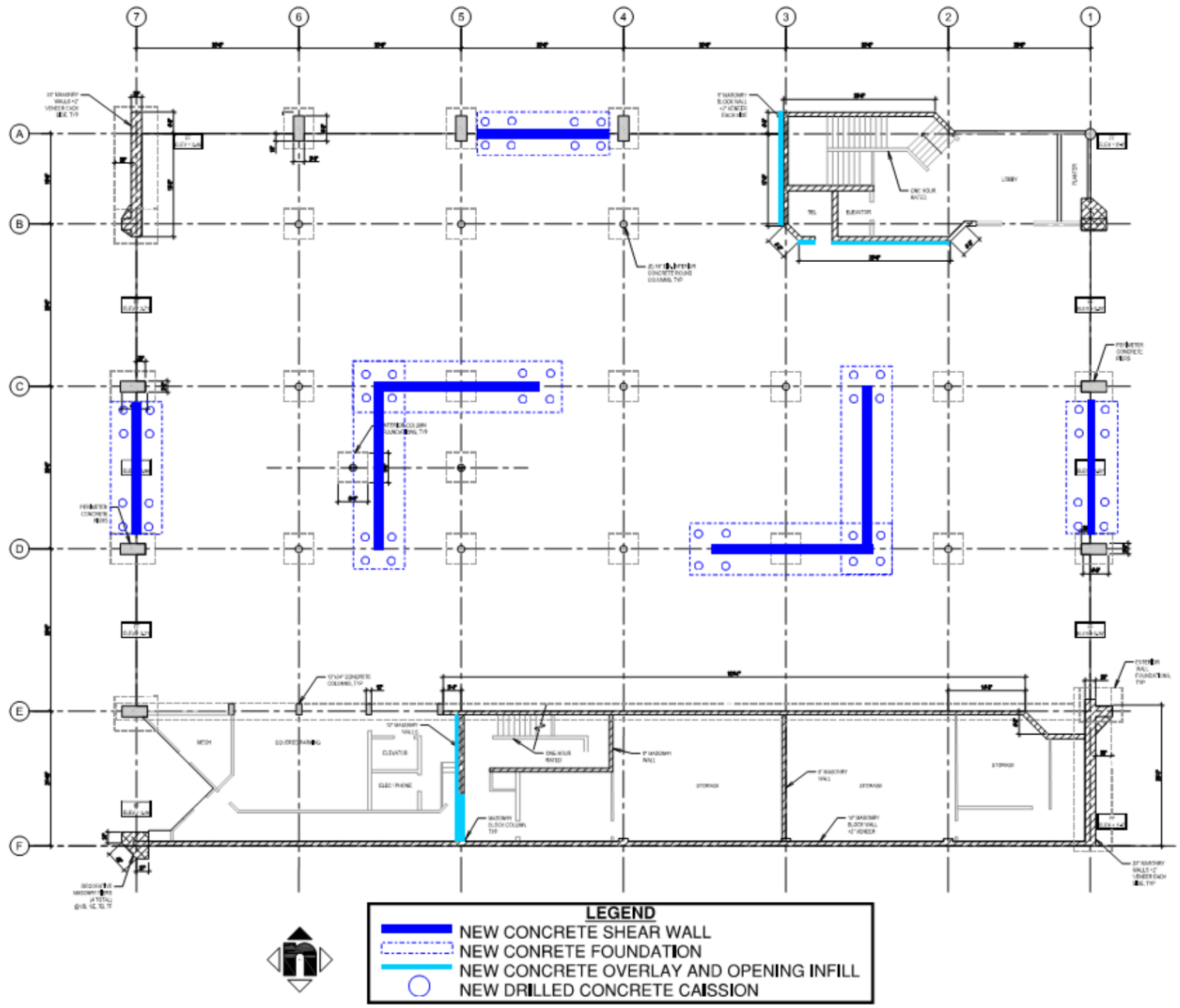


Figure 5-3: Option No. 1 – Ground Level Strengthening

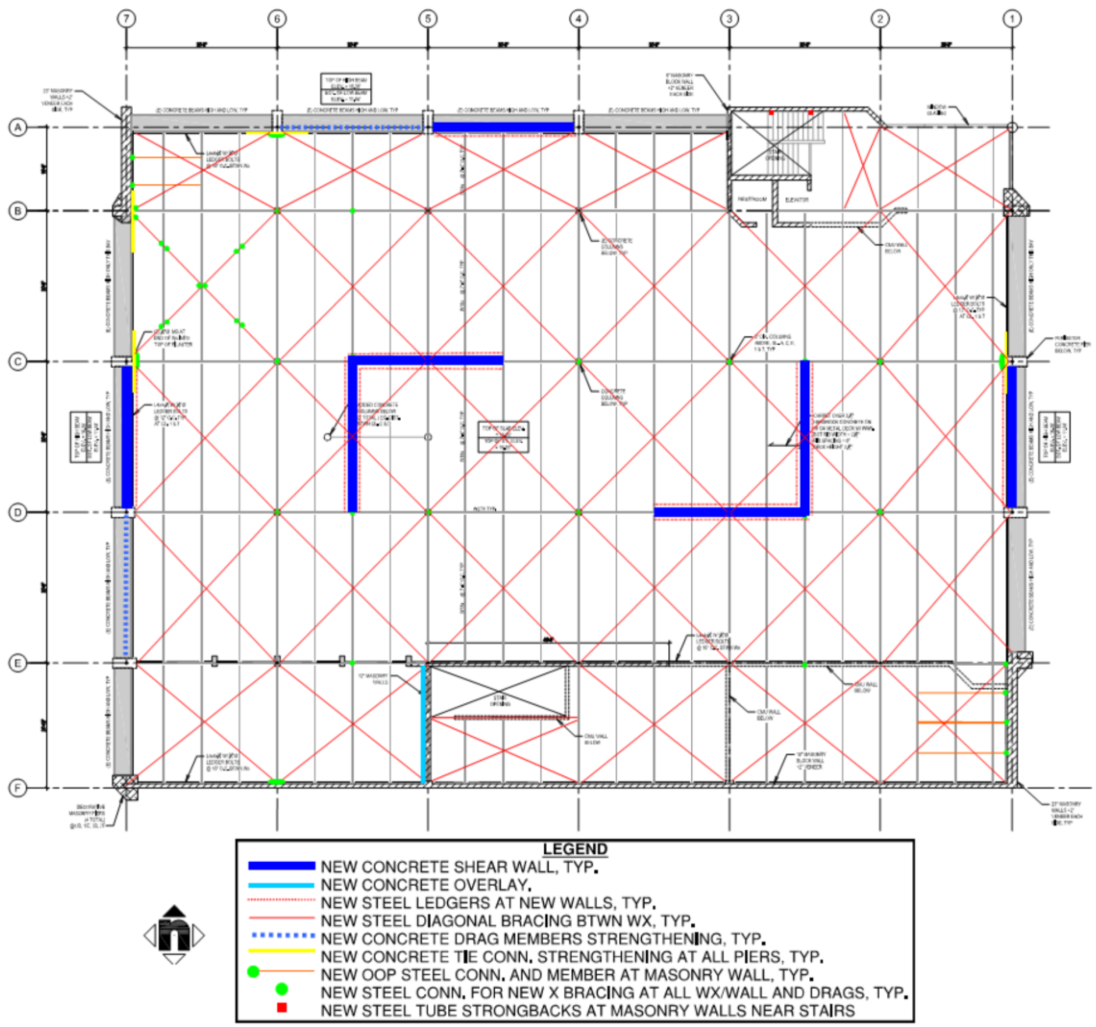


Figure 5-4: Option No. 1 – Second Floor Strengthening

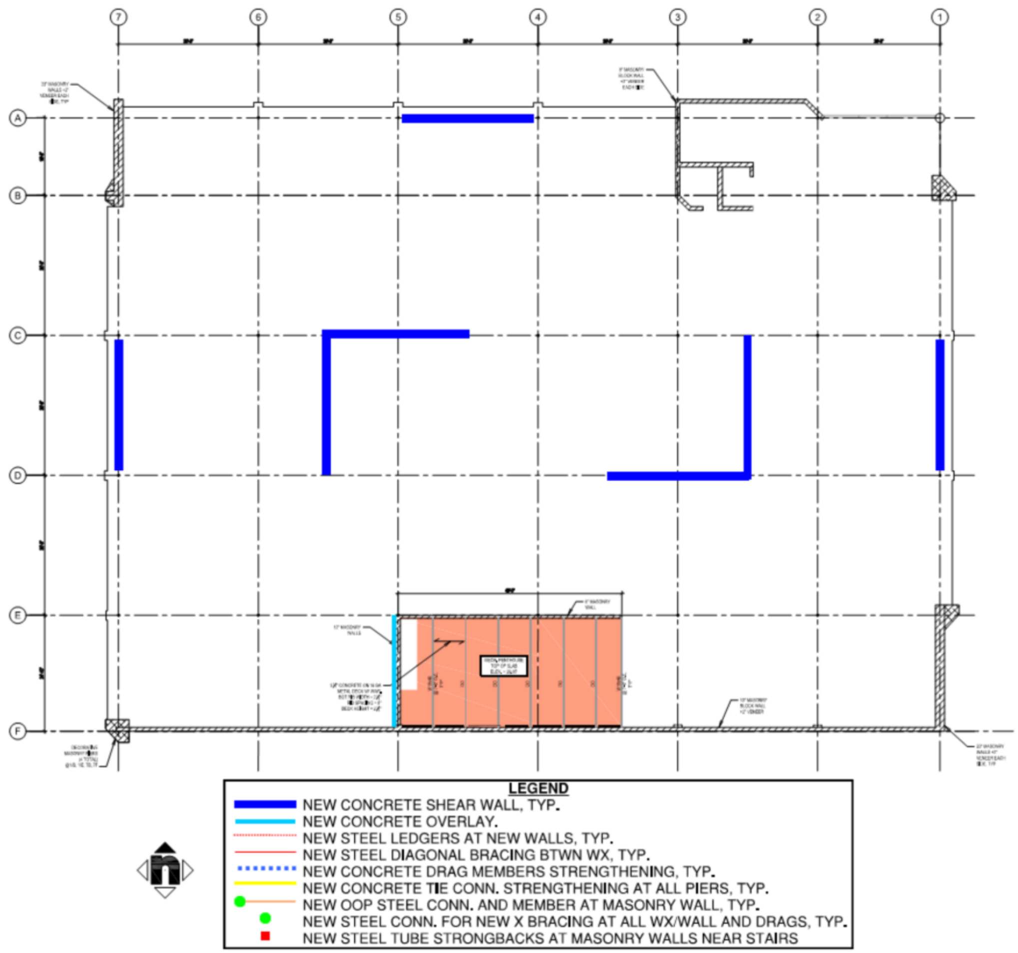


Figure 5-5: Option No. 1 – Mezzanine Level Strengthening

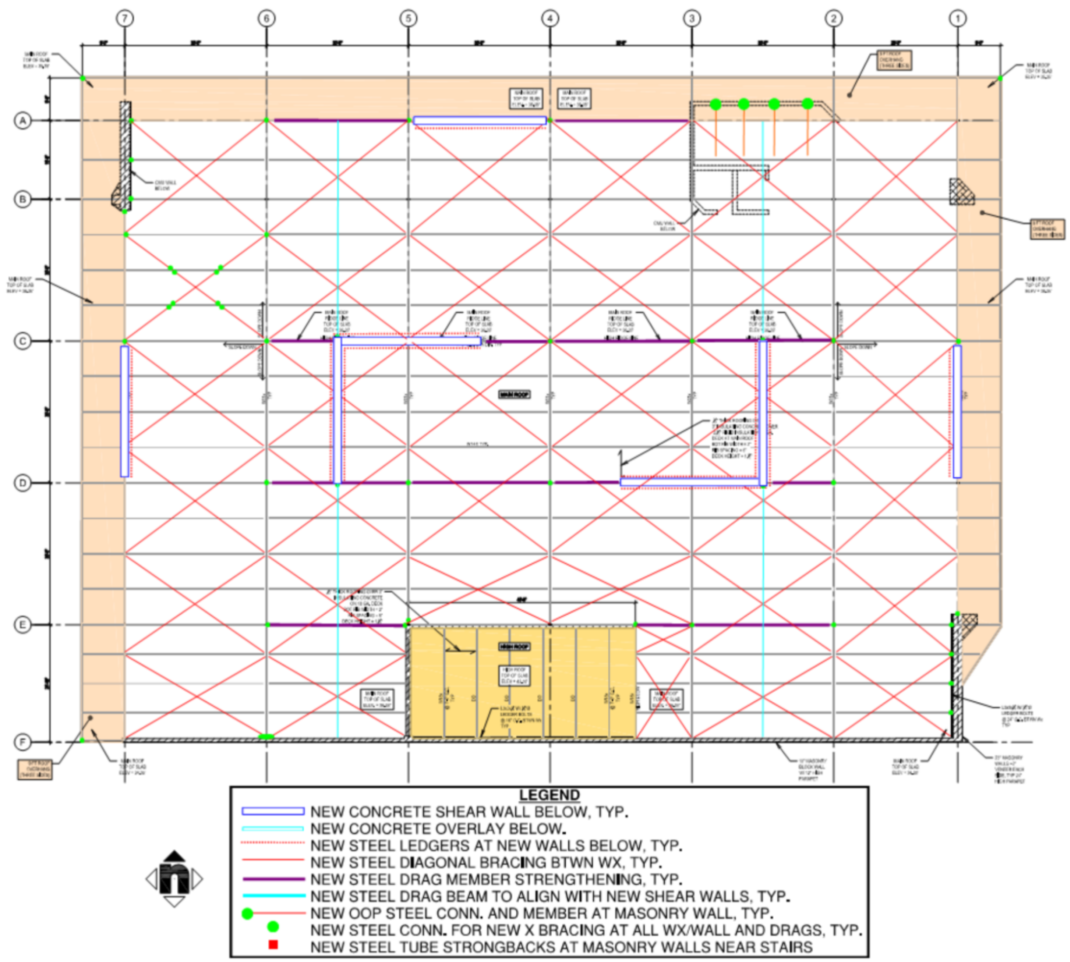


Figure 5-6: Option No. 1 – Roof Level Strengthening

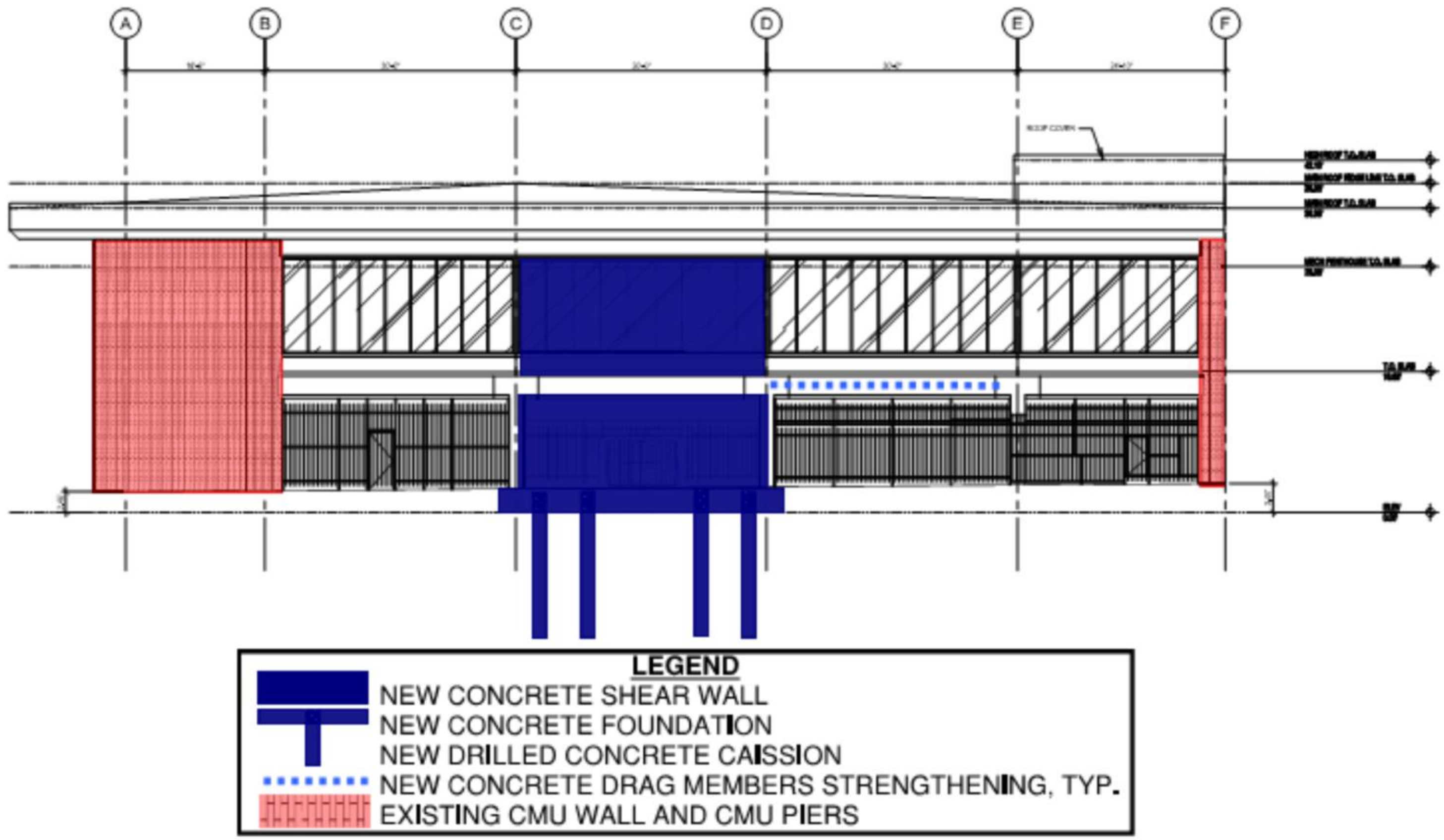


Figure 5-7: Option No. 1 – West Elevation Strengthening

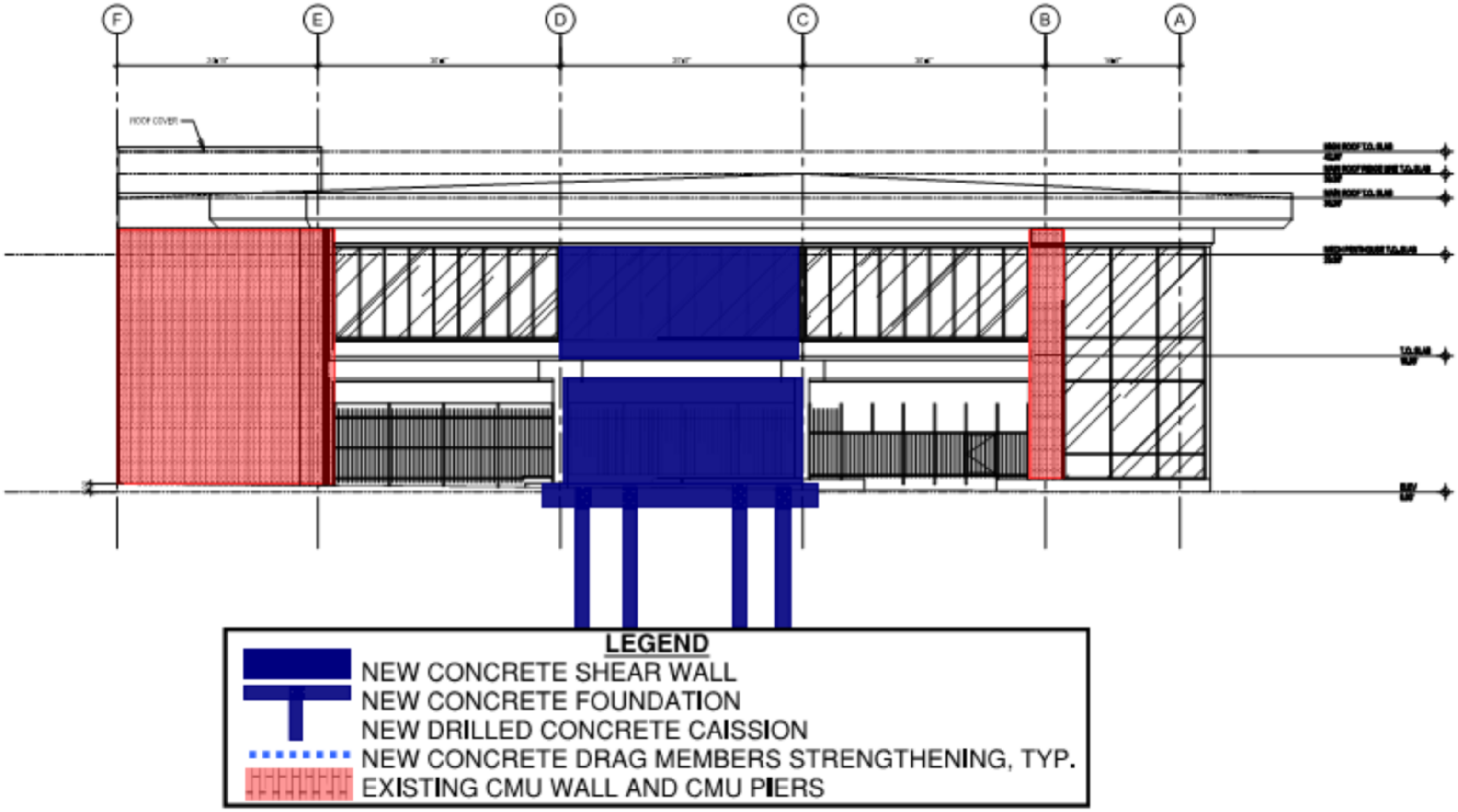


Figure 5-8: Option No. 1 – East Elevation Strengthening

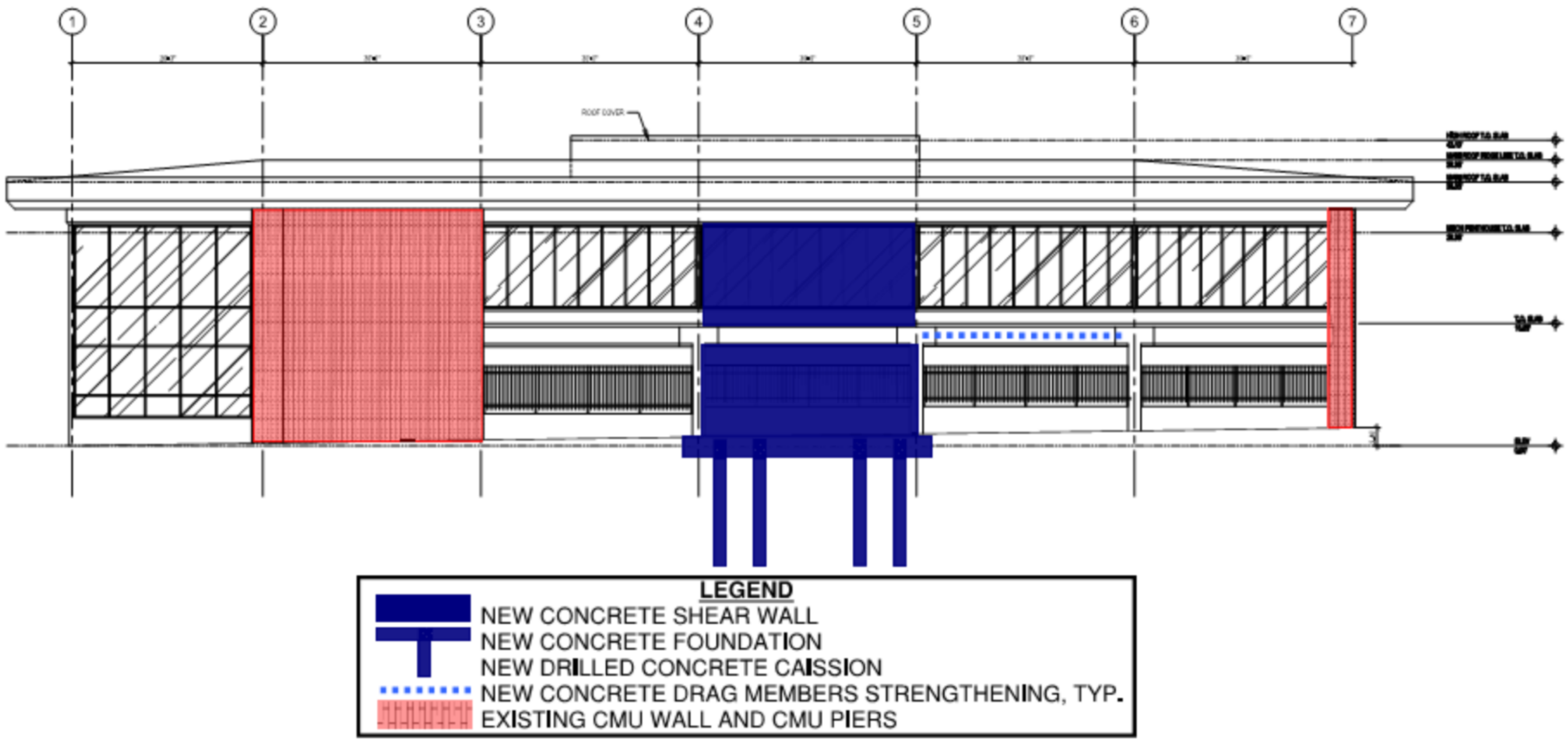


Figure 5-9: Option No. 1 – North Elevation Strengthening

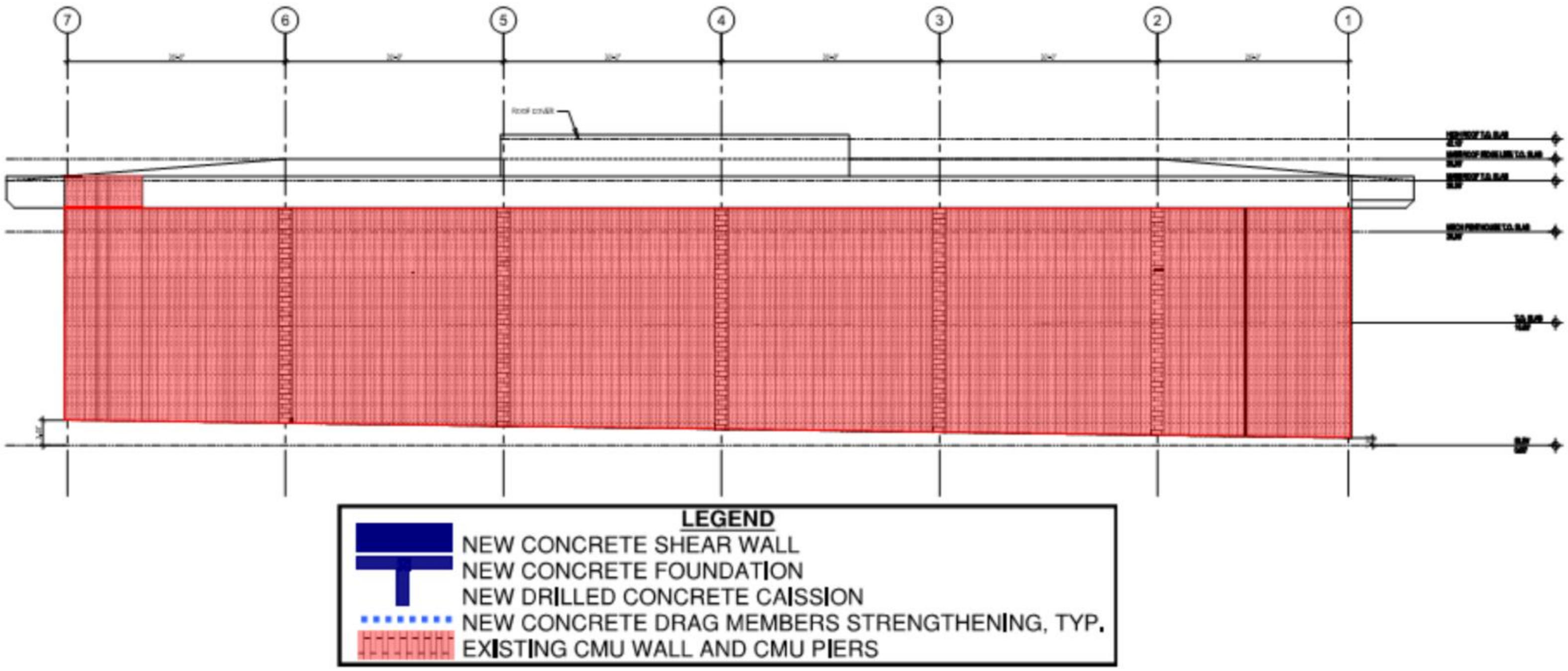


Figure 5-10: Option No. 1 – South Elevation – No Strengthening

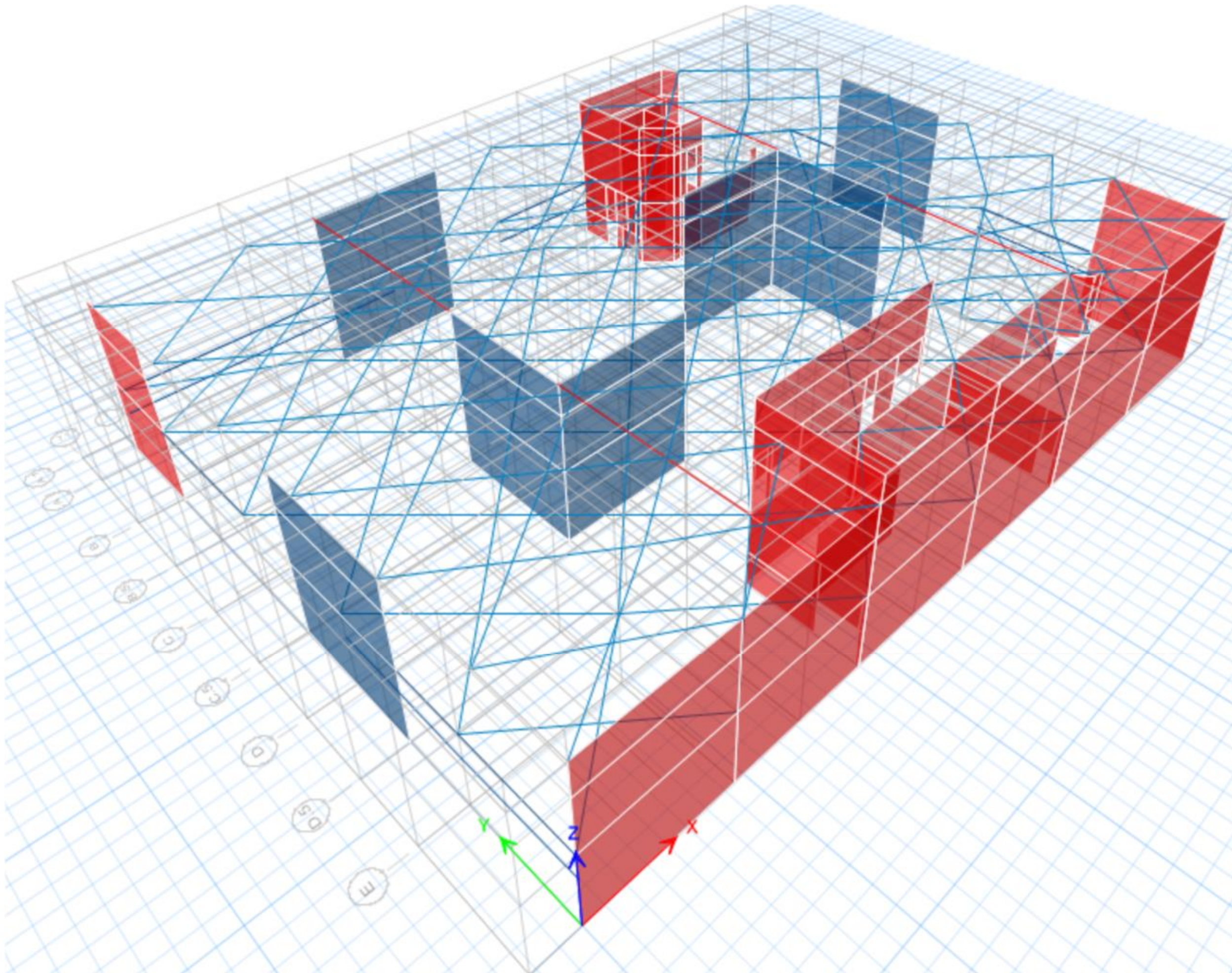


Figure 5-11: Option No. 2 – ETABS Model – Looking Northeast with X-Bracing

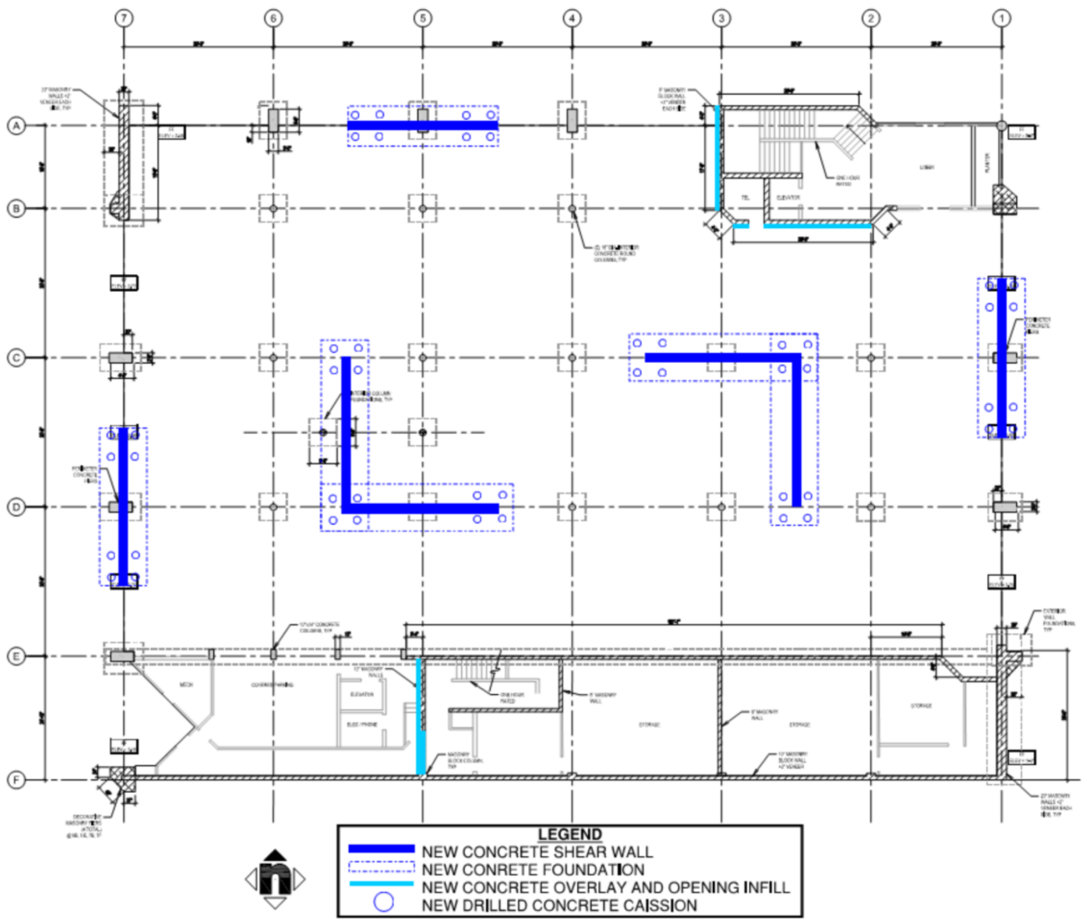


Figure 5-12: Option No. 2 – Ground Level Strengthening

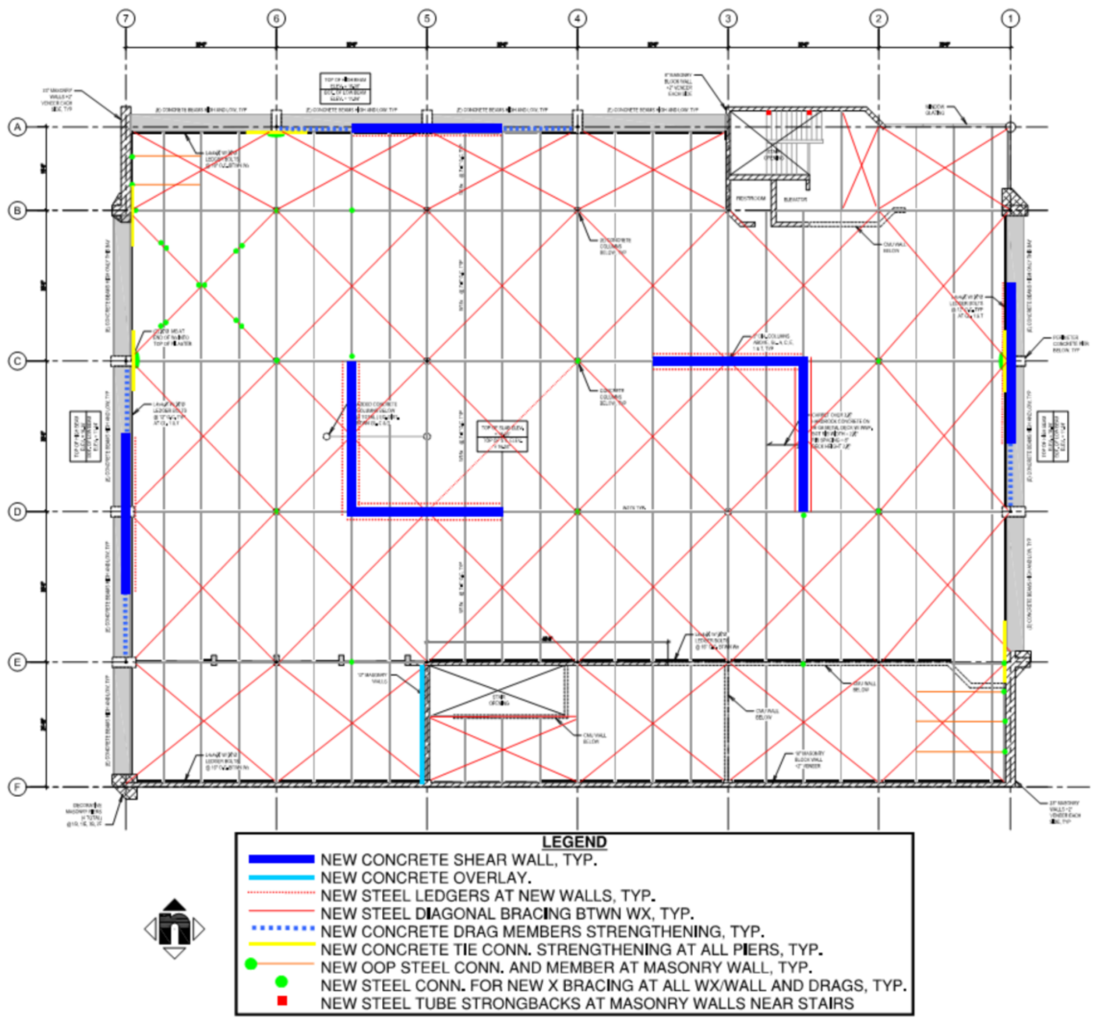


Figure 5-13: Option No. 2 – Second Floor Strengthening

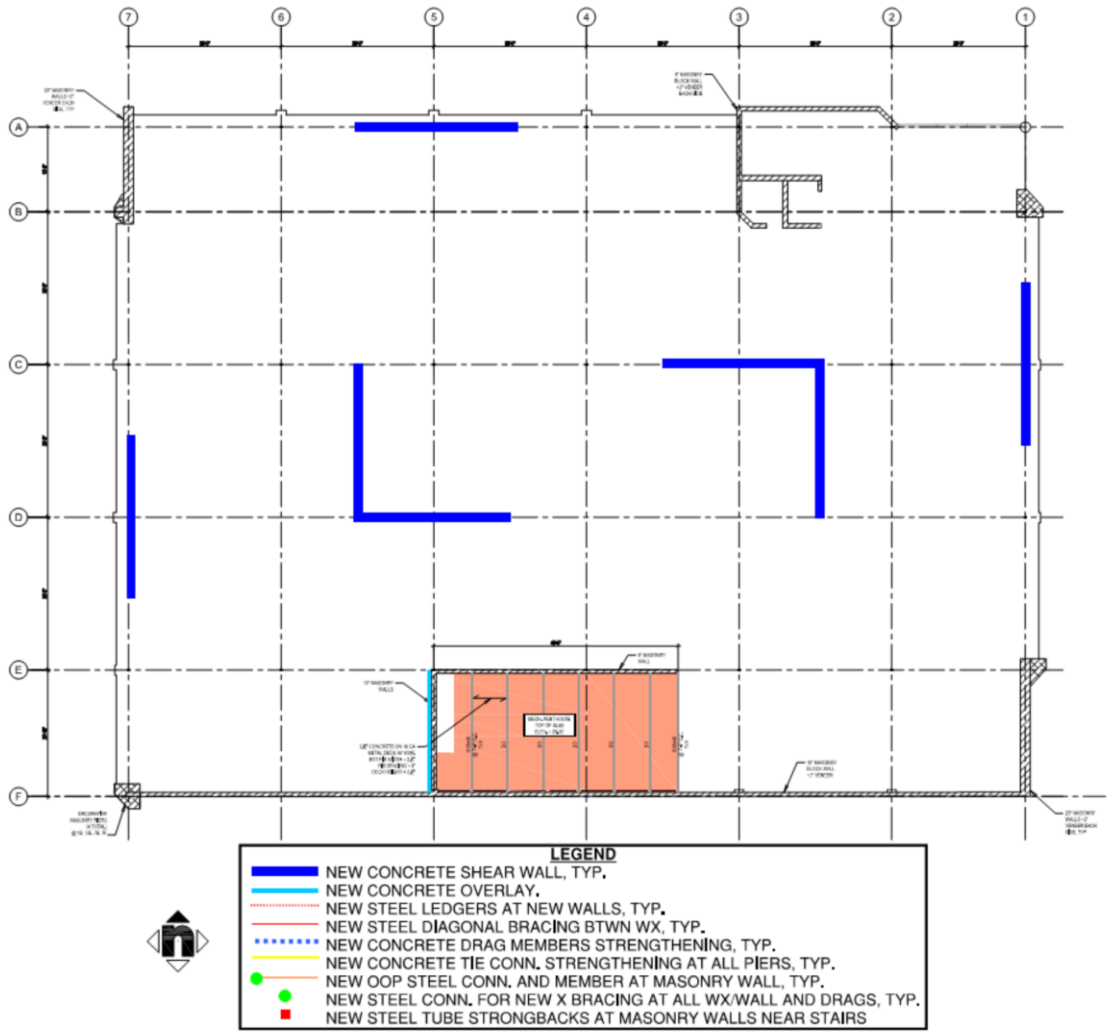


Figure 5-14: Option No. 2 – Mezzanine Level Strengthening

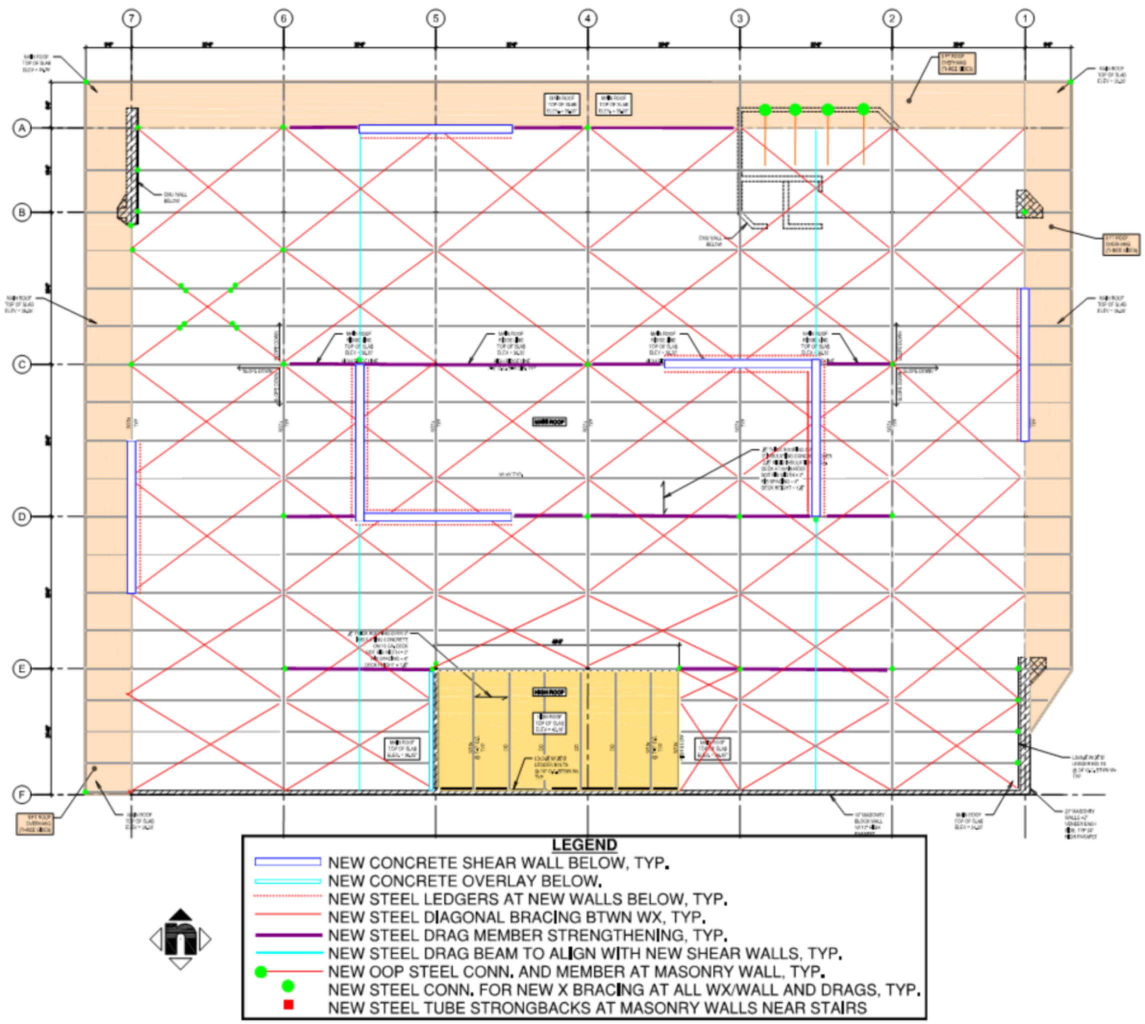


Figure 5-15: Option No. 2 – Roof Level Strengthening

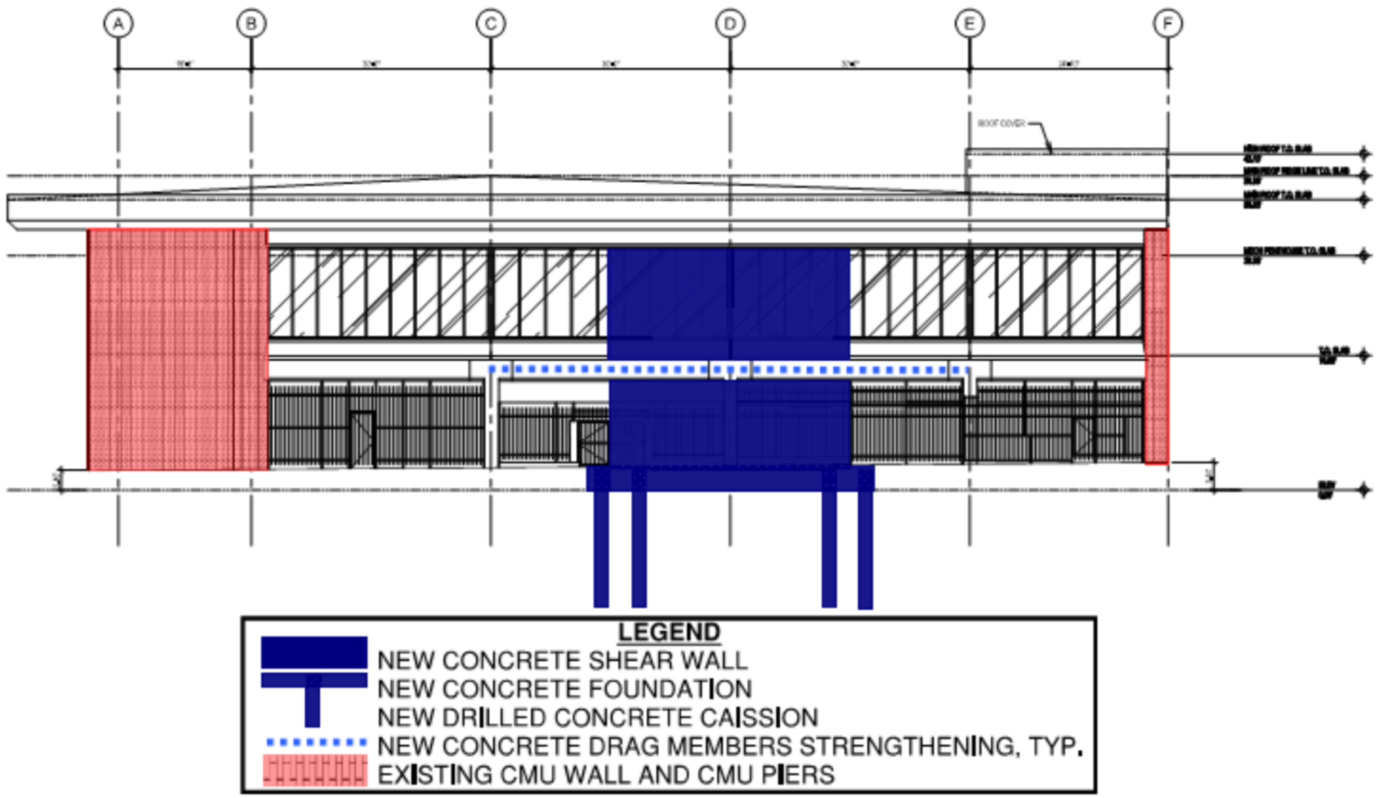


Figure 5-16: Option No. 2 – West Elevation Strengthening

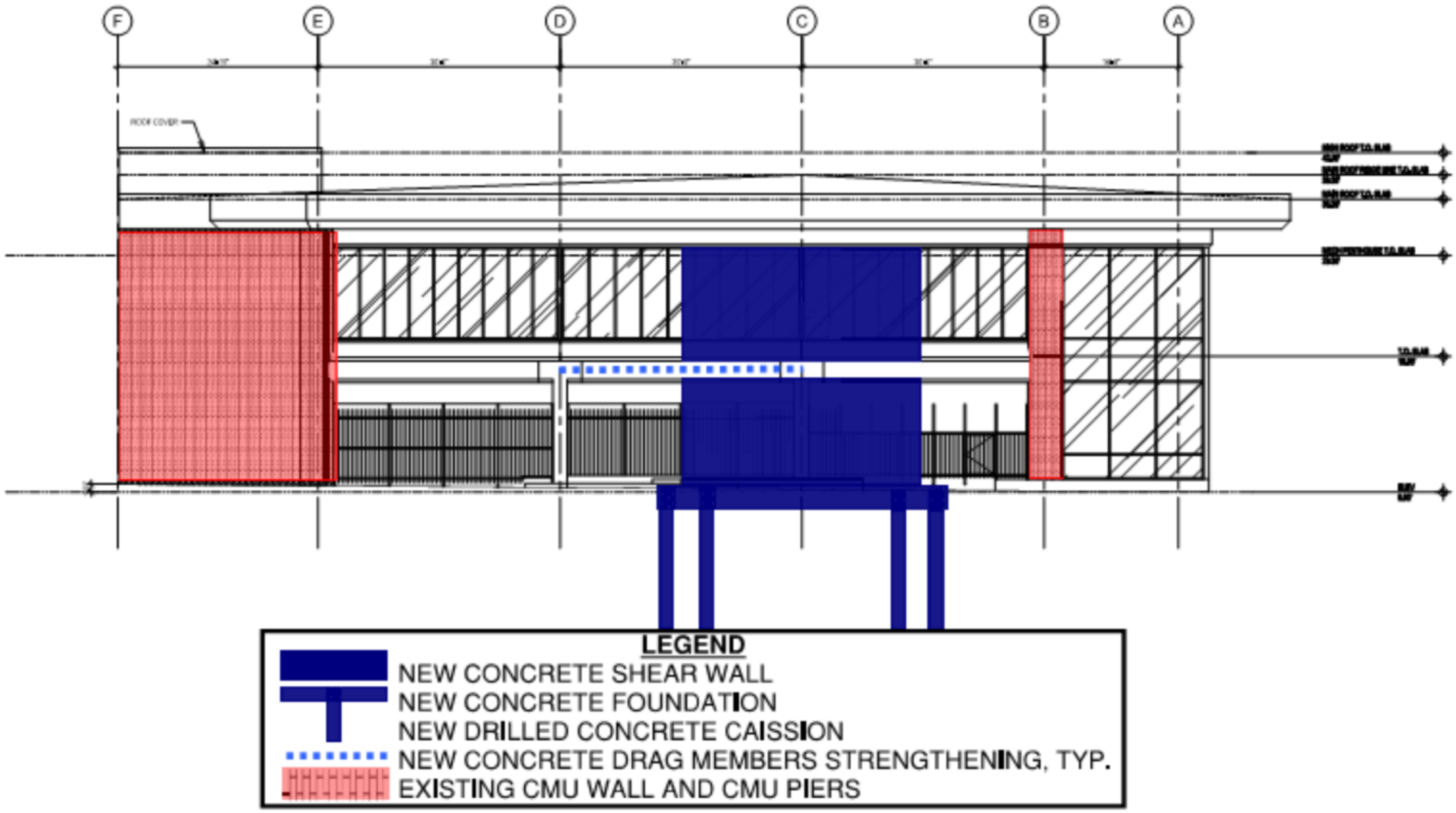


Figure 5-17: Option No. 2 – East Elevation Strengthening

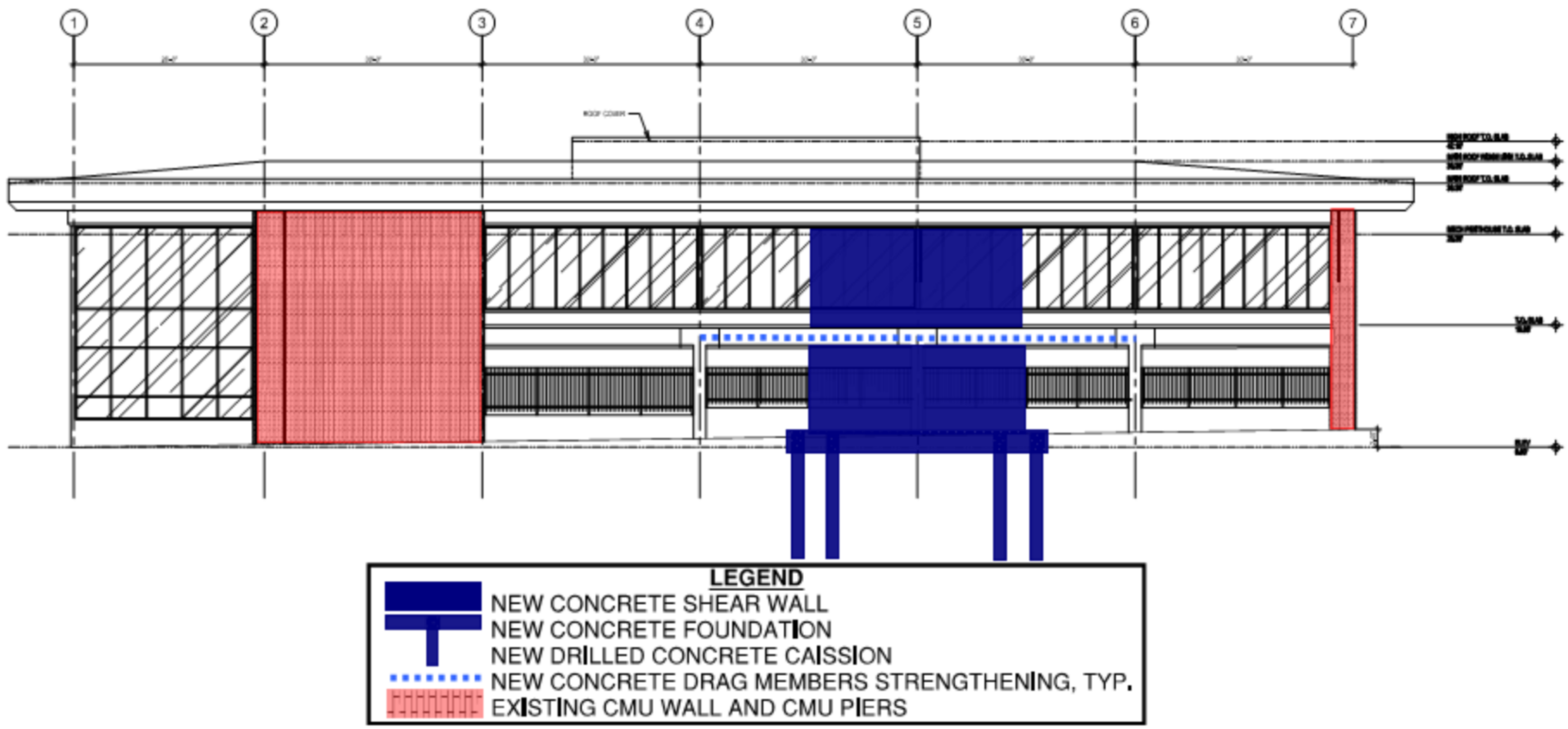


Figure 5-18: Option No. 2 – North Elevation Strengthening

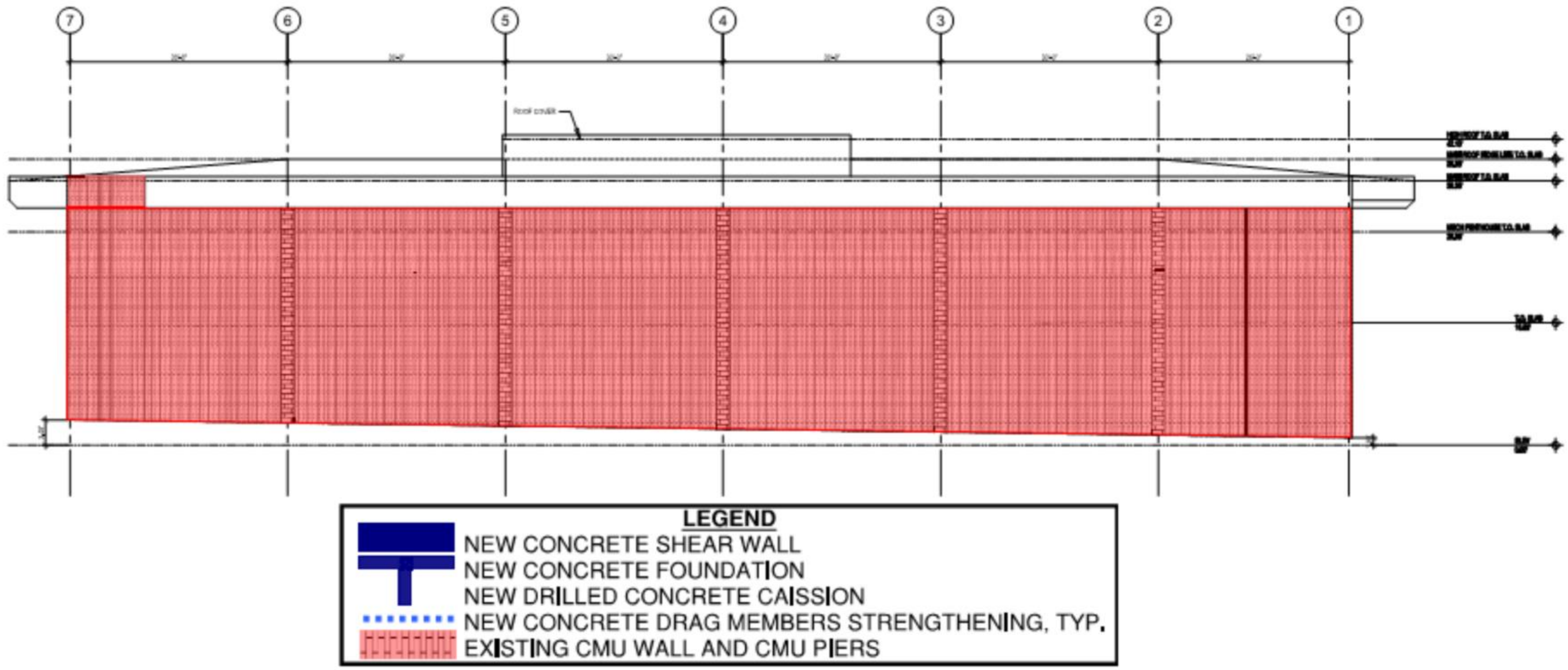


Figure 5-19: Option No. 2 – South Elevation - No Strengthening

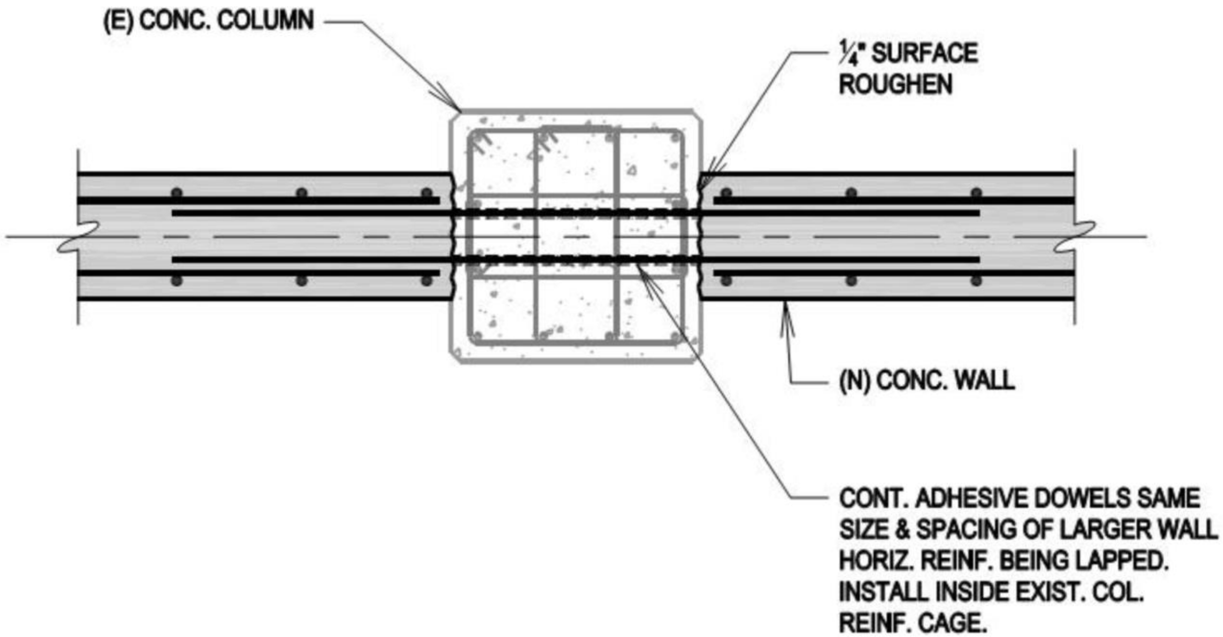


Figure 5-20: Example Detail of Shear Wall/Column Dowel Connection

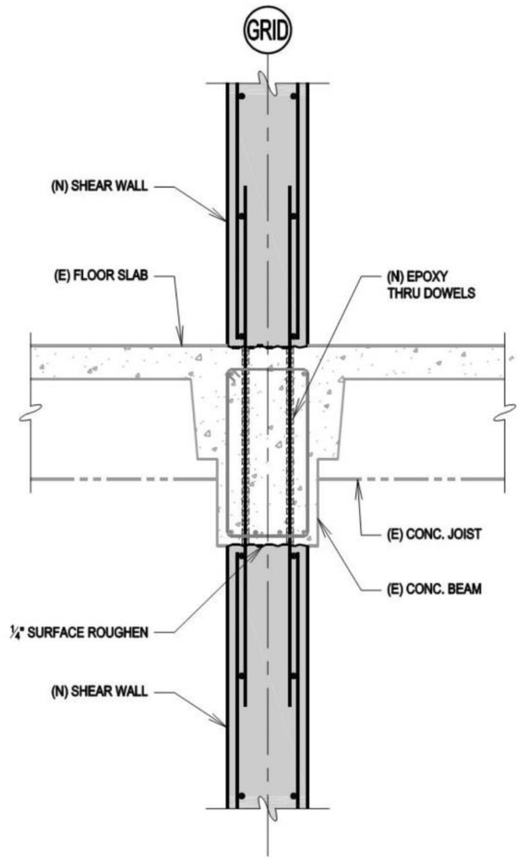


Figure 5-21: Example Detail of Shear Wall/Beam Dowel Connection

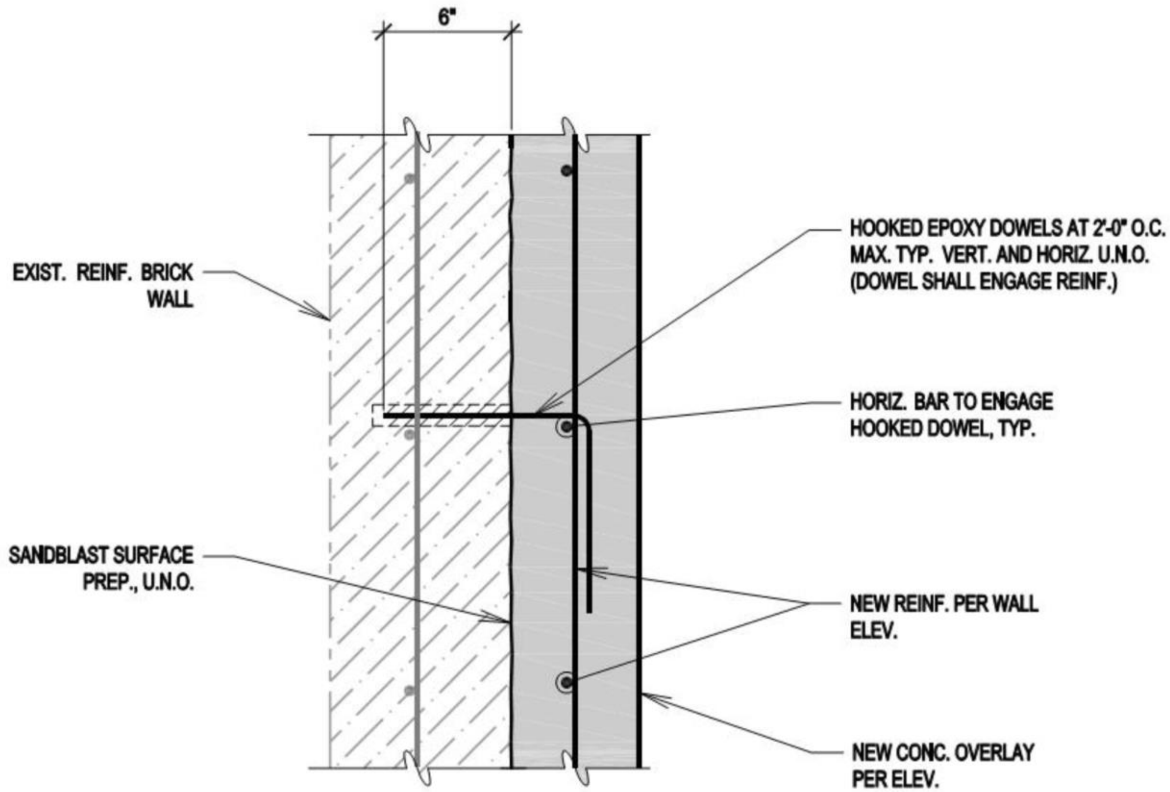


Figure 5-22: Example Detail of Concrete Overlay

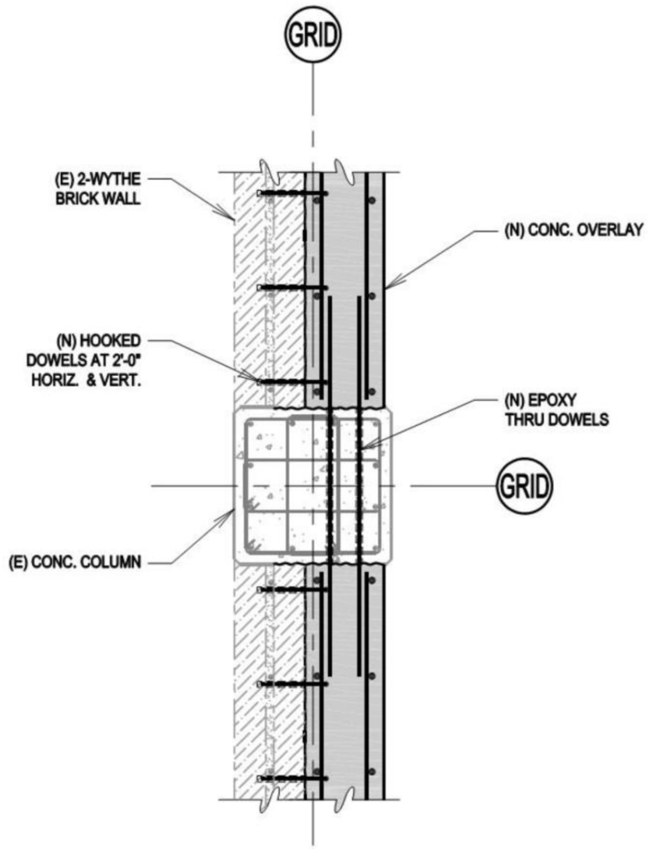


Figure 5-23: Example Detail of Concrete Overlay