Rating form completed by:

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Text in green is to be part of UCSF building database and may be part of UCOP database.

DATE: 2020-06-19

06/19/2020

UCSF building seismic ratings

Oyster Point

CAAN #2418
612 Forbes Boulevard, South San Francisco
UCSF Campus: Outlying Area

Plan

Elevation (looking south)

Rating summary	Entry	Notes
UC Seismic Performance Level (rating)	V	Findings based on drawing review, site visit, and ASCE 41-17 Tier 1 evaluation[1]
Rating basis	Tier 1	ASCE 41-17
Date of rating	2020	
Recommended UCSF priority category for retrofit	A	Priority A=Retrofit ASAP Priority B=Retrofit at next permit application for modification
Ballpark total project cost to retrofit to IV rating	Medium	See recommendations on further evaluation and retrofit.
Is 2018-2019 rating required by UCOP?	Yes	
Further evaluation recommended?	Yes	Tier 2 deficiency-based check of CMU portion of building

[1] The evaluations at UCSF translate the Tier 1 evaluation to a Seismic Performance Level rating using professional judgment discussed among the Seismic Review Committee. Non-compliant items in the Tier 1 evaluation do not automatically put a building into a particular rating category, but such items are evaluated along with the combination of building features and potential deficiencies, focused on the potential for collapse or serious damage to the gravity supporting structure that may threaten occupant safety.

Building information used in this evaluation

- Architectural Drawings by Avanessian Associates Architect, "Fairbanks Ward, 612 Forbes Boulevard, South San Francisco," dated 14 May 1973. Sheets 1, 3-10, 12-17 (15 drawings).
- Architectural Drawings by Robert C. Davidson Architect AIA & Associates, "UCSF Purchasing Department 626 Forbes Boulevard Oyster Point Warehouse S. San Francisco, Calif.", dated 7 January 1980. Sheets A1-A8 (8 drawings).
- Architectural and Structural Drawings by Robert C. Davidson Architect AIA & Associates, "Phase 2 UCSF Purchasing Department 626 Forbes Boulevard Oyster Point Warehouse S. San Francisco, Calif.", dated 18 August 1980. Sheets A1-A6, S1-S2 (8 drawings).
- Structural Drawings by H.J. Degenkolb Associates Engineers, "UCSF Warehouse Facility," dated 1 August 1985, Sheets S-1, S2, R-S1 (3 drawings).
- Structural Drawings by Wilddeck Mezzanines, "University of California – S.F.," 15 June 1989, Sheets 2 to 4 (3 drawings).
- Structural Drawings by Rutherford and Chekene Consulting Engineers, "Oyster Point Seismic Improvements," 23 December 1998, Sheets 1S to S8 (8 drawings).
- Geotechnical Investigation Report by Rutherford and Chekene, "Geotechnical Investigation. UCSF Oyster Point Seismic Upgrade," 15 February 1996.

Additional building information known to exist

- Structural Drawings by KPFF, "M4630E Oyster Point Warehouse Solar PV," 26 July 2018, Sheets S001 to S701 (8 drawings).

Scope for completing this form

Review of existing structural and architectural drawings served as the basis for completing the ASCE 41-17 Tier 1 evaluation. SRC member Charles Thiel visited the site June 6, 2019. Site visits have been completed by Estructure on numerous occasions for various projects. Alix Kottke visited the site on September 16, 2019.

Brief description of structure

The Oyster Point building is located on the south side of Forbes Boulevard in the City of South San Francisco. The building is a concrete tilt-up building with a wood framed roof. It serves primarily as a shipping and receiving warehouse and storage facility. It also contains a large freezer farm containing research specimens. It was constructed in the mid-1970s according to plans by Avanessian Associates AIA Architect. The building underwent a voluntary seismic retrofit in the mid-1980s in accordance with plans prepared by H.J. Degenkolb Associates Engineers. A subsequent seismic retrofit was completed in the late-1990s according to plans prepared by Rutherford and Chekene Consulting Engineers.

The building is L-shaped in plan structure with approximate overall dimensions of 353 feet by 368 feet. The west segment of the L between lines A-L and 1-11 measures in plan 243 feet by 368 feet, and the east segment between lines A-F and 11-17 measures 80 feet by 300 feet.

The building contains two mezzanines. One unused mezzanine, referred to as Mezzanine 1 in this report, between gridline A-F and 15-17 is framed with wood and was part of the original building construction. A second steel framed mezzanine, referred to as Mezzanine 2 in this report, was added in the late 1980s, located between gridline J-L and 1-11. (Refer to Figures 2 and 3 for grid line references)

There is also a small CMU office at the inside corner of the L of the building (approximately between gridline 9-11 and F-H) which was built as hazardous material storage when UCSF purchased the building around 1980.

Identification of levels:

The site slopes gently to the south with the first floor approximately 4 feet above finished grade to the north at the parking lot where the loading docks occur and even with the finished grade at the south side of the building. The building is mostly a one-story structure with the roof approximately 22'-6.75" above the first floor at gridline I. The existing building drawings call for the roof beams to camber and post heights vary by 2'-3" to achieve required roof slope for drainage. Mezzanine 1 floor framing is 13'-5.75" above the first floor. Mezzanine 2 is 11'-11" above the first floor. The roof of the CMU structure is 13'-2" above the slab.

Foundation system:

Foundation support is provided by shallow reinforced concrete spread footings and perimeter grade beams. The perimeter wall panels are anchored to the concrete slab-on-grade with bent reinforcing dowels spaced at three feet on center. Pilasters are positively doweled into concrete footings. The first floor is a 5" thick concrete slab-on-grade reinforced with one layer of welded wire fabric.

Structural system for vertical (gravity) load:

The roof is constructed of ½" structural wood panel (plywood) deck that is supported on 2x4 sub-purlins at 24" on center spanning to 4x14 purlins at 8' on center spanning to glulam beam girders at 24'-2" on center. The roof beams are supported on 6" diameter interior steel pipe columns and reinforced concrete pilasters of the perimeter walls that are parts of the individual wall panels.

Mezzanine 1 (between lines 15-17) includes a structural wood panel (plywood) deck that is supported on 4x12 joists at 16" on center spanning to perimeter reinforced concrete tilt-up walls and glulam beam girders. The glulam beams are supported on glulam posts.

Mezzanine 2 (between lines J-L and 1-11) floor is framed with plywood over 1 ½" x 20 gauge metal deck supported by steel open-web joists. The joists span to steel wide flanged beams supported by tube steel columns.

The CMU office (between gridline 9-11 and F-H) is constructed with 8" concrete masonry units over a 6" high concrete curb. Existing drawings indicate the walls to be reinforced and call out the vertical reinforcement as #4 at 32". The existing drawings do not list the horizontal reinforcement or if the wall is fully or partially grouted. The roof of this portion of the structure consists of plywood and gypsum board over cold formed steel joists. The joists are typically 10" deep x 16 gauge at 24" on center. The joists sit on top of the CMU wall and expansion anchor into the top of the wall.

Structural system for lateral forces:

The structural wood panel roof and mezzanine floor diaphragms distribute earthquake loads to the perimeter reinforced concrete shear walls. Concrete walls are either 6" or 7.5" thick with a single layer of reinforcement, typically #4 bars at 12" on center each way. The wall panels are interconnected at the pilasters with double #11 bars at first floor level and at the top of the panel, above the plane of the roof diaphragm. The interconnection at the top of the panel is made by welding the chord bar ends to 15" long steel angles, and at the floor level by a 28-bar diameter lap splice.

A 1985 voluntary seismic retrofit designed by H.J. Degenkolb Associates, Engineers consisted of adding out-of-plane wall anchorage at the glulam beams and purlins and providing continuous ties at glulam beams purlins.

A 1998 seismic retrofit, designed by Rutherford & Chekene, was also a voluntary retrofit with the intention to improve the seismic performance of the building from "Poor" to "Good" in accordance with the UCSF Seismic Safety Guidelines at the time. The retrofit included infilling select existing wall openings with shotcrete, improving

diaphragm continuity by adding ties at roof purlins, adding out-of-plane wall anchorage at perimeter concrete walls, and adding a steel collector to address the re-entrant corner on GL F. The retrofit also included the addition of two concrete buttresses along gridline F, one at gridline 15 and one between GL 12 and 13. A steel collector was added at the roof diaphragm to transfer load into each buttress.

The 1998 seismic retrofit also addressed deficiencies in Mezzanine 2. Some of the framing was converted into steel moment frames and at other locations knee-braces were added.

The lateral system of the CMU office relies on the plywood diaphragm to distribute load to CMU shear walls. The size and spacing of the expansion anchors connecting the roof framing to the top of the wall is not specified on the drawings.

Building code: The original building drawings are dated 1973 but do not contain a building code reference. The building is presumed to have been designed in accordance with the 1970 UBC. The 1985 voluntary partial seismic retrofit did not list a building code. The 1997 voluntary partial seismic retrofit was designed in accordance with the 1995 CBC.

Building condition: Good. The building is well maintained. Flagging and barriers at interior steel columns have been added to prevent damage from forklifts.

Building response in 1989 Loma Prieta Earthquake: The report titled "Performance of UCSF Buildings During the October 17, 1989 Loma Prieta Earthquake" dated 17 November 1989 by Impell Corporation states: "The building was inspected from the exterior by current standards, in the interior and at the roof. There are two departments housed in the same structure: Materials Management and Laundry area. A partition sheathed with gypsum board on one side of long and slender (approximately 30 ft. high) metal studs separates the two departments. Considerable damage was observed at this partition as many of the gypsum board panels had pulled off from the metal studs and fell on the side of the Laundry area of the building. Several of the metal studs buckled in a lateral torsional mode of failure. The partition does not carry any lateral or vertical load and the damage is primarily architectural. However, some of the remaining gypsum board panels constituted a falling hazard and occupants were warned to keep away from that area until it is repaired. Some vertical plaster cracks were observed in the office part of the building. There was no damage observed on the roof or the connections of the tilt-up exterior wall panels and no bulging or liquefaction of the soil occurred around the structure. Based on the inspection results, the structure was determined safe for occupancy." The referenced partitions are no longer in place.

Brief description of seismic deficiencies and expected seismic performance including mechanism of nonlinear response and structural behavior modes

Identified seismic deficiencies of the building include the following:

- Out-of-plane wall anchorage does not satisfy ASCE 41-17 requirements. In particular, a single out-of-plane wall anchor is provided at each purlin (spaced at 8' on center) and these anchors are overstressed by 100%.
- Continuous chord elements are provided at the top of the panel (rather than at the roof level) by a 7" double sided fillet weld between a #11 bar and angle. The location of the chord above the roof framing may result more damage at the roof level.
- A positive connection is not provided between the panel and the footings. The panels are tied in with short dowels at the slab on grade level and pilasters are positively attached to the footing.
- The CMU structure has expansion anchors between the roof framing and top of CMU wall. The size, embedment and spacing of the anchors is not called out on the drawings. Additionally, the horizontal reinforcement is not called out of the drawings. The connections between the roof framing and the wall may not be adequate to

transfer seismic forces from the wall into the diaphragm. Additionally, where anchorage at the top of the wall is not sufficient, seismic forces could transfer through horizontal spanning of the walls to perpendicular walls. However, since horizontal reinforcement was not specified on the original drawings, this load path could not be verified.

- The CMU structure connection to the slab on grade is through a post installed adhesive dowel. The lap length of the dowel with the vertical wall reinforcement is 9 inches, which is less than the required code minimum.

Structural deficiency	Affects rating?	Structural deficiency	Affects rating?
Lateral system stress check (wall shear, column shear or flexure, or brace axial as applicable)	N	Openings at shear walls (concrete or masonry)	N
Load path	N	Liquefaction	N
Adjacent buildings	N	Slope failure	N
Weak story	N	Surface fault rupture	N
Soft story	N	Masonry or concrete wall anchorage at flexible diaphragm	N
Geometry (vertical irregularities)	N	URM wall height-to-thickness ratio	N
Torsion	N	URM parapets or cornices	N
Mass – vertical irregularity	N	URM chimney	N
Cripple walls	N	Heavy partitions braced by ceilings	N
Wood sills (bolting)	N	Appendages	N
Diaphragm continuity	N		

Summary of review of nonstructural life-safety concerns, including at exit routes. [2]

A large number of tall prefabricated steel storage racks are located in the building. The racks are bolted to the concrete slab-on-grade, and have a nominal lateral load system sized to accommodate operational loads of placing and removing heavily loaded palates. Each stanchion typically has two anchor bolts attached to the concrete floor, with some bolts not installed or missing. At some locations it was clear that the stanchions had been struck by forklifts and the anchor bolts were no longer reliable. These racks could pose a potential life safety issue or block exits if damaged in an earthquake.

There are may freezers on site with high value contents. The freezers are anchored with a strap over the top and anchored each side to the slab-on-grade. This connection relies on friction. No testing is known to have been conducted to validate the seismic performance of this type of restraint.

UCOP non-structural checklist item	Life safety hazard?	UCOP non-structural checklist item	Life safety hazard?
Heavy ceilings, feature or ornamentation above large lecture halls, auditoriums, lobbies or other areas where large numbers of people congregate	None observed	Unrestrained hazardous materials storage	None observed
Heavy masonry or stone veneer above exit ways and public access areas	None observed	Masonry chimneys	None observed
Unbraced masonry parapets, cornices or other ornamentation above exit ways and public access areas	None observed	Unrestrained natural gas-fueled equipment such as water heaters, boilers, emergency generators, etc.	None observed

[2] For these Tier 1 evaluations, we do not visit all spaces of the building; we rely on campus staff to report to us their understanding of if and where nonstructural hazards may occur.

Basis of Seismic Performance Level rating

Retrofits in 1985 and 1998 addressed the most serious seismic deficiencies. However, out-of-plane wall anchorage at the roof purlin framing is inadequate for demands associated with ASCE 41-17. Additionally, the CMU structure may have inadequate anchorage from the roof framing to the top of wall, and/or insufficient horizontal reinforcement to span to perpendicular walls under seismic loading.

Recommendations for further evaluation or retrofit

Additional verification of the CMU structure framing and wall reinforcement is recommended.

Peer review comments on rating

The structural members of the UCSF Seismic Review Committee (SRC) reviewed the evaluation on 2 June 2020 and were unanimous that the Seismic Performance Level Rating is Level V.

Additional building data	Entry	Notes
Latitude	37.65837	
Longitude	-122.385	
Are there other structures besides this one under the same CAAN#	No	
Number of stories above lowest perimeter grade	1	There are two mezzanines areas with 2 levels, however the majority of the building is 1 level.
Number of stories (basements) below lowest perimeter grade	0	Building pad is flat
Building occupiable area (OGSF)	144,429	Calculated
Risk Category per 2016 CBC 1604.5	II	
Building structural height, h_n	24.5 ft	Structural height defined per ASCE 7-16 Section 11.2
Coefficient for period, C_t	0.02	Estimated using ASCE 41-17 equation 4-4 and 7-18
Coefficient for period, β	0.75	Estimated using ASCE 41-17 equation 4-4 and 7-18
Estimated fundamental period	0.22 sec	Estimated using ASCE 41-17 equation 4-4 and 7-18
Site data		
975-year hazard parameters S_s, S_1	1.640g, 0.657g	UCSF Group 2 Buildings – Tier 1 Geotechnical Assessment, Egan (2019)
Site class	E	
Site class basis		UCSF Group 2 Buildings – Tier 1 Geotechnical Assessment, Egan (2019)
Site parameters F_a, F_v	1.3, 4.2	Applied Technology Council website
Ground motion parameters S_{cs}, S_{c1}	2.13g, 2.758g	UCSF Group 2 Buildings – Tier 1 Geotechnical Assessment, Egan (2019)
S_a at building period	2.13g	W = 897 kips, V base = 1,697 kips

Site V_{s30}	210 m/s	
V_{s30} basis	Estimated	UCSF Group 2 Buildings – Tier 1 Geotechnical Assessment, Egan (2019)
Liquefaction potential/basis	No	UCSF Group 2 Buildings – Tier 1 Geotechnical Assessment, Egan (2019)
Landslide potential/basis	No	UCSF Group 2 Buildings – Tier 1 Geotechnical Assessment, Egan (2019)
Active fault-rupture hazard identified at site?	No	UCSF Group 2 Buildings – Tier 1 Geotechnical Assessment, Egan (2019)
Site-specific ground motion study?	No	
Applicable code		
Applicable code or approx. date of original construction	Built: 1973 Code: 1970 UBC	Code assumed
Applicable code for partial retrofit	N/A	1985 Partial Voluntary Retrofit
Applicable code for partial retrofit	1995 CBC	1998 Partial Voluntary Retrofit
Applicable code for full retrofit	None	No full retrofit known
Model building data		
Model building type North-South	PC1	Pre-cast or concrete tilt up shear walls (with flexible diaphragm)
Model building type East-West	PC1	Pre-cast or concrete tilt up shear walls (with flexible diaphragm)
FEMA P-154 score	N/A	Not included here because we performed ASCE 41 Tier 1 evaluation.
Previous ratings		
Most recent rating	IV	
Date of most recent rating	2013	2013 "UCSF Building Seismic Survey and Ratings"
2nd most recent rating	-	
Date of 2nd most recent rating	-	
3rd most recent rating	-	
Date of 3rd most recent rating	-	
Appendices		
ASCE 41 Tier 1 checklist included here?	Yes	Refer to attached checklist file

Appendix A

Additional Images

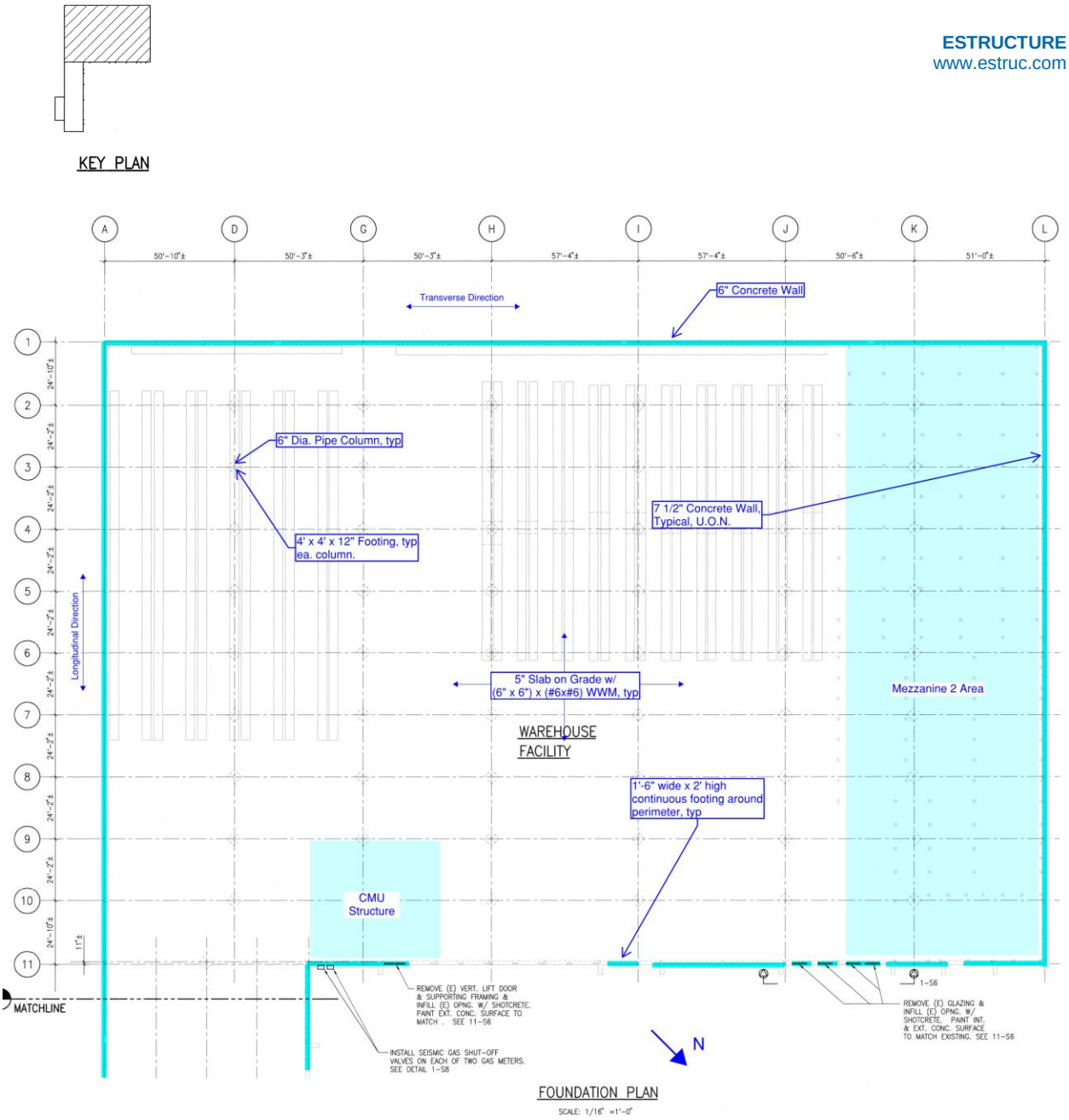


Figure 1 - Foundation Plan (1 of 2)

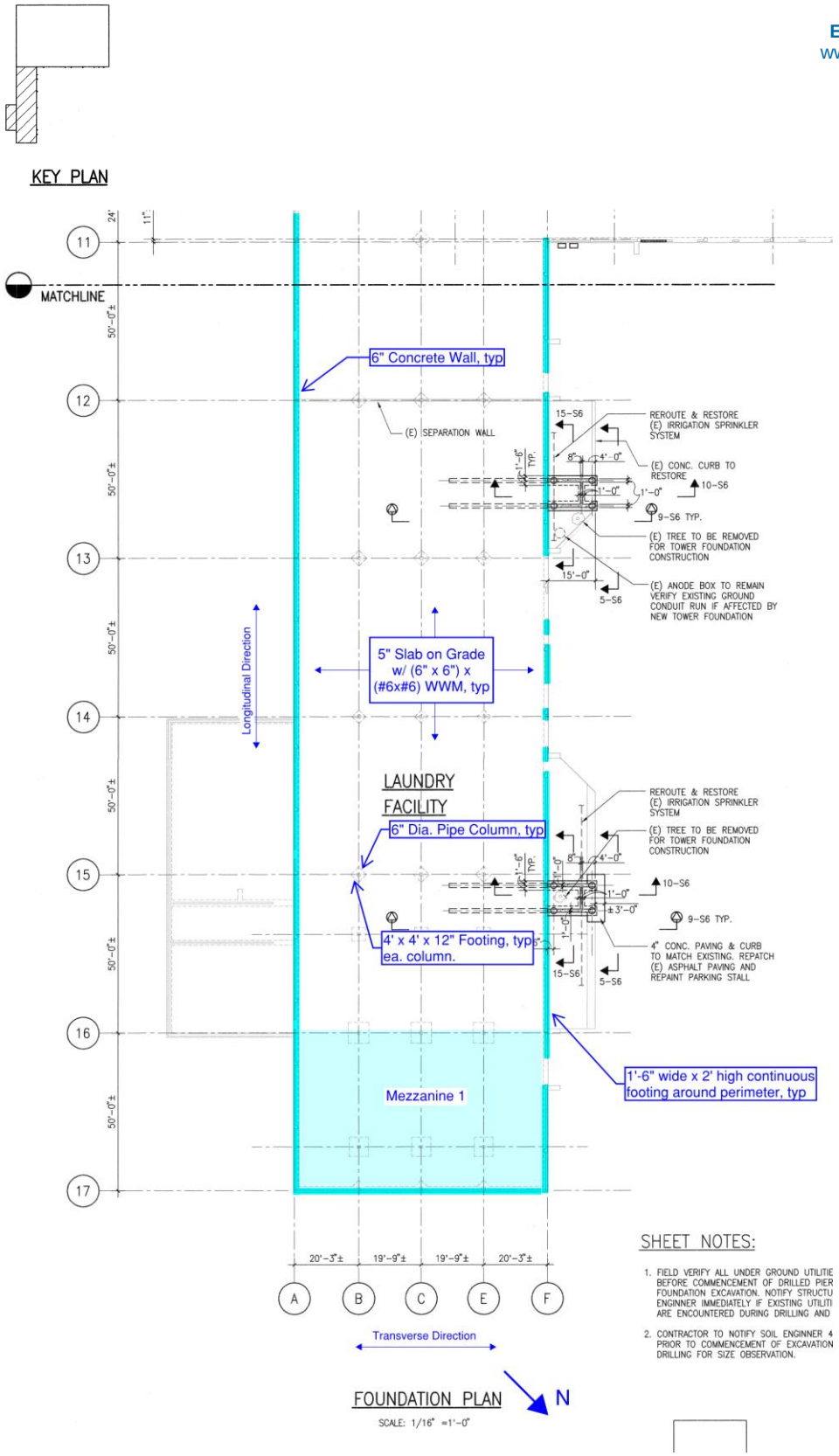


Figure 2 - Foundation Plan (2 of 2)

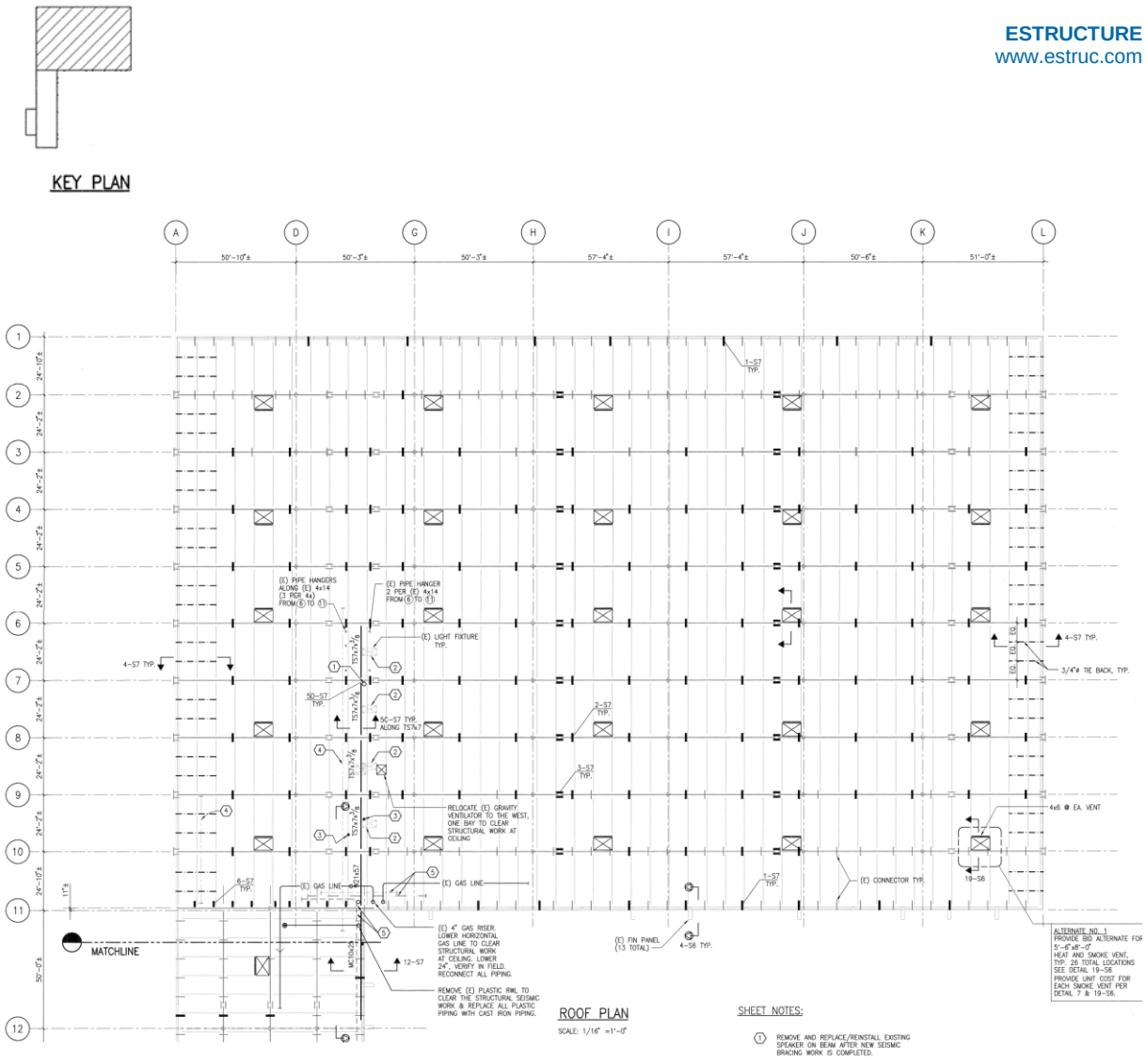
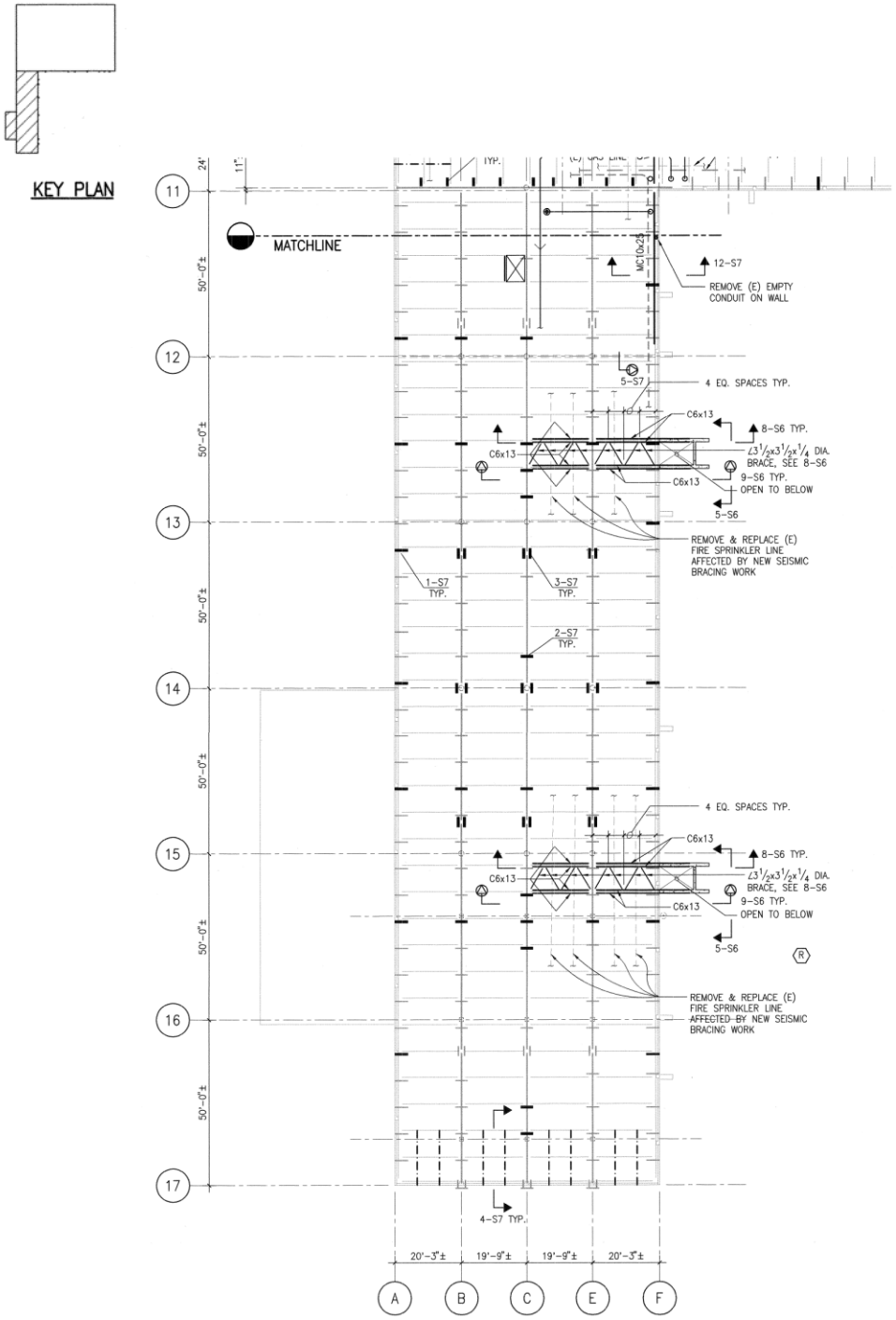


Figure 3 - Roof Plan (1 of 2) 1997 Seismic Retrofit



ROOF PLAN
SCALE: 1/16" =1'-0"

Figure 4 - Roof Plan (2 of 2) 1997 Seismic Retrofit

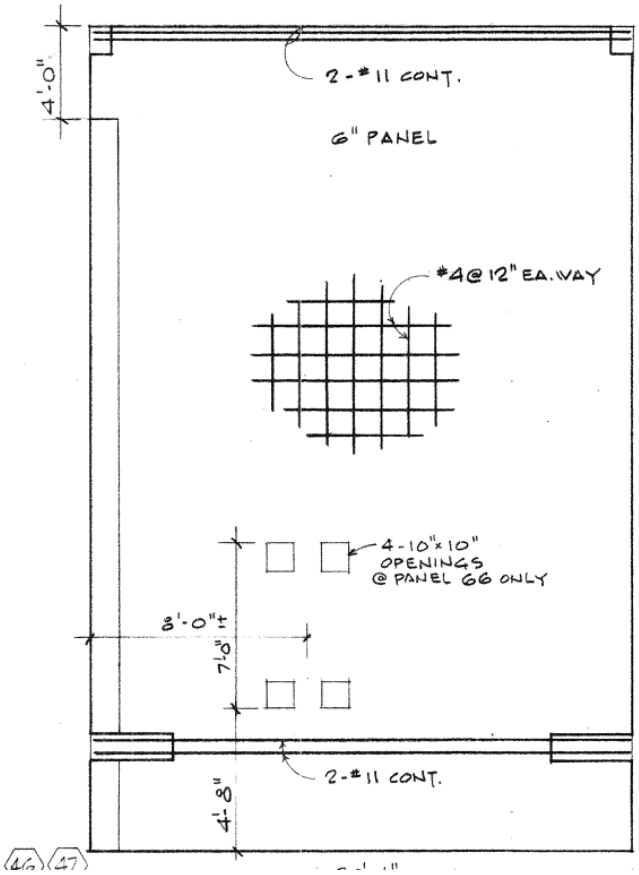


Figure 5 - Typical Panel Elevation

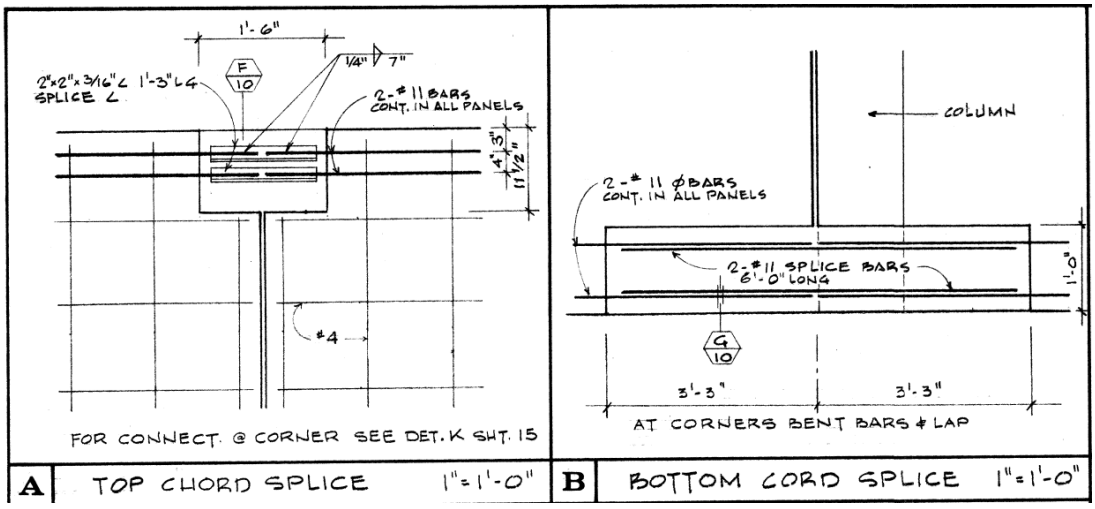


Figure 6 - Chord Splice Details

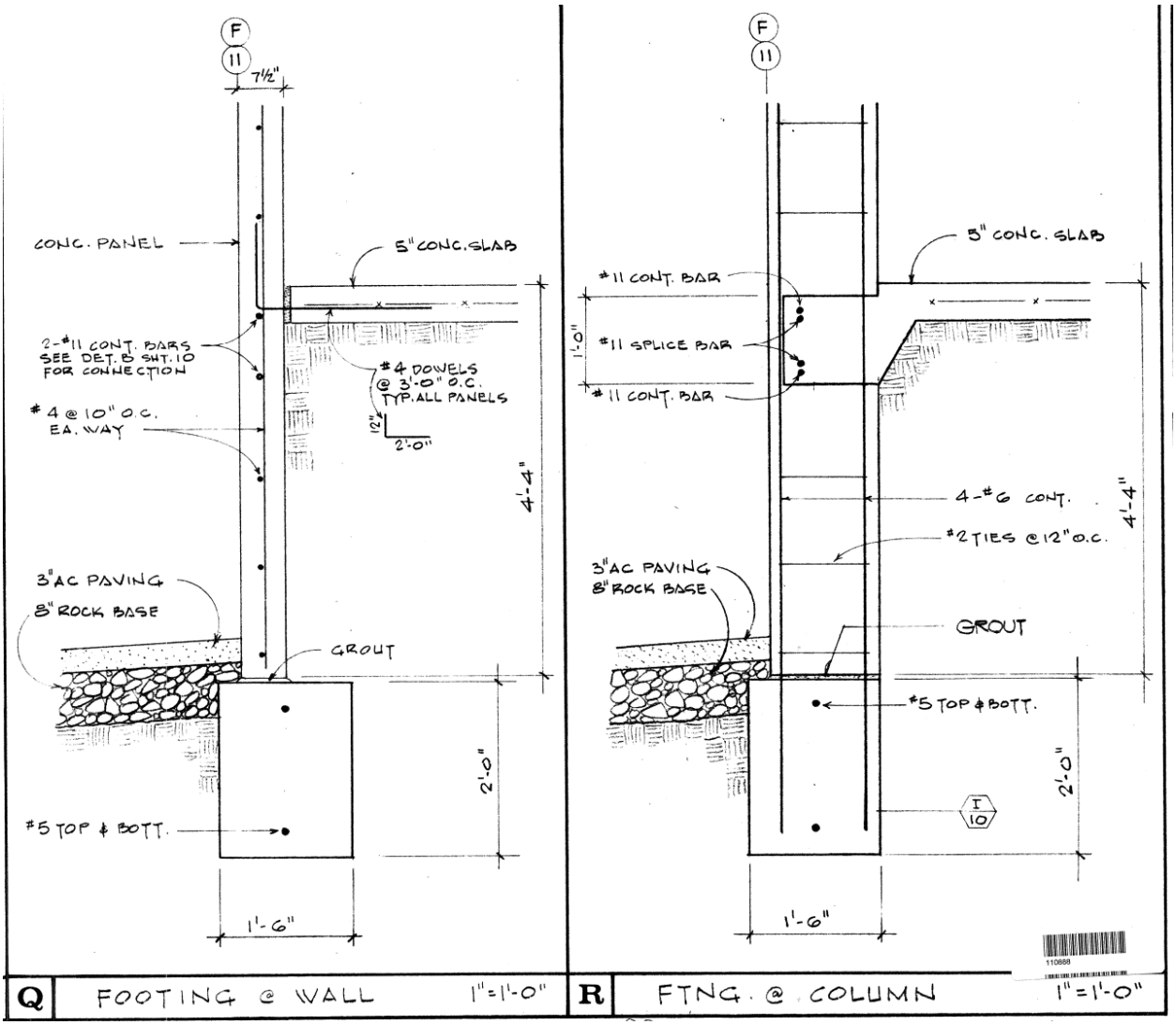


Figure 7 - Foundation / Wall Connection Details

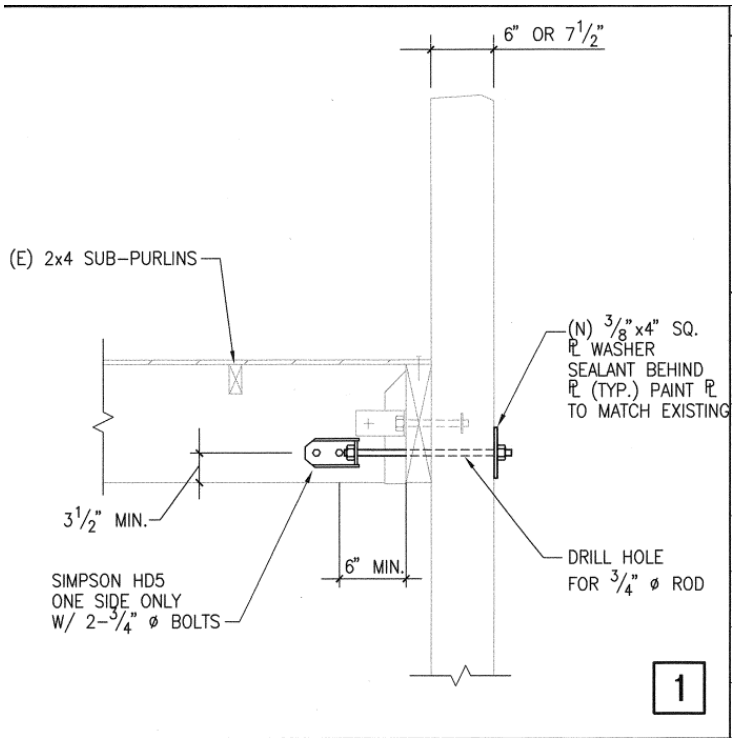


Figure 8 – Out-of-Plane Wall Anchorage at Purlin (1985 and 1998 Seismic Retrofit)

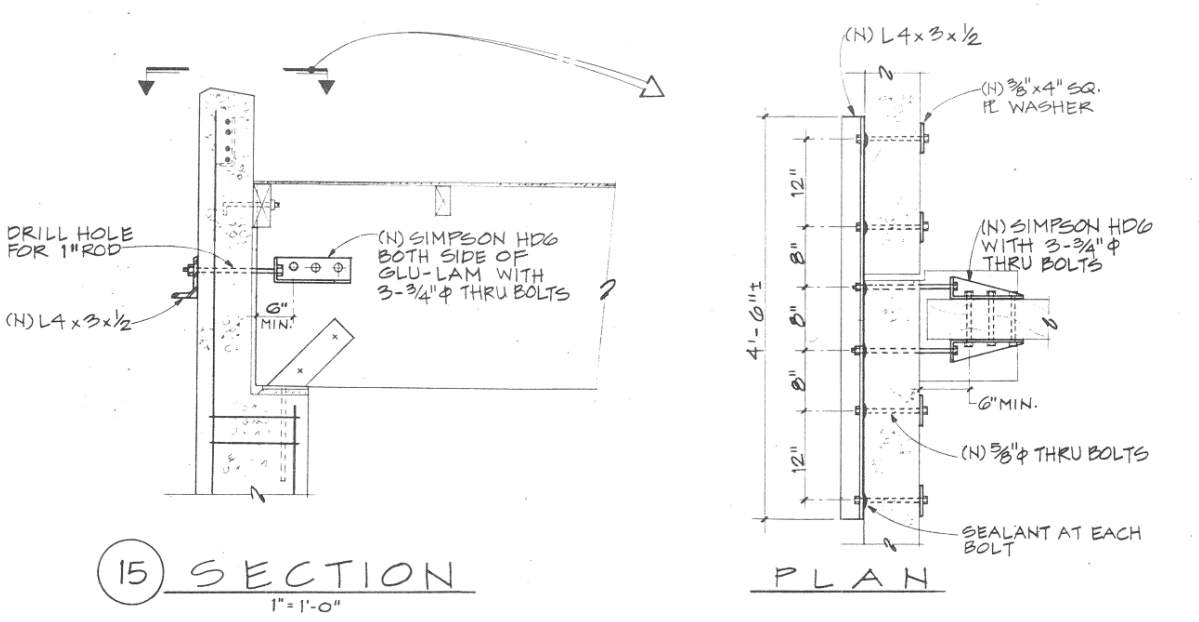


Figure 9 - Out of Plane Wall Anchorage at Glulam (1985 Seismic Retrofit)

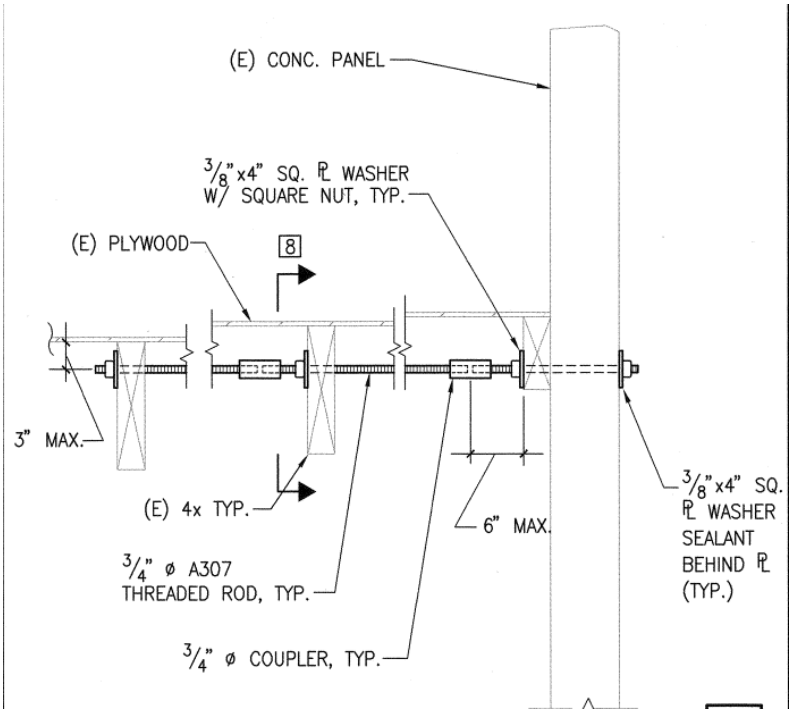


Figure 10 - Out of Plane Wall Anchor (2 added between Glulams, 1998 Retrofit)

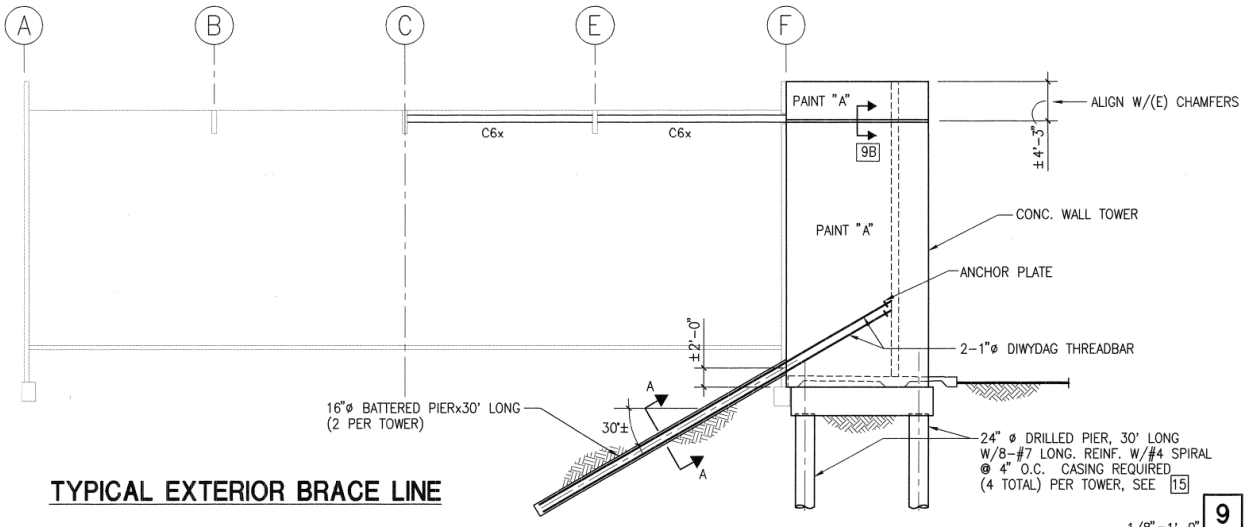


Figure 11 - Section at Buttress (1998 Retrofit)

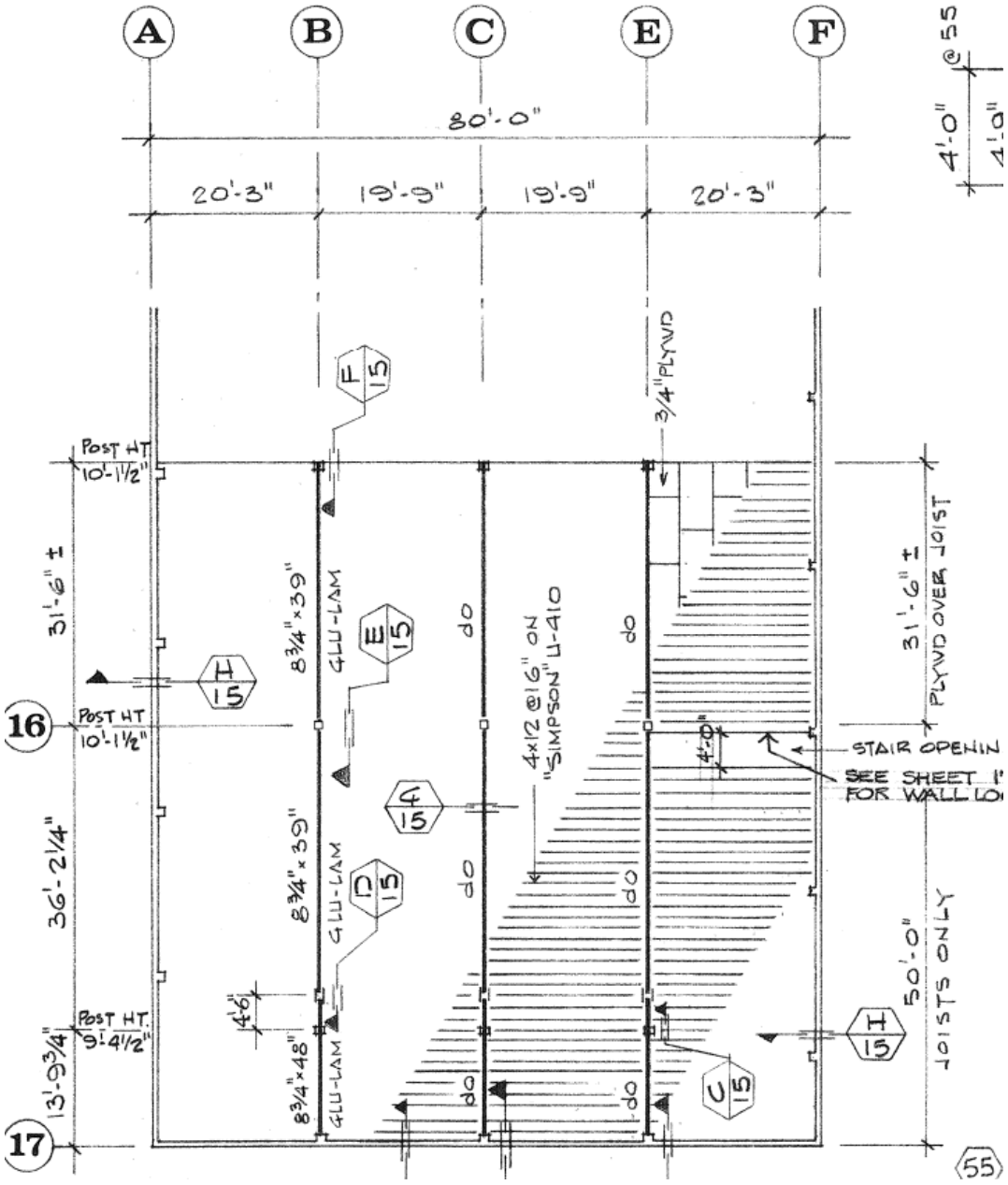


Figure 12: Mezzanine 1 Plan

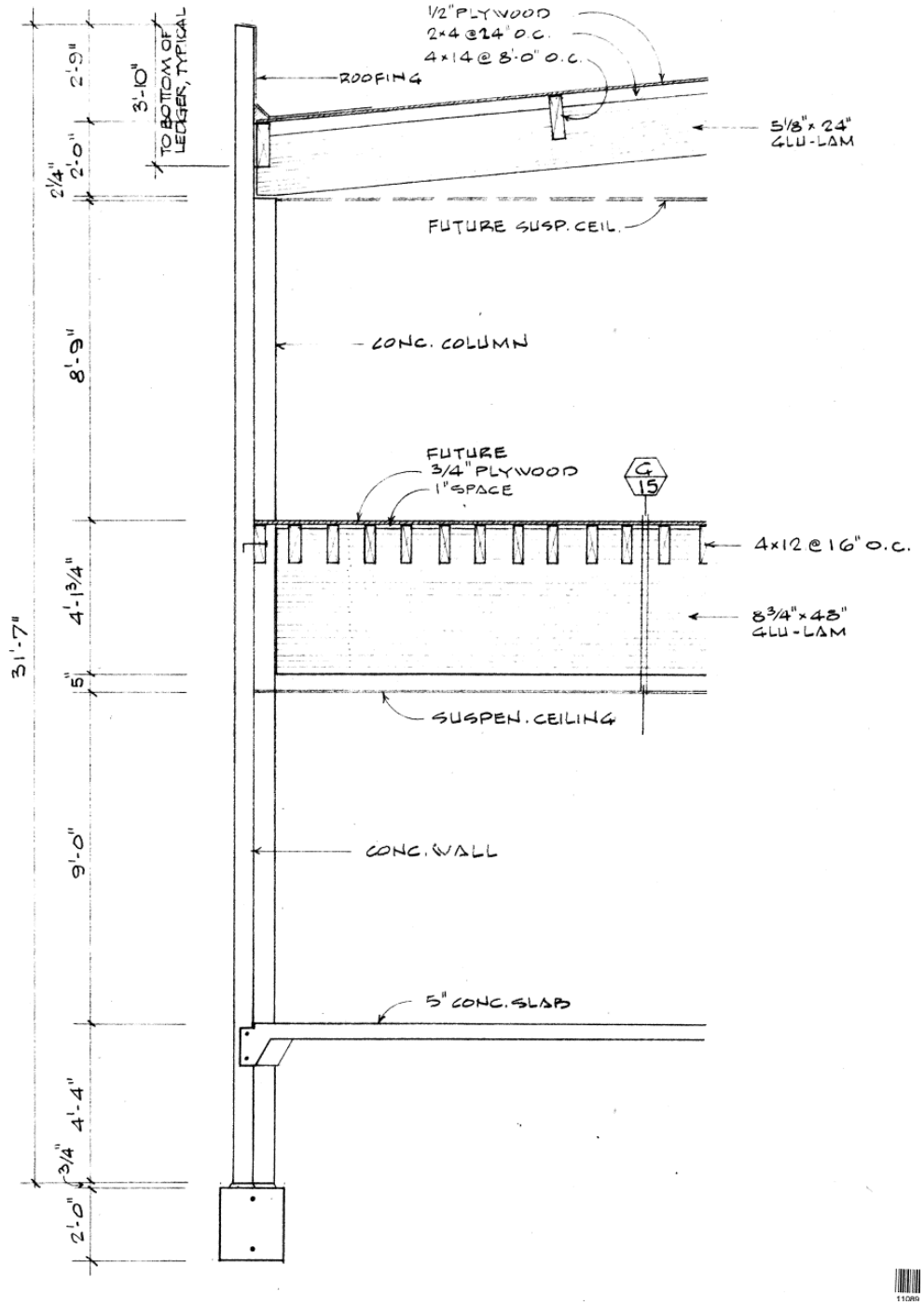
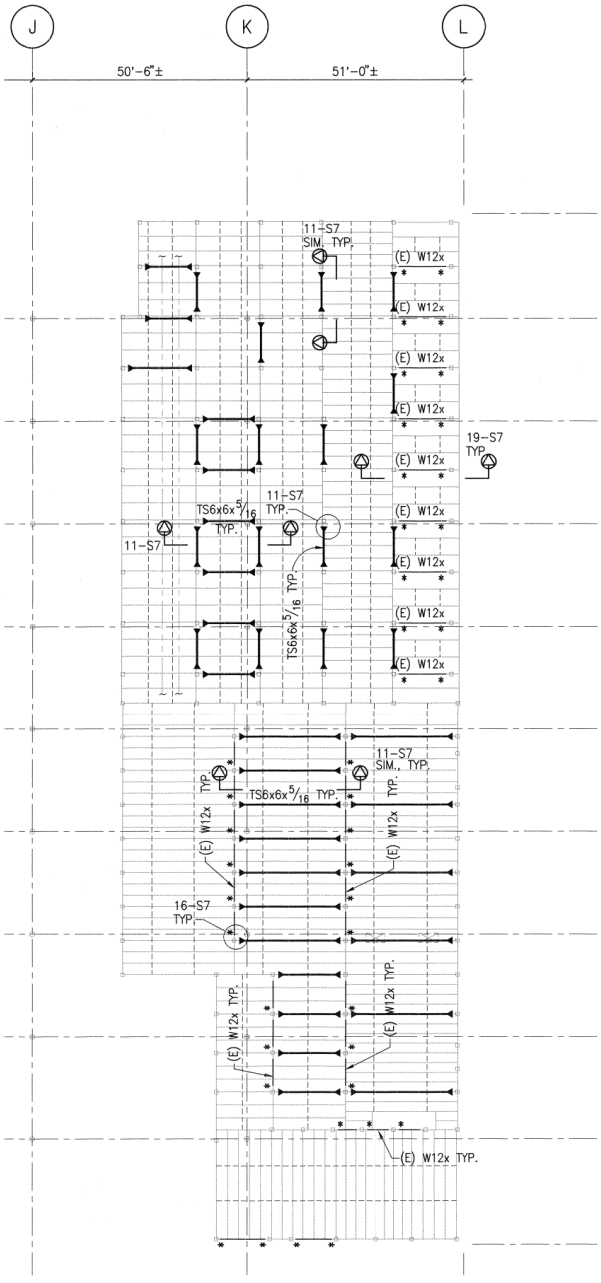


Figure 13 - Section at Mezzanine 1



MEZZANINE PLAN

SCALE: 1/16" =1'-0"

Figure 14 - Mezzanine 2 Plan

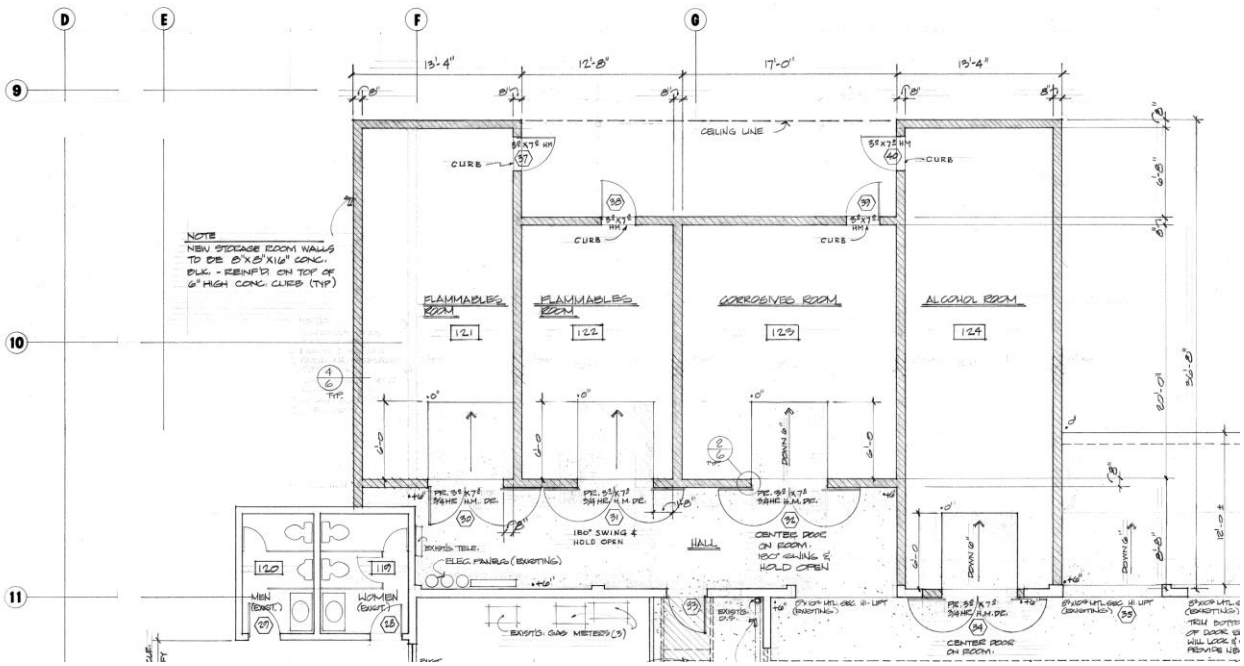


Figure 15 - CMU Structure Plan

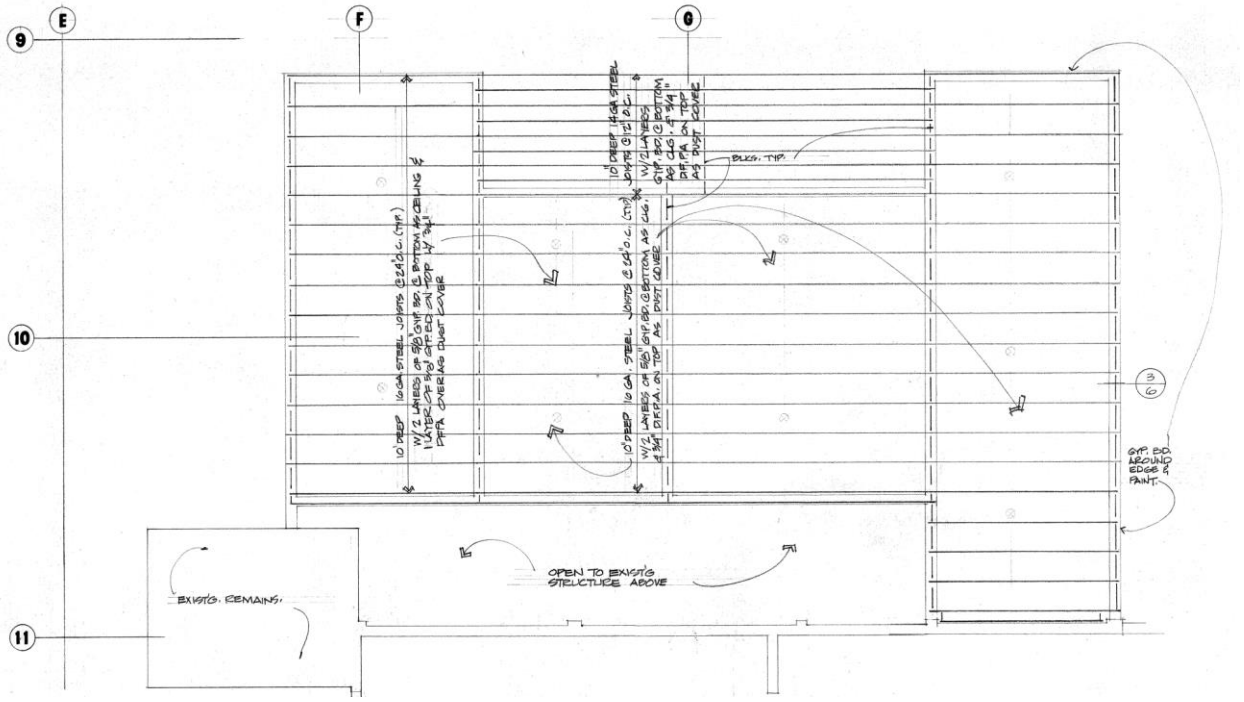


Figure 16 - CMU Structure Roof Plan

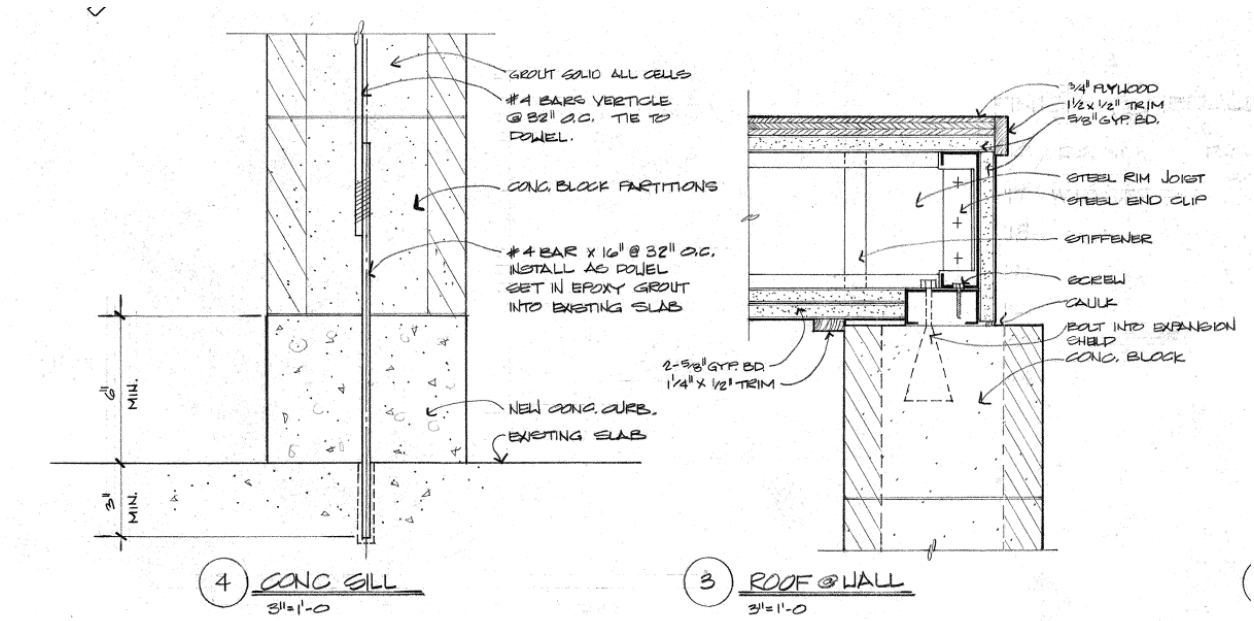


Figure 17 - CMU Structure Details

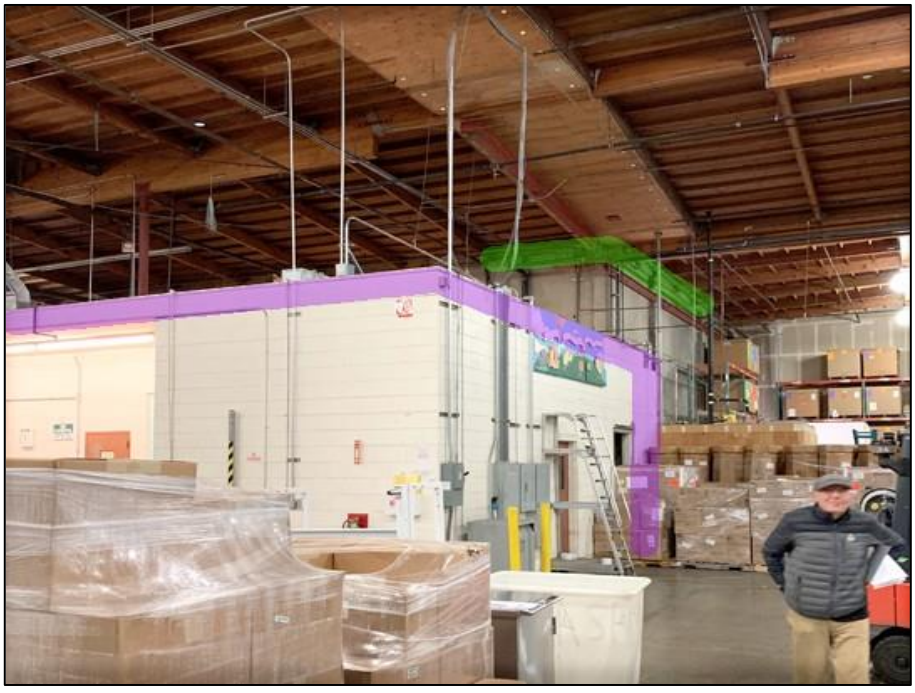


Figure 18 - CMU Structure

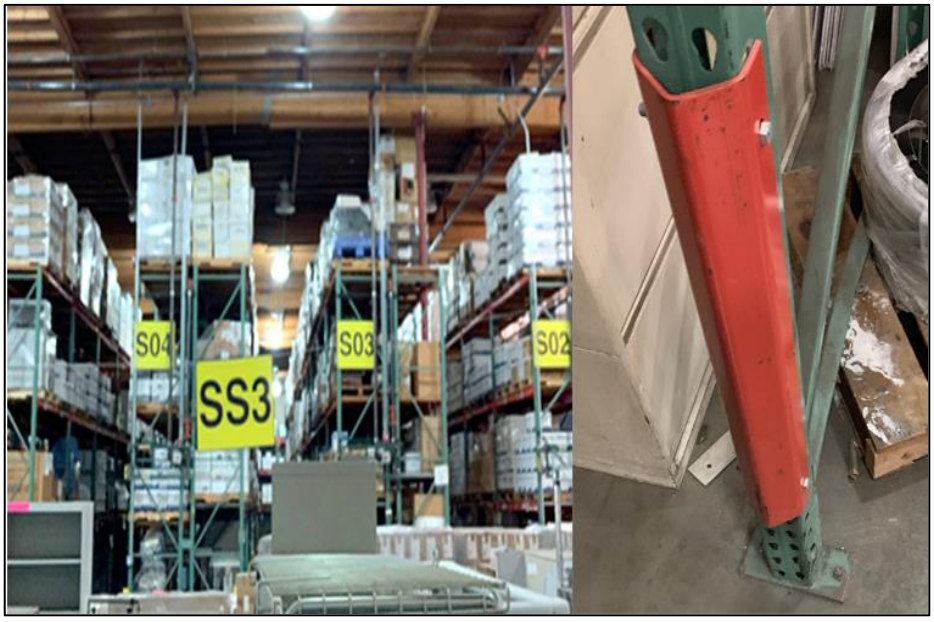


Figure 19 - Storage Rack

Figure 20 - Freezer Anchorage

Appendix B

ASCE 41- 17 Tier 1 Checklists (Structural)

UC Campus:	San Francisco			Date:	June 19, 2020		
Building CAAN:	2418	Auxiliary CAAN:		By Firm:	Estructure		
Building Name:	Oyster Point			Initials:	*ARK*	Checked:	*MTP*
Building Address:	612 Forbes Boulevard, South San Francisco			Page:	1	of	3

ASCE 41-17
Collapse Prevention Basic Configuration Checklist

LOW SEISMICITY

BUILDING SYSTEMS - *GENERAL*

C	NC	N/A	U	Description
◉	○	○	○	LOAD PATH: The structure contains a complete, well-defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation. (Commentary: Sec. A.2.1.1. Tier 2: Sec. 5.4.1.1) **Comments:**
○	○	◉	○	ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building is greater than 0.25% of the height of the shorter building in low seismicity, 0.5% in moderate seismicity, and 1.5% in high seismicity. (Commentary: Sec. A.2.1.2. Tier 2: Sec. 5.4.1.2) **Comments:**
◉	○	○	○	MEZZANINES: Interior mezzanine levels are braced independently from the main structure or are anchored to the seismic-force-resisting elements of the main structure. (Commentary: Sec. A.2.1.3. Tier 2: Sec. 5.4.1.3) **Comments:** Mezzanine 1 is anchored to the exterior CMU walls. Mezzanine 2 and the CMU structure are independent from the main structure. An independent checklist is provided for the CMU structure.

BUILDING SYSTEMS - *BUILDING CONFIGURATION*

C	NC	N/A	U	Description
○	○	◉	○	WEAK STORY: The sum of the shear strengths of the seismic-force-resisting system in any story in each direction is not less than 80% of the strength in the adjacent story above. (Commentary: Sec. A2.2.2. Tier 2: Sec. 5.4.2.1) **Comments:**
○	○	◉	○	SOFT STORY: The stiffness of the seismic-force-resisting system in any story is not less than 70% of the seismic-force-resisting system stiffness in an adjacent story above or less than 80% of the average seismic-force-resisting system stiffness of the three stories above. (Commentary: Sec. A.2.2.3. Tier 2: Sec. 5.4.2.2) **Comments:**

Note: **C =** Compliant **NC =** Noncompliant **N/A =** Not Applicable **U =** Unknown

ASCE 41-17
Collapse Prevention Basic Configuration Checklist

C	NC	N/A	U	
◉	○	○	○	VERTICAL IRREGULARITIES: All vertical elements in the seismic-force-resisting system are continuous to the foundation. (Commentary: Sec. A.2.2.4. Tier 2: Sec. 5.4.2.3) **Comments:**
○	○	◉	○	GEOMETRY: There are no changes in the net horizontal dimension of the seismic-force-resisting system of more than 30% in a story relative to adjacent stories, excluding one-story penthouses and mezzanines. (Commentary: Sec. A.2.2.5. Tier 2: Sec. 5.4.2.4) **Comments:**
○	○	◉	○	MASS: There is no change in effective mass of more than 50% from one story to the next. Light roofs, penthouses, and mezzanines need not be considered. (Commentary: Sec. A.2.2.6. Tier 2: Sec. 5.4.2.5) **Comments:**
◉	○	○	○	TORSION: The estimated distance between the story center of mass and the story center of rigidity is less than 20% of the building width in either plan dimension. (Commentary: Sec. A.2.2.7. Tier 2: Sec. 5.4.2.6) **Comments:**

MODERATE SEISMICITY (COMPLETE THE FOLLOWING ITEMS IN ADDITION TO THE ITEMS FOR LOW SEISMICITY)

GEOLOGIC SITE HAZARD

C	NC	N/A	U	Description
◉	○	○	○	LIQUEFACTION: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance do not exist in the foundation soils at depths within 50 ft (15.2m) under the building. (Commentary: Sec. A.6.1.1. Tier 2: 5.4.3.1) **Comments:** From UCSF Group 2 Buildings – Tier 1 Geotechnical Assessment, Egan (2019): Mapped liquefaction susceptibility is "Very High. Given, however, the nature of the surficial fill (dense to very dense clayey broken rock and very stiff sandy clay), liquefaction-related hazard at the site and to the building is probably low."
◉	○	○	○	SLOPE FAILURE: The building site is located away from potential earthquake-induced slope failures or rockfalls so that it is unaffected by such failures or is capable of accommodating any predicted movements without failure. (Commentary: Sec. A.6.1.2. Tier 2: 5.4.3.1) **Comments:**

Note: **C =** Compliant **NC =** Noncompliant **N/A =** Not Applicable **U =** Unknown

UC Campus:	San Francisco			Date:	June 19, 2020		
Building CAAN:	2418	Auxiliary CAAN:		By Firm:	Estructure		
Building Name:	Oyster Point			Initials:	*ARK*	Checked:	*MTP*
Building Address:	612 Forbes Boulevard, South San Francisco			Page:	3	of	3

ASCE 41-17
Collapse Prevention Basic Configuration Checklist

MODERATE SEISMICITY (COMPLETE THE FOLLOWING ITEMS IN addition TO THE ITEMS FOR LOW SEISMICITY)

GEOLOGIC SITE HAZARD

C	NC	N/A	U	
◉	○	○	○	SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site are not anticipated. (Commentary: Sec. A.6.1.3. Tier 2: 5.4.3.1) **Comments:**

HIGH SEISMICITY (COMPLETE THE FOLLOWING ITEMS IN ADDITION TO THE ITEMS FOR MODERATE SEISMICITY)

FOUNDATION CONFIGURATION

				Description
C	NC	N/A	U	
◉	○	○	○	OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$. (Commentary: Sec. A.6.2.1. Tier 2: Sec. 5.4.3.3) **Comments:** 0.6 Sa = 1.278 < Base / Height = 80 ft / 24.5 ft = 3.27
C	NC	N/A	U	
◉	○	○	○	TIES BETWEEN FOUNDATION ELEMENTS: The foundation has ties adequate to resist seismic forces where footings, piles, and piers are not restrained by beams, slabs, or soils classified as Site Class A, B, or C. (Commentary: Sec. A.6.2.2. Tier 2: Sec. 5.4.3.4) **Comments:** Continuous grade beam under exterior walls part of the seismic force resisting system.

Note: **C =** Compliant **NC =** Noncompliant **N/A =** Not Applicable **U =** Unknown

UC Campus:	San Francisco			Date:	5/22/2020		
Building CAAN:	2418	Auxiliary CAAN:		By Firm:	Estructure		
Building Name:	Oyster Point			Initials:	*ARK*	Checked:	*MTP*
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ASCE 41-17
Collapse Prevention Structural Checklist For Building Type PC1-PC1A

LOW SEISMICITY

CONNECTIONS

	Description
C ○ NC ◉ N/A ○ U ○	WALL ANCHORAGE: Exterior concrete or masonry walls that are dependent on the diaphragm for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections have strength to resist the connection force calculated in the Quick Check procedure of Section 4.4.3.7. (Commentary: Sec. A.5.1.1. Tier 2: Sec. 5.7.1.1) **Comments:** See attached calculations, out-of-plane wall anchors at glulam beams are adequate. Wall anchorage at purlins is not adequate.

MODERATE SEISMICITY (COMPLETE THE FOLLOWING ITEMS IN ADDITION TO THE ITEMS FOR LOW SEISMICITY)

SEISMIC-FORCE-RESISTING SYSTEM

	Description
C ◉ NC ○ N/A ○ U ○	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2. (Commentary: Sec. A.3.2.1.1. Tier 2: Sec. 5.5.1.1) **Comments:**
C ◉ NC ○ N/A ○ U ○	WALL SHEAR STRESS CHECK: The shear stress in the precast panels, calculated using the Quick Check procedure of Section 4.4.3.3, is less than the greater of 100 lb/in.^{2} (0.69 MPa) or $2\sqrt{f'_c}$ (Commentary: Sec. A.3.2.3.1. Tier 2: Sec. 5.5.3.1.1) **Comments:**
C ◉ NC ○ N/A ○ U ○	REINFORCING STEEL: The ratio of reinforcing steel area to gross concrete area is not less than 0.0012 in the vertical direction and 0.0020 in the horizontal direction. (Commentary: Sec. A.3.2.3.2. Tier 2: Sec. 5.5.3.1.3) **Comments:**
C ◉ NC ○ N/A ○ U ○	WALL THICKNESS: Thicknesses of bearing walls are not less than 1/40 the unsupported height or length, whichever is shorter, nor less than 4 in. (101 mm) (Commentary: Sec. A.3.2.3.5. Tier 2: Sec. 5.5.3.1.2) **Comments:**

ASCE 41-17
Collapse Prevention Structural Checklist For Building Type PC1-PC1A

DIAPHRAGMS

C	NC	N/A	U	Description
○	○	◉	○	TOPPING SLAB: Precast concrete diaphragm elements are interconnected by a continuous reinforced concrete topping slab with a minimum thickness of 2 in. (51 mm) (Commentary: Sec. A.4.5.1. Tier 2: Sec. 5.6.4) **Comments:**

CONNECTIONS

C	NC	N/A	U	Description
◉	○	○	○	WOOD LEDGERS: The connection between the wall panels and the diaphragm does not induce cross-grain bending or tension in the wood ledgers. (Commentary: Sec. A.5.1.2. Tier 2: Sec. 5.7.1.3) **Comments:**
◉	○	○	○	TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls. (Commentary: Sec. A.5.2.1. Tier 2: Sec. 5.7.2) **Comments:**
○	○	◉	○	TOPPING SLAB TO WALLS OR FRAMES: Reinforced concrete topping slabs that interconnect the precast concrete diaphragm elements are doweled for transfer of forces into the shear wall or frame elements. (Commentary: Sec. A.5.2.3. Tier 2: Sec. 5.7.2) **Comments:**
◉	○	○	○	GIRDER–COLUMN CONNECTION: There is a positive connection using plates, connection hardware, or straps between the girder and the column support. (Commentary: Sec. A.5.4.1. Tier 2: Sec. 5.7.4.1) **Comments:**

ASCE 41-17
Collapse Prevention Structural Checklist For Building Type PC1-PC1A

HIGH SEISMICITY (COMPLETE THE FOLLOWING ITEMS IN addition TO THE ITEMS FOR LOW AND MODERATE SEISMICITY)

SEISMIC-FORCE-RESISTING SYSTEM

	Description
C NC N/A U ○ ○ ◉ ○	DEFLECTION COMPATIBILITY FOR RIGID DIAPHRAGMS: Secondary components have the shear capacity to develop the flexural strength of the components. (Commentary: Sec. A.3.1.6.2. Tier 2: Sec. 5.5.2.5.2) **Comments:**
C NC N/A U ◉ ○ ○ ○	WALL OPENINGS: The total width of openings along any perimeter wall line constitutes less than 75% of the length of any perimeter wall when the wall piers have aspect ratios of less than 2-to-1. (Commentary: Sec. A.3.2.3.3. Tier 2: Sec. 5.5.3.3.1) **Comments:**

DIAPHRAGMS

	Description
C NC N/A U ◉ ○ ○ ○	CROSS TIES IN FLEXIBLE DIAPHRAGMS: There are continuous cross ties between diaphragm chords. (Commentary: Sec. A.4.1.2. Tier 2: Sec. 5.6.1.2) **Comments:**
C NC N/A U ○ ○ ◉ ○	STRAIGHT SHEATHING: All straight-sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered. (Commentary: Sec. A.4.2.1. Tier 2: Sec. 5.6.2) **Comments:**
C NC N/A U ◉ ○ ○ ○	SPANS: All wood diaphragms with spans greater than 24 ft (7.3 m) consist of wood structural panels or diagonal sheathing. (Commentary: Sec. A.4.2.2. Tier 2: Sec. 5.6.2) **Comments:**
C NC N/A U ○ ○ ◉ ○	DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 ft (12.2 m) and aspect ratios less than or equal to 4-to-1. (Commentary: Sec. A.4.2.3. Tier 2: Sec. 5.6.2) **Comments:**

ASCE 41-17
Collapse Prevention Structural Checklist For Building Type PC1-PC1A

C	NC	N/A	U	
○	○	◉	○	OTHER DIAPHRAGMS: Diaphragms do not consist of a system other than wood, metal deck, concrete, or horizontal bracing. (Commentary: Sec. A.4.7.1. Tier 2: Sec. 5.6.5) **Comments:**

CONNECTIONS

C	NC	N/A	U	Description
◉	○	○	○	MINIMUM NUMBER OF WALL ANCHORS PER PANEL: There are at least two anchors connecting each precast wall panel to the diaphragm elements. (Commentary: Sec. A.5.1.3. Tier 2: Sec. 5.7.1.4) **Comments:**
○	◉	○	○	PRECAST WALL PANELS: Precast wall panels are connected to the foundation. (Commentary: Sec. A.5.3.6. Tier 2: Sec. 5.7.3.4) **Comments:** Precast wall panels are connected to the slab on grade, pilasters are connected to the footings.
○	○	◉	○	UPLIFT AT PILE CAPS: Pile caps have top reinforcement, and piles are anchored to the pile caps. (Commentary: Sec. A.5.3.8. Tier 2: Sec. 5.7.3.5) **Comments:**
◉	○	○	○	GIRDERS: Girders supported by walls or pilasters have at least two ties securing the anchor bolts unless provided with independent stiff wall anchors with strength to resist the connection force calculated in the Quick Check procedure of Section 4.4.3.7. (Commentary: Sec. A.5.4.2. Tier 2: Sec. 5.7.4.2) **Comments:** See attached calculations. Wall anchorage at purlins is acceptable.

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ASCE 41-17
Collapse Prevention Structural Checklist For Building Type RM1-RM2

LOW AND MODERATE SEISMICITY

SEISMIC-FORCE-RESISTING SYSTEM

	Description
C ● NC ○ N/A ○ U ○	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2. (Commentary: Sec. A.3.2.1.1. Tier 2: Sec. 5.5.1.1) **Comments:**
C ● NC ○ N/A ○ U ○	SHEAR STRESS CHECK: The shear stress in the reinforced masonry shear walls, calculated using the Quick Check procedure of Section 4.4.3.3, is less than 70 lb/in.^{2} (0.48 MPa). (Commentary: Sec. A.3.2.4.1. Tier 2: Sec. 5.5.3.1.1) **Comments:** By Inspection the walls are adequate for in plan shear based on the light weight of the roof and amount of wall.
C ○ NC ○ N/A ○ U ●	REINFORCING STEEL: The total vertical and horizontal reinforcing steel ratio in reinforced masonry walls is greater than 0.002 of the wall with the minimum of 0.0007 in either of the two directions; the spacing of reinforcing steel is less than 48 in. (1220 mm), and all vertical bars extend to the top of the walls. (Commentary: Sec. A.3.2.4.2. Tier 2: Sec. 5.5.3.1.3) **Comments:** Existing drawings do not include horizontal reinforcement information.

STIFF DIAPHRAGMS

	Description
C ○ NC ○ N/A ● U ○	TOPPING SLAB: Precast concrete diaphragm elements are interconnected by a continuous reinforced concrete topping slab. (Commentary: Sec. A.4.5.1. Tier 2: Sec. 5.6.4) **Comments:**

CONNECTIONS

	Description
C ○ NC ○ N/A ○ U ●	WALL ANCHORAGE: Exterior concrete or masonry walls that are dependent on the diaphragm for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections have strength to resist the connection force calculated in the Quick Check procedure of Section 4.4.3.7. (Commentary: Sec. A.5.1.1. Tier 2: Sec. 5.7.1.1) **Comments:** Size and spacing of out-of-plane wall anchors is not indicated on the existing drawings.

ASCE 41-17
Collapse Prevention Structural Checklist For Building Type RM1-RM2

C	NC	N/A	U	
○	○	◉	○	WOOD LEDGERS: The connection between the wall panels and the diaphragm does not induce cross-grain bending or tension in the wood ledgers. (Commentary: Sec. A.5.1.2. Tier 2: Sec. 5.7.1.3) **Comments:** There are no wood ledgers.
◉	○	○	○	TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls. (Commentary: Sec. A.5.2.1. Tier 2: Sec. 5.7.2) **Comments:**
○	○	◉	○	TOPPING SLAB TO WALLS OR FRAMES: Reinforced concrete topping slabs that interconnect the precast concrete diaphragm elements are doweled for transfer of forces into the shear wall or frame elements. (Commentary: Sec. A.5.2.3. Tier 2: Sec. 5.7.2) **Comments:**
◉	○	○	○	FOUNDATION DOWELS: Wall reinforcement is doweled into the foundation. (Commentary: Sec. A.5.3.5. Tier 2: Sec. 5.7.3.4) **Comments:** Positive connection is provided between walls to the slab on grade. The lap splice from the foundation dowel to the wall is only 9”.
○	○	◉	○	GIRDER–COLUMN CONNECTION: There is a positive connection using plates, connection hardware, or straps between the girder and the column support. (Commentary: Sec. A.5.4.1. Tier 2: Sec. 5.7.4.1) **Comments:**

HIGH SEISMICITY (COMPLETE THE FOLLOWING ITEMS IN ADDITION TO THE ITEMS FOR LOW AND MODERATE SEISMICITY)

STIFF DIAPHRAGMS

C	NC	N/A	U	Description
○	○	◉	○	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 25% of the wall length. (Commentary: Sec. A.4.1.4. Tier 2: Sec. 5.6.1.3) **Comments:**
○	○	◉	○	OPENINGS AT EXTERIOR MASONRY SHEAR WALLS: Diaphragm openings immediately adjacent to exterior masonry shear walls are not greater than 8 ft (2.4 m) long. (Commentary: Sec. A.4.1.6. Tier 2: Sec. 5.6.1.3) **Comments:**

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ASCE 41-17
Collapse Prevention Structural Checklist For Building Type RM1-RM2

FLEXIBLE DIAPHRAGMS

C	NC	N/A	U	Description
◉	○	○	○	CROSS TIES: There are continuous cross ties between diaphragm chords. (Commentary: Sec. A.4.1.2. Tier 2: Sec. 5.6.1.2) **Comments:**
○	○	◉	○	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 25% of the wall length. (Commentary: Sec. A.4.1.4. Tier 2: Sec. 5.6.1.3) **Comments:**
○	○	◉	○	OPENINGS AT EXTERIOR MASONRY SHEAR WALLS: Diaphragm openings immediately adjacent to exterior masonry shear walls are not greater than 8 ft (2.4 m) long. (Commentary: Sec. A.4.1.6. Tier 2: Sec. 5.6.1.3) **Comments:**
○	○	◉	○	STRAIGHT SHEATHING: All straight-sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered. (Commentary: Sec. A.4.2.1. Tier 2: Sec. 5.6.2) **Comments:**
◉	○	○	○	SPANS: All wood diaphragms with spans greater than 24 ft (7.3 m) consist of wood structural panels or diagonal sheathing. (Commentary: Sec. A.4.2.2. Tier 2: Sec. 5.6.2) **Comments:**
◉	○	○	○	DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 ft (12.2 m) and aspect ratios less than or equal to 4-to-1. (Commentary: Sec. A.4.2.3. Tier 2: Sec. 5.6.2) **Comments:**
○	○	◉	○	OTHER DIAPHRAGMS: Diaphragms do not consist of a system other than wood, metal deck, concrete, or horizontal bracing. (Commentary: Sec. A.4.7.1. Tier 2: Sec. 5.6.5) **Comments:**

Note: **C** = Compliant **NC** = Noncompliant **N/A** = Not Applicable **U** = Unknown

UC Campus:	San Francisco			Date:	May 22, 2020		
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ASCE 41-17
Collapse Prevention Structural Checklist For Building Type RM1-RM2

CONNECTIONS

C	NC	N/A	U	Description
○	○	○	◉	STIFFNESS OF WALL ANCHORS: Anchors of concrete or masonry walls to wood structural elements are installed taut and are stiff enough to limit the relative movement between the wall and the diaphragm to no greater than 1/8 in. (3 mm) before engagement of the anchors. (Commentary: Sec. A.5.1.4. Tier 2: Sec. 5.7.1.2) **Comments:** Specification for the out-of-plane wall anchors is not provided on the existing drawings.

Appendix C

UCOP Seismic Safety policy Falling Hazards Assessment Summary

UC Campus:	San Francisco			Date:	May 22, 2020		
Building CAAN:	2418	Auxiliary CAAN:		By Firm:	Estructure		
Building Name:	Oyster Point			Initials:	*ARK*	Checked:	*MTP*
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UCOP SEISMIC SAFETY POLICY
Falling Hazard Assessment Summary

	Description
P ☐ N/A ☒	**Heavy ceilings, features or ornamentation above large lecture halls, auditoriums, lobbies, or other areas where large numbers of people congregate (50 ppl or more)** **Comments:**
P ☐ N/A ☒	**Heavy masonry or stone veneer above exit ways or public access areas** **Comments:**
P ☐ N/A ☒	**Unbraced masonry parapets, cornices, or other ornamentation above exit ways or public access areas** **Comments:**
P ☐ N/A ☒	**Unrestrained hazardous material storage** **Comments:**
P ☐ N/A ☒	**Masonry chimneys** **Comments:**
P ☐ N/A ☒	**Unrestrained natural gas-fueled equipment such as water heaters, boilers, emergency generators, etc.** **Comments:**
P ☒ N/A ☐	**Other: Storage Racks** **Comments:** A large number of tall prefabricated steel storage racks are located in the building. The racks are bolted to the concrete slab-on-grade, and have a nominal lateral load system sized to accommodate operational loads of placing and removing heavily loaded palates. Each stanchion typically has two anchor bolts attached to the concrete floor, with some bolts not installed or missing. At some locations it was clear that the stanchions had been struck by forklifts and the anchor bolts were no longer reliable. These racks could pose a potential life safety issue or block exits if damaged in an earthquake.
P ☒ N/A ☐	**Other:** Freezers **Comments:**
P ☐ N/A ☐	**Other:** **Comments:**

Falling Hazards Risk: *Moderate*

Note: P= Present, N/A = Not Applicable; Falling Hazards Risk: Low, Moderate, or High

Appendix D

Quick Check Structural Calculations

Dead loads & Seismic Weight Calculation				
Roof				
Roofing		2	*psf*	*Assumed*
1/2" Plywood		1.5	*psf*	
Roof Subpurlins		1	*psf*	
Roof Purlins		1	*psf*	
Roof Girders		2	*psf*	
MEP		4	*psf*	
Misc		2	*psf*	
Exterior Walls		42	*psf*	*7.5" Thick Concrete Wall, conservatively includes inplane walls*
Total	$\sum$	**55**	**psf**	
Area	A_{roof}	117000	ft^2	
Seismic Weight	W	**6489**	**kips**	

Earthquake	Site Parameters - UCSF Group 3 Buildings – Tier 1 Geotechnical Assessment, Egan (2019)		
BSE-C	S_s = 1.64	F_a = 1.30	S_{Cs} = 2.13
	S_1 = 0.657	F_v = 4.2	S_{C1} = 2.758

Building Period			
Empirical factor	C_t	0.02	*ASCE 41-17 Sec. 4.4.2.4*
Roof level height	h	24.5 ft	
Empirical factor	β	0.75	*ASCE 41-17 Sec. 4.4.2.4*
Fundamental period, $T= C_t h_n^{\beta}$ = *ASCE 41-17 Sec. 4.4.2.4 eqn. 4-4*		0.220 sec	

Calculate Base Shear				
Spectral Acceleration	$S_a = S_{X1} / T$ =	12.52		*ASCE 41-17, 4.4.2.3*
	$S_{a,max} = S_{XS}$ =	2.13	*governs*	*ASCE 41-17, 4.4.2.3*
Modification Factor	C =	1.00		*ASCE 41-17, Table 4-7*
Pseudo Seismic Force	$V = S_a \times C \times W$ =	2.13 W		*ASCE 41-17, Eqn. 4-1*
	V =	**13821 kips**		

Seismic Force Vertical Distribution						
Level	Weight (kips)	Height (ft)	$w_x h_x$ (kip_ft)	$C_{vx} = w_x h_x / \sum w_x h_x$	$F_x = C_{vx} V$	Story Shear, V
Roof	6489	24.5	158969	1.00	13821	13821
∑	*6489*	∑	*158969*	*1.00*	*13821*	

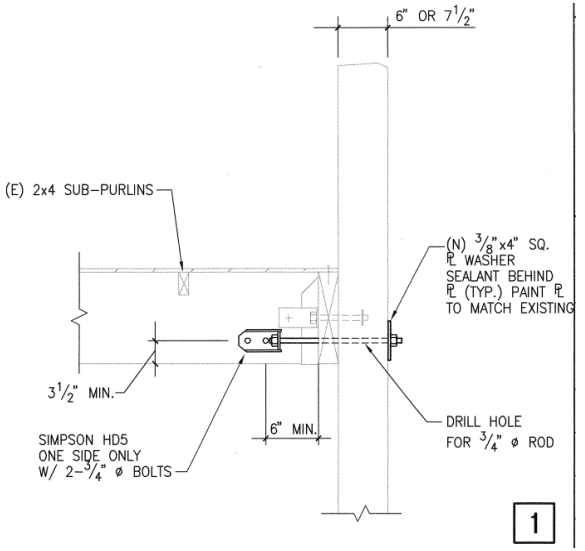
Longitudinal Direction (see plan Page 1 of report)							
Story	Story Shear (kips)	Length of Wall (ft)	Area of Wall (ft^2)*	M_s Factor (ASCE 41-17, Table 4-8)	Average Story Shear Stress (psi)	Quick Check Shear Capacity (psi)	Pass? (Y/N)
R	13821	994	497	4.5	43	100	Y

Transverse Direction (see plan Page 1 of report)							
Story	Story Shear (kips)	Length of Wall (ft)	Area of Wall (ft^2)*	M_s Factor (ASCE 41-17, Table 4-8)	Average Story Shear Stress (psi)	Quick Check Shear Capacity (psi)	Pass? (Y/N)
R	13821	634.5	317	4.5	67	100	Y

*Panel Thickness vary from 6" to 7.5". Quick check calculations conservatively use 6" typically.
**Conservatively does not include buttresses

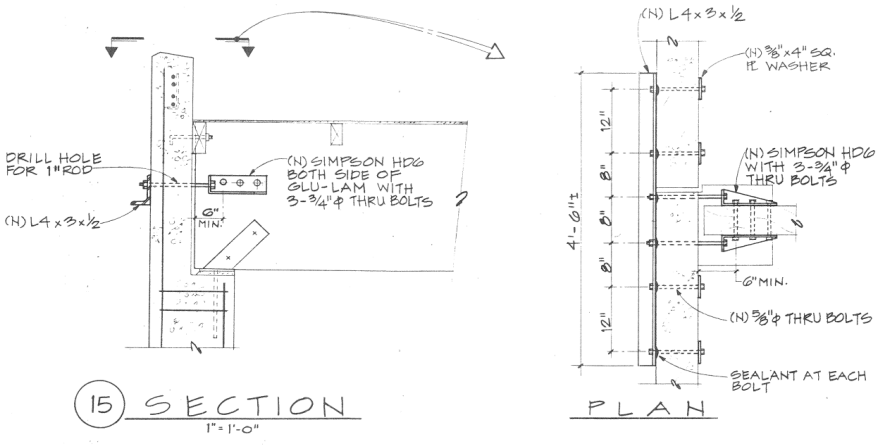
Wall Anchorage Check (ASCE 41-17, Section 7.2.11.1) - Purlin

Fp =	**17253**	**lbs**	(ASCE 41-17, eqn. 7-9)
χ =	0.9		(Collapse Prevention)
Sxs =	2.13		
Ap =	120	ft^2	(8 ft Purlin Spacing * (2.75 ft + 24.5/2))
wp =	93.75	psf	
k_a=	2		Max
k_h =	1		Flexible Diaphragm
HDU5 Capacity	**8704**	**lbs**	4835# (Simpson Capacity) / 1.6 * 0.85 * 3.32 Capacity From FEMA P2006 (Section 4.6.5.5)
D/C	**1.98**		

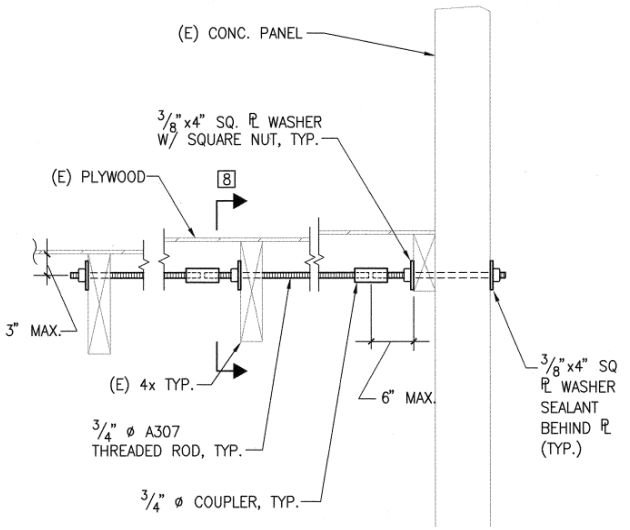


Wall Anchorage Check (ASCE 41-17, Section 7.2.11.1) - Glulam

Fp =	**13048**	**lbs**	(ASCE 41-17, eqn. 7-9)
χ =	0.9		(Collapse Prevention)
Sxs =	2.13		
Ap =	90.75	ft^2	(24.2 ft Glulam Spacing * (2.75 ft + 24.5/2)) / 4 Anchors
wp =	93.75	psf	
k_a=	2		Max
k_h =	1		Flexible Diaphragm
HDU5 Capacity	**12955**	**lbs**	7345# (Simpson Capacity - HD7) / 1.6 * 0.85 * 3.32 Capacity From FEMA P2006 (Section 4.6.5.5)
D/C	**1.01**		



1980s Renovation - Out of Plane Wall Anchorage at Glulam



1998 Renovation - 2 additional out of plane wall anchors added between Glulam Beams