STRUCTURAL REPORT
FOR

FORMER VETERANS MEMORIAL BUILDING
VALLEJO, CALIFORNIA

Prepared for

SOLANO COUNTY

April 12, 2013

TENNEBAUM-MANHEIM ENGINEERS
414 Mason Street, Suite 605
San Francisco, CA 94102

STRUCTURAL ASSESMENT AND EXISTING CONDITIONS

Tennebaum-Manheim Engineers (TMe) has been retained under two separate contracts by Solano County to perform a structural engineering assessment and preliminary analysis of the Former Veterans Memorial Building in Vallejo California. The purpose of the evaluation and assessment is to provide an understanding of the building's seismic performance which was originally intended to be incorporated in an Historic Structures Report. A structural report was completed under the first contract in which Tennebaum-Manheim Engineers provided an assessment and preliminary analysis based on visual observation and review of existing drawings based on ASCE 31. In that report, we recommended that the building undergo further investigation (following code requirements) to determine the strength of the concrete and steel, verification of reinforcing, floor ties, framing, and geotechnical recommendations for a future elevator. After review of the report and discussions, the County requested that the investigations be completed for a better understanding of the structure's seismic performance and for use in future feasibility assessments for the County. Tennebaum-Manheim Engineers developed an investigation plan and requirements based on structural engineering standards and reviewed and updated the preliminary analysis based on the findings of the investigation.

Background

The Former Veterans Memorial Building is a two story plus basement structure located at 444 Alabama Street, Vallejo, California (Solano County). Keith Hanson, Real Estate Manager with the County of Solano, found through discussions with Bill Tuikka (City Planner of the City of Vallejo) and the State Office of Historic Preservation, that the building is already a contributor to the St. Vincent's Hill Historic District listed on the National Register of Historic Places. Bill Tuikka also stated that the building is listed on the City's Historical Resources inventory, but not a designated landmark. The building was built in 1929 as the Former Veterans Memorial Building and houses a kitchen and banquet area in the basement, offices and meeting rooms on the first floor and auditorium, stage and balcony on the second floor. The use of the building has remained the same and very little has been altered since the time it was built.

Scope of Work

The scope of work of this study is limited to a structural evaluation of the building and a preliminary assessment of the existing building's capacity to resist earthquake forces based on the California State Historic Building Code. Prior to performing a preliminary seismic analysis we have used ASCE 31-03 Standard "Seismic Evaluation of Existing Buildings" which is a nationally accepted methodology for evaluating the seismic performance of existing buildings (see attached). ASCE 31-03 is a preliminary screening to identify possible problem areas. It serves as an indication of other areas which need more analysis or investigation. TMe identified areas which required further investigation according to code standards and an investigation was completed by MatriScope Engineering Laboratories. The results of the investigation have been reviewed, analyzed and incorporated in this report.

Sources of Information

Two site visits during the first contract were performed by Tennebaum-Manheim Engineers and the following documents were provided by the Owner for this study:

- Original drawings dated 6-29-29 (Photos).

Architectural drawings by Coffman-Sahlberg – Stafford - Sheets 1 -15
Structural drawings by Coffman-Sahlberg – Stafford – Sheets S2 – S7
Electrical drawings by Coddington Co. Mechanical Engineers – Sheets E1 –E4.
Mechanical drawings – Sheets H1 – H4.
Plumbing drawings – Sheets P1 – P3

- Fire Escape Addition – 1 drawing dated Dec. 1947 (Photo)

- Parking Lot Adjacent to Former Veterans Memorial Building designed by Solano County Engineer dated May 1958.

- Insulation Added - Drawing by Thermal-Tac Insulation Co. (Ceiling detail) dated January 1982.

- As-Built architectural drawings (by unknown) dated June 1991.

- Re-roofing Drawing – Date unknown.

Investigation:

Site visits were performed by Nancy Tennebaum during the investigation period on April 12, 2012 October 9, 2012 in which she reviewed areas of investigation with MatriScope. MatriScope provided the following investigation reports:

- Pit below concrete wall to expose continuous footing
- Concrete Core Compression Test Results
- Steel Coupons Test Results – columns and beams
- Concrete Wall Reinforcing (Daily Field Report)
- Floor to wall anchorage investigation (Daily Field Report)
- Balcony Framing (Daily Field Report)
- Limited Geotechnical Engineering Investigation – Proposed New Elevator Pit

Nancy Tennebaum and Dan Manheim, SE (Tennebaum-Manheim Engineers) performed a site visit on May 10, 2014 for a final review of the building.

BUILDING DESCRIPTION

General Description

The Former Veterans Memorial Building is a two story plus basement structure located at 444 Alabama Street, Vallejo, California (Solano County) with historic significance. The building was designed by Coffman-Sahlberg-Stafford Architects and Engineers and constructed in 1929. The structure is steel frame with wood infill at floors and roof. The exterior walls are shown on the original 1929 drawings as reinforced concrete with reinforced concrete beams below each floor level around the perimeter. The interior walls are wood frame. The foundation consists of square pad footings below the columns and continuous trenched wall footings.

The building is rectangular having overall plan dimensions of approximately 113 feet by 86.5 feet. The building has two stories above a complete basement. The story heights (floor to floor) are approximately 11 feet at the basement level, 17 feet at the first floor and 22 feet at the second floor auditorium to the ceiling. A balcony is 10 feet above the auditorium floor. There is an accessible attic space above the ceiling and the top of roof varies.

Very few alterations have been made to the building over the years and the structure appeared to be in good condition prior to our first report before the investigation phase. The alterations made from the original design include a new steel frame fire escape on the north side of the building (back side), a parking lot adjacent to the structure on the west side, insulation added in the ceiling, floor tiles in auditorium, and minor openings have been added at existing window openings for additional access to basement or mechanical ducts. Unfortunately, when arriving at the site on April 12, 2012 there was significant damage observed due to roof leaks and bird infestation, which had not been there previously.

Building Framing System

Walls:
The exterior walls are shown as reinforced concrete, 6” to 8” thick on the original structural drawings. The General Notes on Sheet S2 of the original drawings note wall reinforcing to be 3/8” square bars at 24” on center vertical and 3/8” square bars at 16” on center horizontal.

The MatriScope investigation however, did not find typical reinforcing as noted on the drawings. We were informed that they used a hand held device called an R-meter (pacometer) for locating the reinforcing in walls. MatriScope found no indication of reinforcing on an area of the east basement wall which had been chipped off over a horizontal span, approximately four feet long. In addition, Matriscope scanned over areas of concrete walls in the upper floors where furring had been removed and could not register reinforcing from their device. The exterior walls were scanned and some reinforcing was detected and located sporadically as well as form ties (1/4”x2” plates) which were found cast perpendicular to the walls at a spacing of 16” to 24” on center. Although we are not familiar with the limitations or success of finding reinforcing with an R-meter (pacometer), we believe it may be worthwhile to use another means of finding rebar in the walls, such as with ground penetrating radar. The reason we believe there is reinforcing in the wall is because we saw some exposed reinforcing on the front exterior window wall and north and west walls. In addition, Matriscope was unable to detect the #3 rebar accidentally cut in one of their concrete core samples. It may be that the placement of the reinforcing is covered too deep to detect with their equipment. We have had experience in the past with another project where concrete reinforcing was undetected from a hand held device and was subsequently discovered using another method of investigation. We also believe that reinforcing was placed in the building because the construction of the building appears to coincide with the original documents and both the architectural and structural drawings indicate reinforcing in the walls. The County may want to consider another method of investigation which may expose reinforcing and avoid unnecessary costs for future upgrade remedies.

Eighteen concrete core tests were obtained from the project as requested (6 per floor). The compressive tests ranged from 1,207 to 4,020 psi. The design strength used for preliminary analysis is the mean minus one standard deviation as defined in ASCE 41-06 section 6.2.2.3.1.

Basement walls concrete compressive strength Fc = 2644 psi (avg.)
First Floor walls concrete compressive strength = Fc = 1452 psi (avg.)
Second Floor walls concrete compressive strength Fc = 1994 psi (avg.)

The default Lower-Bound Compressive strength of structural concrete for buildings of this time period is 2,000 to 3,000 (ASCE 41-06 Table 6.3). Therefore, we can conclude that the first floor walls are of poor strength relative to the other buildings constructed during this era, although it would be typical for buildings built with the time frame of 1900 to 1919 which ranges from 1000 psi to 2500 psi. We determined that the strength of concrete may be adequate for this building; however the lack of apparent reinforcing is a concern.

We calculated that the allowable capacity of the concrete + reinforcing (as noted on the drawings) at the basement level, 1st floor level and 2nd floor level. We found that the existing capacity of the concrete with the # 3 square bar reinforcing as noted on the drawings have the capacity to resist the code level forces per Historic Building Code. Because Matriscope was unable to detect reinforcing, we must assume the building does not meet the Historic Building Code requirements or current code. Therefore, if the County decided to upgrade the building, we would recommend an upgrade method of either applying fiberwrap to the existing concrete wall surface or reinforced shotcrete walls on portions of the perimeter walls (on one side).

Reinforced concrete beams spanning below diaphragms were provided to prevent the walls from falling outward by spanning between beams. This is also called out of plane wall loading, in which the walls are prevented from falling outward by spanning between the beams. Concrete pilasters or steel columns at the exterior walls support the incoming concrete or steel beams. The interior walls are wood frame stud walls.

Foundation:
The building foundation system consists of individual reinforced concrete spread footing pads under the interior and exterior columns (see foundation plan figure 1). The column footings are typically square shaped having plan dimensions ranging from 4'-9" to 9'-6" and 15" to 24" deep. At the base of each steel column are 2 steel angles 6"x4"x1/2" on a 1" to 2" thick steel base plate over grout with 4-3/4" diameter bolts into footing.

As a part of the investigation, a 4x4 test pit was excavated inside the building basement by MatriScope, in order to determine the undocumented footing below the concrete wall. The exterior concrete wall extends approximately 7" below the bottom of the concrete slab and has a continuous footing approximately 14" deep poured in a trench. The footing appeared to be V-shape and extends 13" from the wall face (see attached sketch by MatriScope). The continuous wall footings are between the square column footings. We believe this would be adequate for transferring shear from the walls into the foundation.

MatriScope found that the existing concrete slab in basement is 3 ½" thick with wire mesh 3/16" thick and spaced at 6" on center in each direction.

1st Floor Framing:
The first floor framing was viewed from below and seen to have diagonal sheathing (planking). The sheathing is supported over 2x14 wood joists @ 16" on center spanning to steel "I" beams. The "I" beams span in both directions and are supported on steel columns of built up sections or steel "H" sections or reinforced concrete pilasters. The ceiling supported below the beams is plaster. A second location was viewed by MatriScope which exposed 2"x13" (actual) joists at 16" on center supporting diagonal ¾" sheathing + hardwood flooring. No wall anchorage from floor to wall was found at either location (parallel or perpendicular to joists). All joists appeared to be in good condition.

2nd Floor Framing:
A 12"x12" hole was cut in the second floor at the auditorium by the east wall to view the existing conditions. At the location cut, the flooring consists of 1/8" vinyl flooring over ¾" x6" planking, over 2-7/8" x 2-3/4" sleepers over ¾" x7-1/2" straight sheathing over 2x 14 joists (13" actual depth). There is a plaster ceiling below the wood joists. The joists span to long spanning deep built up steel beams or shorter spanning "I" beams, which are supported by steel columns or concrete pilasters. The steel beams

span in both directions from columns. The floor framing investigated by MatriScope documented that the floor framing was 2”x13” at 16” on center with straight sheathing + hardwood flooring above and 2x2 bridging between the joists. Wall anchorage to floor framing was not found (parallel or perpendicular to joists). All joists appeared to be in good condition.

Balcony:
The balcony is wood frame and drawings indicate sloped 2x14 beams at 16” on center and 2-2x10 floor/ceiling joists (spacing unknown) supported by steel beams. During the investigation, two holes were cut in the floor of the theater balcony. The exposed framing was found to consist of ¾” sheathing over 2”x 9” (actual) single and sistered joists at varied spacing of 10” to 12” apart. These members are in agreement with those shown in the existing drawings. They appeared to be in good condition. Because the framing is not uniform throughout, we are unable to adequately assess the total capacity; however; the original documents indicate that the balcony was designed for a live load of 75 psf. Current code defines a minimum live load for assembly with fixed seating to be 60 psf minimum and assembly with moveable seats (or no seats) to be 100 psf. Therefore if there is fixed seating on the balcony, the existing framing and support should be adequate if it is constructed as designed.

Roof Framing:
The drawings indicate that the original roofing consists of composite roofing material. (We did not go on top of the roof to view this). There is access through the storage room into the attic space (approximately 2’-6” to 4’ clearance) and we were able to partially verify the framing and conditions within that space. The roofing is supported on straight sheathing (slightly sloped for roof drainage) typically on 2x6 roof rafters at 24” on center. Where rafters span 25’-6” at south side (east and west corners) roof rafters are 2x14 at 16” on center. The joists are picked up by steel trusses or steel “I” beams as noted on the roof plan. Horizontal 7/8” diagonal rods span in two bays of the structure at the ceiling level. Ceiling joists are 2x6 at 16” on center and support the plaster ceiling below with fiberglass batt insulation above. Insulation was added to the building in 1982. There is also a drawing indicating that the built-up roof was replaced.

In our findings, none of the steel has fireproofing and all visible framing appears to be in good condition.

Steel Coupon Testing by MatriScope Engineering Laboratories:
Fourteen (14) steel coupons were taken for analysis from existing steel columns and beams from basement and first floor and roof truss members. Spectrochemical analysis and tensile strength tests were performed on the coupons. The tensile and yield strengths were very high (range from 49,900 to 70,100 psi) which means the members have a great deal of capacity for bending and axial loading. In addition, the carbon equivalent was found to be low (usually below 0.3) which means the steel is acceptable for welding. Welding may be required for future upgrade work, such as the proposed horizontal bracing shown on the roof plan following.

Allowable Live Loads:

The original drawings state the live loads allowed on the following levels to be:

Roof: 30 psf
Second Floor: 125 psf
First Floor: 100 psf
Balcony Floor: 75 psf

The design live load of the roof is 10psf greater than what the current code requires. Therefore additional weight for solar panels, which typically weigh less than 5 psf may be considered for this building in the future. We also calculated the member capacity of the wood joists and believe the joists can sustain at least 5 psf additional weight on the roof. The entire system should be investigated more thoroughly before additional weight is placed on the roof.

LATERAL SYSTEM

It should be noted that a key requirement of an appropriate seismic system is to provide continuous connection between all members resisting seismic forces starting from the points where these forces are generated (typically building floors and roof), through floor diaphragms and shear walls and frames, down to the foundation and supporting soil. In engineering terminology, this feature is referred to as a "complete load path". The Veterans Building Load path is substantially complete.

The following descriptions present the main features of the building's seismic resisting system.

The building's wood floors and roof serve as the seismic resisting diaphragms and transfer the seismic inertia forces to the lateral resisting walls. It is expected that the original straight sheathing of the original roof and floor construction do not have sufficient capacity for resisting seismic forces by current code.

The exploratory hole at the second floor allowed us to review the conditions of attachment between the floor framing members, columns and walls. We believe the roof and floor diaphragms (floor planking) are nailed to the wood joists and the joists are connected to the steel beams by nailers and bolts. The steel beams have riveted connections to the steel columns and steel girders. We believe the steel beams and columns have attachment to the walls where they intersect, however MatriScope did not find any anchorage between the wood diaphragm and walls, which creates a load path deficiency.

Concrete walls must be anchored to the floor diaphragms to prevent out of plane falling and to transfer seismic shear parallel to the walls. This is required in current code; however this was not standard practice at the time this building was constructed. We did not see evidence of hardware from the wall to beam or floor framing, which indicates that there may be a deficiency in the out of plane wall connections. Investigation by MatriScope indicates that there are no anchors from the floor or roof framing into the walls. There are concrete beams below the floor joists at each level. It may be that the walls were designed to span between the concrete beams for out of plane loading. The concrete beams in turn span to the concrete pilasters. However, because MatriScope could not locate wall reinforcing during the investigation phase, the walls may not currently meet out of plane wall loading. Where there are steel beams encased in concrete spanning between columns, the walls should be adequate in resisting out of plane loading. This only occurs at the floor and roof of the auditorium walls on the east and west sides. It appears that the building was well designed and detailed for its time, however it is unclear whether it was constructed with the reinforcing as shown on the drawings and may be inadequate unless proven otherwise by further wall reinforcing investigation.

The exterior concrete walls serve as the main vertical resisting seismic elements. There are some interior wood frame walls at each level, however they were not designed to sustain horizontal forces and because they do not have continuous footings below, we believe that they simply behave as partition walls. They may in reality take some horizontal loading from the diaphragm. Our preliminary analysis assumes that the exterior reinforced concrete walls act as the shear walls and take all of the seismic loading in both directions.

Geotechnical Investigation and Recommendations

A geotechnical investigation was performed for understanding of the soil conditions at the site as well as for providing information for the incorporation of an elevator. MatriScope has provided a limited geotechnical engineering investigation and recommendations for the new elevator pit. In summary, a boring was taken at the western side of the building. Subsurface soils consisted of about 2.5 feet of silty sand underlain by hard siltstone (bedrock) to the maximum depth drilled of approximately 30.5 feet below the existing grade. Water was encountered at a depth of approximately 13 feet below the existing ground surface.

Geotechnical recommendations were provided for temporary excavations and shoring design, engineered fill and trench backfill, slab and foundation design including lateral earth pressures and bearing pressures. Seismic design parameters were also provided. Laboratory tests were performed for soil corrosivity. According to MatriScope, the soils are not considered to have high corrosive potential to buried metallic improvements. In addition, the laboratory tests do not indicate a significant corrosive potential to buried concrete structures. The Geotechnical Report is attached in Appendix following.

Existing Conditions

As stated previously the structure appeared to be in excellent condition in most areas during our first visit before the investigation phase began. We previously noted there are small minor areas on the exterior façade that have some spalling and rusting. During our last site visit we found more areas of exposed reinforcing on the back of the north wall, west wall and stair tower on the east side. We recommend that the reinforcing be cleaned and concrete be replaced to prevent further rusting. There is also some cracking and deterioration at ornamental window balconies (see photos in appendix).

We noticed during the investigation phase that the building has undergone some deterioration due to water leaks at roof and windows as well as infestation from birds, including dead birds throughout the building. We recommend that the structure be repaired or covered adequately to prevent further leakage. We also recommend that the dead birds and debris be removed as soon as possible.

As stated in our letter regarding Structural Assessment Report Comments, dated 12-19-12, "All existing concrete removed for coring must be replaced with non-shrink grout after review by Engineer and tests are completed". In addition, any concrete removed during investigation must be replaced with non-shrink grout. This affects the integrity of the building and is required by engineering standards noted in Seismic Rehabilitation of Existing Structures ASCE 41-06. In addition during our last site visit, we found steel coupons were removed in the incorrect location on the columns at the stairs of the first floor. The investigation plans provided by TMe specified a coupon from the web, however the flange was removed. This must be repaired with a steel plate the same thickness of the flange and welded on three sides (Investigation plan showing location of columns is attached in appendix).

The wood members visible in the roof and in some floor areas appear to be in very good condition. Upon our first visit to the site, no signs of rot and no visible sagging were seen. During the investigation phase site visits, water damage was observed where leaking occurred on second floor and ceiling below collapsed.

The steel members seen in the roof framing are in very good condition. There is no fireproofing on these members.

There are no apparent signs of settlement within the building and outside the structure which indicates that it is on adequate soil and is now substantiated with the geotechnical investigation report attached.

There may be asbestos in the vinyl flooring or other areas, however this is not in our expertise and is not a part of our scope of work.

Seismic Performance:

The building was well designed and detailed for its time; however it does not meet current code as is the case with presumably any building built during early 1900s. By code, the building does not require a seismic upgrade if it maintains the same occupancy and use, however it is important that the County understand the deficiencies in order to make a decision whether any upgrade work is warranted. We first evaluated the structure using a rapid evaluation method using ASCE Standard ASCE/SEI 31-03 "Seismic Evaluation of Existing Buildings", in which the following deficiencies were determined:

- There is minor deterioration in the exterior concrete primarily by window openings. Some spalling has occurred indicating corroded steel.
- The existing reinforcing (as noted in the original drawings) in the concrete walls is less than the minimum required for Life Safety. (The investigation by MatriScope was unable to locate the wall reinforcing as noted on the drawings and only found sporadic wall reinforcing using an R- meter).
- We did not find adequate connection from the diaphragm to allow transfer of load to the shear walls. The investigation by MatriScope did not find any hardware connecting the floor diaphragms to the concrete shearwalls.
- The diaphragms are primarily straight sheathed (at roof and second floor) and have spans greater than 24 feet which is greater than permitted for Life Safety. This means there could be a lot of out of plane displacement of the walls.

We also performed a preliminary structural analysis using the 2007 California Historic Building Code Chapter 8-7. The Historic code states "Seismic loads need not exceed .75 times the seismic forces prescribed by the 1995 Edition of the California Building code.

The basis for design is as follows:

Seismic V = $\dfrac{ZICW}{R_w}$

Z= 1.0 I=1.0 C-2.75 R_w= 8 W=2,263 Kips (EW Direction)
W=2,072 Kips (NS Direction)

V_{EW}= .103 (2263) = 233.1 KIPS

V_{NS}= .103 (2072) = 213.4 KIPS

Using the above as our design level earthquake for life safety we found the following deficiencies:

- The roof diaphragm with straight sheathing, does not meet the required code capacity. The allowable shear/ft based on Table 8-8A-Allowable Values for Existing Materials is less than the code design shear values. Therefore, this is not adequate.

- The first floor diaphragms have diagonal sheathing and second floor has straight sheathing found from exposed areas viewed at the second floor and first floors. The 2nd floor composition (in auditorium) exposed prior to the investigation phase did not have evidence of hardwood flooring over the sheathing, however MatriScope found that hardwood flooring was seen in other exposed areas. Straight sheathing alone is five times less in shear capacity than finished flooring over straight sheathing. Floor diaphragms which have straight sheathing only (with no hardwood flooring above) may be deficient in shear capacity. The actual maximum design shear forces based on our analysis are greater than the allowable shear based on 2010 CBC Table 8-8A Allowable Values for Existing Materials for straight sheathing. Transfer of loads from the floors to the walls may not be possible if diaphragms do not have adequate capacity. MatriScope did find evidence of finished flooring over the sheathing during the investigation phase and we believe those areas do have adequate capacity to transfer loads to the walls provided the walls are attached to the floor. Unfortunately, MatriScope did not find any hardware attaching the floor diaphragm to the walls for shear transfer, thus creating a deficiency in the load path.

- The original drawings indicate that there is 3/8" square horizontal and vertical reinforcing in the concrete walls; however MatriScope could not locate the reinforcing using their R-meter device. Without the minimal reinforcing, as noted on the drawings, the structure is not adequate for resisting lateral loads. Should it be found that there actually is reinforcing as noted on the drawings, we believe the building would be adequate based on the Historic Building Code design level forces.

CONCLUSIONS AND RECOMMENDATIONS

The following conclusions are based on our visual observations, review of existing documents and an investigation provided by MatriScope of the Former Veterans Memorial Building. We also have performed limited structural calculations to determine lateral deficiencies of the building.

The Former Vallejo Veterans Memorial Building is located at 444 Alabama Street and is nearest to the Rogers Creek and Napa faults. It is not within the Alquist-Priolo Earthquake Fault Zone, which means it is not located on a rupture zone according to USGS hazard maps. Based on a preliminary structural lateral analysis, we found that the building does not meet current code design values for lateral resistance. Wind is determined not to govern because of the mass of the building. We believe the existing structure is deficient in both directions. The Former Veterans Memorial Building is analyzed using the Historic Building Code, which does provide values for archaic materials and lower seismic shear design forces. Engineering judgment is also permitted to some extent.

According to Code, the building does not require a mandatory seismic upgrade if the use and occupancy remains the same. The California Building Code, Section 3408.4 states: " When a change of occupancy results in a structure being reclassified to a higher occupancy category, the structure shall conform to the seismic requirements for a new structure of the higher occupancy category". A change of higher occupancy classification, alterations or additions may require a mandatory seismic upgrade. Solano County may want to take measures to further protect the building and occupants and therefore we are providing a priority list of recommendations to improve the building in bringing it up to code.

Priority #1 – As stated in our letter regarding Structural Assessment Report Comments, dated 12-19-12, "All existing concrete removed for coring must be replaced with non-shrink grout after review by Engineer and tests are completed". In addition, any concrete removed during investigation must be replaced with non-shrink grout. This affects the integrity of the building and is required by engineering standards noted in Seismic Rehabilitation of Existing Structures ASCE 41-06. In addition, steel coupons were taken in the incorrect location on the columns at the stairs of the first floor which weakens the columns. Columns are noted as CLM 1st Floor #1 and CLM 1st Floor #2. The investigation plans provided by TMe specified a coupon from the web, however the coupon was taken from the column flange. This must be repaired with a steel plate the same thickness of the flange and welded on four sides.

Priority #2- Building should be protected from water and bird infestation. This is important to the structural integrity.

Priority #3 – Repair the exterior façade where spalling and exposed reinforcing has occurred. Clean reinforcing and patch concrete. (See photos attached).

Priority #4 – Design and detail for out of plane wall loading and tying the floor diaphragms to the walls. This will require anchorage from the wall beam to the flooring. Anchors usually consist of drilled and epoxy anchors with straps tying the wall to the floor framing. Blocking below the straps and floor diaphragm would be added to develop the out of plane wall load into the floor diaphragm. See attached detail.

Priority #5 –If wall reinforcing is actually deficient or less than that noted on the original drawings as determined by Matriscope, we recommend adding a shear resisting element to the walls, for example; 2 layers of fiberwrap over a length of approximately 40 feet on the inside of the north, south, east and west perimeter walls of the building at each floor level, or reinforced 6" shotcrete walls on the interior face of concrete perimeter walls to be within the existing furred wall depth.

Priority #6 – The second floor diaphragm shear forces may be too high at boundaries where there is no finished hardwood flooring over sheathing and could be remedied by adding plywood below a portion of the ceiling or above the floor framing. Another remedy would be to add interior plywood shear walls plus footings between the 2nd and 1st floor and 1st floor and basement which would reduce the diaphragm span and forces.

Priority #7 – The roof diaphragm shear forces are too high and could be remedied by adding horizontal bracing shown on roof plan attached.

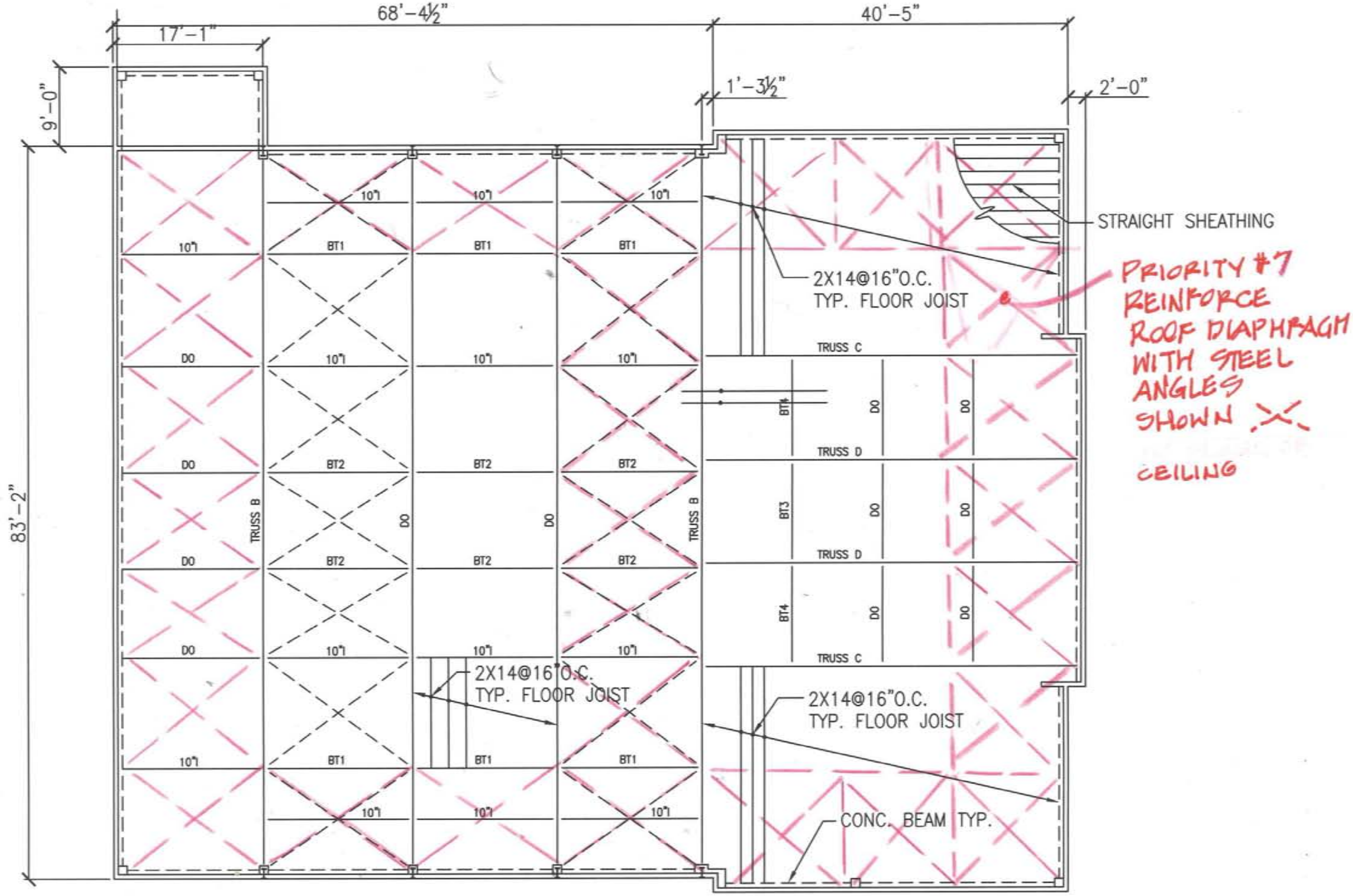
Other issues:

An elevator may be added in the future and we would recommend that it be placed on the exterior of the building by the parking lot. Using an existing window would reduce the amount of historic fabric to be removed and would help to maintain the structural integrity of the building.

The County may want to consider adding solar panels to the building in the future. According to the original drawings the roof framing was designed for a 30 psf live load, which is 10 psf greater than required by current code. Solar panels typically weight less than 5psf. Therefore, it is feasible to add solar panels to the building with little or no reinforcing required and no seismic upgrade. A seismic upgrade is not triggered provided an addition does not increase the seismic forces in any structural element of the existing structure by more than 10% cumulative since the original construction.

The purpose of this evaluation report is to identify structural deficiencies that may pose a risk to life-safety during the design basis earthquake. This evaluation is not intended to identify all structural defects in the existing framing, gravity supporting systems, or lateral-force resisting systems. The findings in this report are based on a review of historical drawings and limited site observations or exposed members. Some conclusions and information presented in this report are dependent on information that has been provided to us. Additionally, a number of factors make it difficult to fully and easily assess the current condition of all existing structural elements. These factors include, but are not limited to; limited available documentation, limited accessibility to visually confirm existing conditions, limited knowledge about the consistency/quality of construction during erection, and lack of as-built drawings which chronicle deviations from the original design.

The services performed for this project have been provided at a level that is consistent with the general level of skill and care that is typically provided by engineers practicing Structural Engineering. Work completed is done under the constraints of time and budget. No warranty is expressed or implied.



ROOF FRAMING PLAN

Intrepretation of Original Drawings

414 Mason Street, Suite 605
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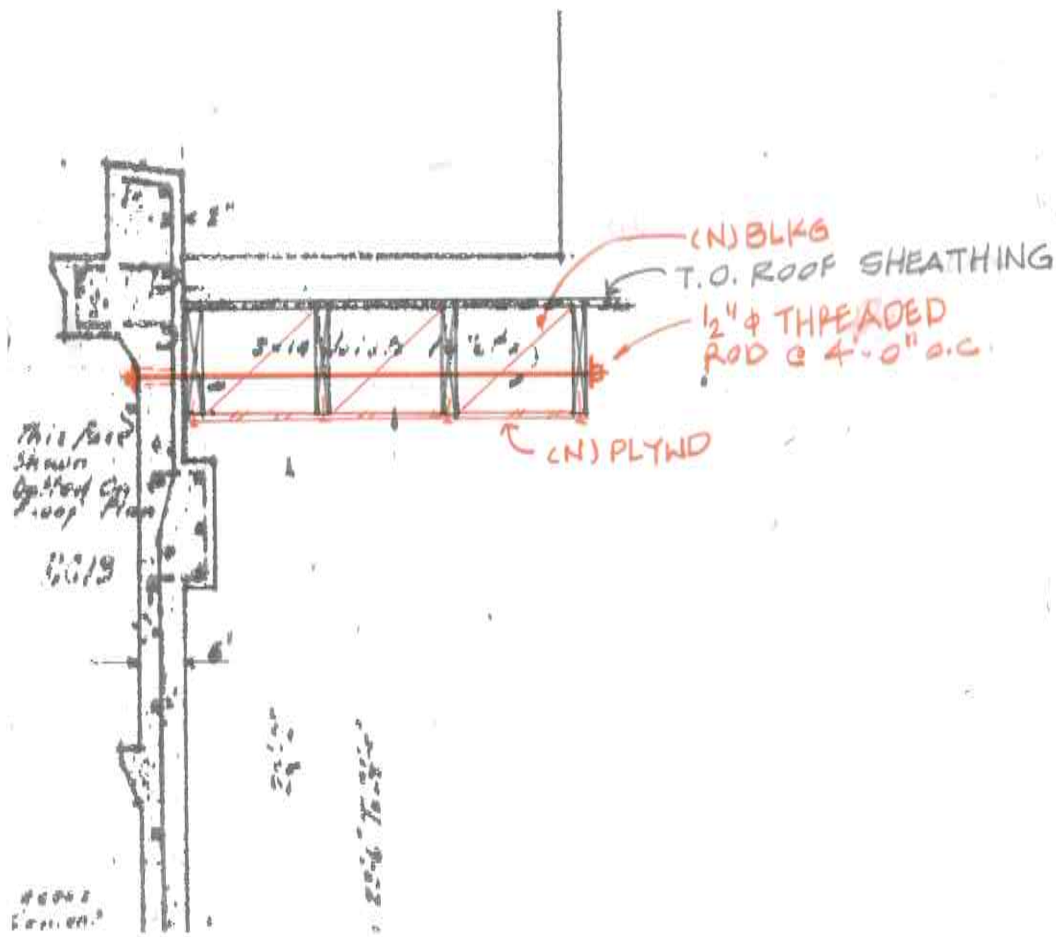
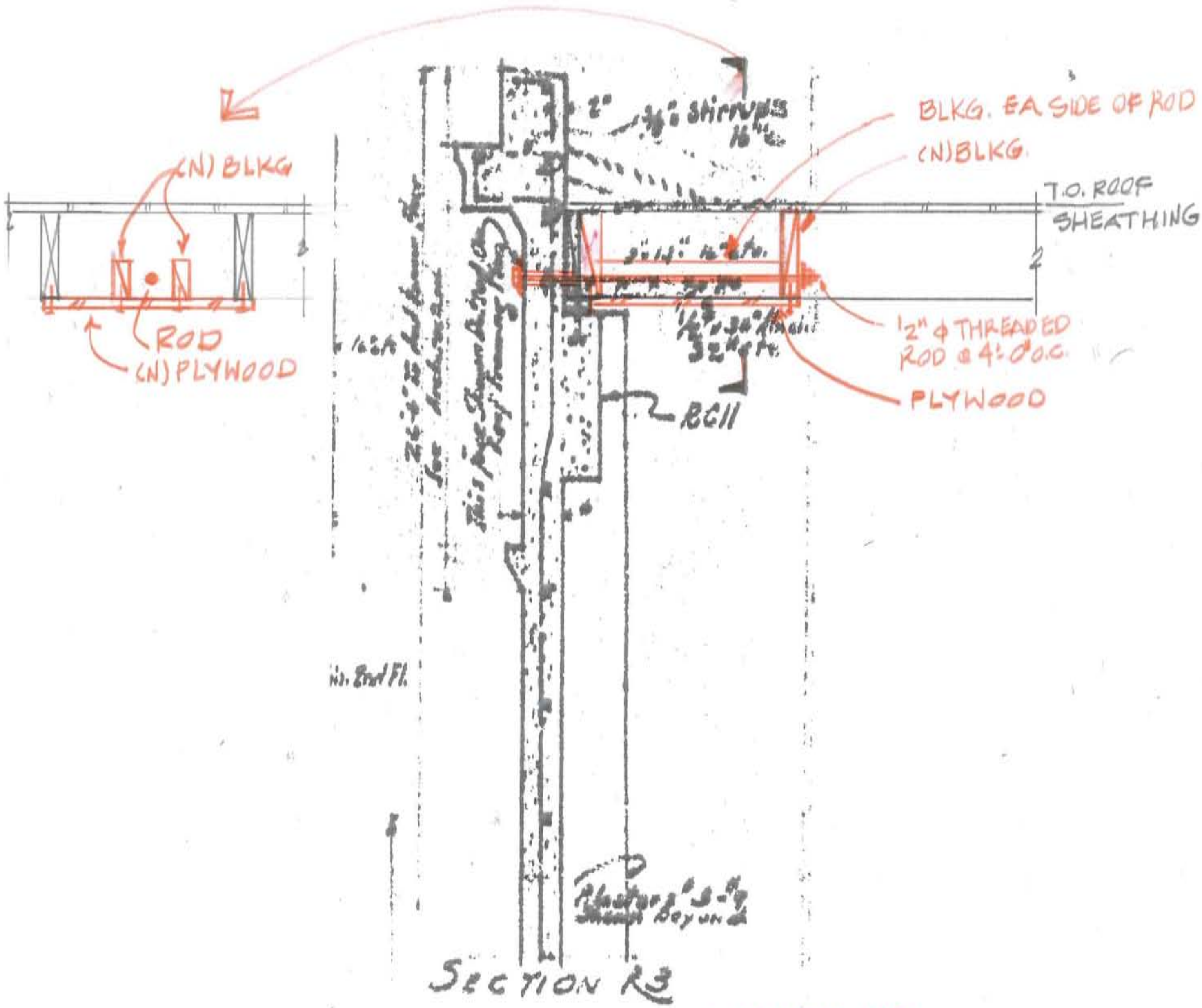
415.772.9891



FLOOR TO WALL ANCHORAGE
JOISTS PARALLEL TO WALL - CONCEPTUAL
FIG. 1

TENNEBAUM-MANHEIM ENGINEERS
CONSULTING STRUCTURAL ENGINEERS

414 Mason Street, Suite 605 San Francisco, CA 94102 415.772.9891	VETERANS MEMORIAL	Job No. Date:	Sheet No. 13 By NT



FLOOR TO WALL ANCHORAGE
JOISTS PERPENDICULAR TO WALL

CONCEPTUAL

FIG. 2

APPENDIX A

TIER 1
SCREENING PHASE
ASCE STANDARD

SEISMIC EVALUTAION OF EXISTING BUILDINGS

3.7.9A Basic Structural Checklist for Building Type C2A: Concrete Shear Walls with Flexible Diaphragms

This Basic Structural Checklist shall be completed where required by Table 3-2.

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

C3.7.9A Basic Structural Checklist for Building Type C2A

These buildings have floor and roof framing that consists of wood sheathing on wood framing and concrete beams. Floors are supported on concrete columns or bearing walls. Lateral forces are resisted by cast-in-place concrete shear walls. In older construction, shear walls are lightly reinforced but often extend throughout the building. In more recent construction, shear walls occur in isolated locations and are more heavily reinforced with boundary elements and closely spaced ties to provide ductile performance. The diaphragms consist of wood sheathing or have large aspect ratios and are flexible relative to the walls. Foundations consist of concrete spread footings or deep pile foundations.

Building System

C NC N/A LOAD PATH: The structure shall contain a minimum of one complete load path for Life Safety and Immediate Occupancy for seismic force effects from any horizontal direction that serves to transfer the inertial forces from the mass to the foundation. (Tier 2: Sec. 4.3.1.1)

C NC N/A ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building shall be greater than 4 percent of the height of the shorter building for Life Safety and Immediate Occupancy. (Tier 2: Sec. 4.3.1.2)

C NC N/A MEZZANINES: Interior mezzanine levels shall be braced independently from the main structure, or shall be anchored to the lateral-force-resisting elements of the main structure. (Tier 2: Sec. 4.3.1.3)

C NC N/A WEAK STORY: The strength of the lateral-force-resisting system in any story shall not be less than 80 percent of the strength in an adjacent story, above or below, for Life Safety and Immediate Occupancy. (Tier 2: Sec. 4.3.2.1)

C NC N/A SOFT STORY: The stiffness of the lateral-force-resisting system in any story shall not be less than 70 percent of the lateral-force-resisting system stiffness in an adjacent story above or below, or less than 80 percent of the average lateral-force-resisting system stiffness of the three stories above or below for Life Safety and Immediate Occupancy. (Tier 2: Sec. 4.3.2.2)

C NC N/A GEOMETRY: There shall be no changes in horizontal dimension of the lateral-force-resisting system of more than 30 percent in a story relative to adjacent stories for Life Safety and Immediate Occupancy, excluding one-story penthouses and mezzanines. (Tier 2: Sec. 4.3.2.3)

C NC N/A VERTICAL DISCONTINUITIES: All vertical elements in the lateral-force-resisting system shall be continuous to the foundation. (Tier 2: Sec. 4.3.2.4)

C NC N/A — MASS: There shall be no change in effective mass more than 50 percent from one story to the next for Life Safety and Immediate Occupancy. Light roofs, penthouses, and mezzanines need not be considered. (Tier 2: Sec. 4.3.2.5)

C NC N/A — DETERIORATION OF WOOD: There shall be no signs of decay, shrinkage, splitting, fire damage, or sagging in any of the wood members, and none of the metal connection hardware shall be deteriorated, broken, or loose. (Tier 2: Sec. 4.3.3.1)

C NC N/A — DETERIORATION OF CONCRETE: There shall be no visible deterioration of concrete or reinforcing steel in any of the vertical- or lateral-force-resisting elements. (Tier 2: Sec. 4.3.3.4)

MINOR

C NC N/A — POST-TENSIONING ANCHORS: There shall be no evidence of corrosion or spalling in the vicinity of post-tensioning or end fittings. Coil anchors shall not have been used. (Tier 2: Sec. 4.3.3.5)

C NC N/A — CONCRETE WALL CRACKS: All existing diagonal cracks in wall elements shall be less than 1/8 inch for Life Safety and 1/16 inch for Immediate Occupancy, shall not be concentrated in one location, and shall not form an X pattern. (Tier 2: Sec. 4.3.3.9)

Lateral-Force-Resisting System

C NC N/A — REDUNDANCY: The number of lines of shear walls in each principal direction shall be greater than or equal to 2 for Life Safety and Immediate Occupancy. (Tier 2: Sec. 4.4.2.1.1)

C NC N/A — SHEAR STRESS CHECK: The shear stress in the concrete shear walls, calculated using the Quick Check procedure of Section 3.5.3.3, shall be less than the greater of 100 psi or $2\sqrt{f'c}$ for Life Safety and Immediate Occupancy. (Tier 2: Sec. 4.4.2.2.1)

C NC N/A — REINFORCING STEEL: The ratio of reinforcing steel area to gross concrete area shall be not less than 0.0015 in the vertical direction and 0.0025 in the horizontal direction for Life Safety and Immediate Occupancy. The spacing of reinforcing steel shall be equal to or less than 18 inches for Life Safety and Immediate Occupancy. (Tier 2: Sec. 4.4.2.2.2)

3/8" □ @ 24" o.c. VERTICALLY
3/8" □ @ 16" o.c. HORIZONTALLY

Connections

C NC N/A — WALL ANCHORAGE: Exterior concrete or masonry walls that are dependent on the diaphragm for lateral support shall be anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections shall have adequate strength to resist the connection force calculated in the Quick Check procedure of Section 3.5.3.7. (Tier 2: Sec. 4.6.1.1)

C NC N/A — TRANSFER TO SHEAR WALLS: Diaphragms shall be connected for transfer of loads to the shear walls for Life Safety and the connections shall be able to develop the lesser of the shear strength of the walls or diaphragms for Immediate Occupancy. (Tier 2 Sec. 4.6.2.1)

UNKNOWN BUT LIKELY

C NC N/A — FOUNDATION DOWELS: Wall reinforcement shall be doweled into the foundation for Life Safety, and the dowels shall be able to develop the lesser of the strength of the walls or the uplift capacity of the foundation for Immediate Occupancy. (Tier 2: Sec. 4.6.3.5)

3.7.9AS Supplemental Structural Checklist for Building Type C2A: Concrete Shear Walls with Flexible Diaphragms

This Supplemental Structural Checklist shall be completed where required by Table 3-2. The Basic Structural Checklist shall be completed prior to completing this Supplemental Structural Checklist.

Lateral-Force-Resisting System

C NC N/A COUPLING BEAMS: The stirrups in coupling beams over means of egress shall be spaced at or less than $d/2$ and shall be anchored into the confined core of the beam with hooks of 135° or more for Life Safety. All coupling beams shall comply with the requirements above and shall have the capacity in shear to develop the uplift capacity of the adjacent wall for Immediate Occupancy. (Tier 2: Sec. 4.4.2.2.3)

C NC N/A OVERTURNING: All shear walls shall have aspect ratios less than 4-to-1. Wall piers need not be considered. This statement shall apply to the Immediate Occupancy Performance Level only. (Tier 2: Sec. 4.4.2.2.4)

C NC N/A CONFINEMENT REINFORCING: For shear walls with aspect ratios greater than 2-to-1, the boundary elements shall be confined with spirals or ties with spacing less than $8d_b$. This statement shall apply to the Immediate Occupancy Performance Level only. (Tier 2: Sec. 4.4.2.2.5)

No

C NC N/A REINFORCING AT OPENINGS: There shall be added trim reinforcement around all wall openings with a dimension greater than three times the thickness of the wall. This statement shall apply to the Immediate Occupancy Performance Level only. (Tier 2: Sec. 4.4.2.2.6)

C NC N/A WALL THICKNESS: Thickness of bearing walls shall not be less than 1/25 the unsupported height or length, whichever is shorter, nor less than 4 inches. This statement shall apply to the Immediate Occupancy Performance Level only. (Tier 2: Sec. 4.4.2.2.7)

Diaphragms

C NC N/A DIAPHRAGM CONTINUITY: The diaphragms shall not be composed of split-level floors and shall not have expansion joints. (Tier 2: Sec. 4.5.1.1)

C NC N/A CROSS TIES: There shall be continuous cross ties between diaphragm chords. (Tier 2: Sec. 4.5.1.2)

C NC N/A OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls shall be less than 25 percent of the wall length for Life Safety and 15 percent of the wall length for Immediate Occupancy. (Tier 2: Sec. 4.5.1.4)

C NC N/A PLAN IRREGULARITIES: There shall be tensile capacity to develop the strength of the diaphragm at re-entrant corners or other locations of plan irregularities. This statement shall apply to the Immediate Occupancy Performance Level only. (Tier 2: Sec. 4.5.1.7)

C NC N/A DIAPHRAGM REINFORCEMENT AT OPENINGS: There shall be reinforcing around all diaphragm openings larger than 50 percent of the building width in either major plan dimension. This statement shall apply to the Immediate Occupancy Performance Level only. (Tier 2: Sec. 4.5.1.8)

C NC N/A STRAIGHT SHEATHING: All straight sheathed diaphragms shall have aspect ratios less than 2-to-1 for Life Safety and 1-to-1 for Immediate Occupancy in the direction being considered. (Tier 2: Sec. 4.5.2.1)

C NC N/A SPANS: All wood diaphragms with spans greater than 24 feet for Life Safety and 12 feet for Immediate Occupancy shall consist of wood structural panels or diagonal sheathing. (Tier 2: Sec. 4.5.2.2)

C NC N/A UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms shall have horizontal spans less than 40 feet for Life Safety and 30 feet for Immediate Occupancy and shall have aspect ratios less than or equal to 4-to-1 for Life Safety and 3-to-1 for Immediate Occupancy. (Tier 2: Sec. 4.5.2.3)

C NC N/A NON-CONCRETE FILLED DIAPHRAGMS: Untopped metal deck diaphragms or metal deck diaphragms with fill other than concrete shall consist of horizontal spans of less than 40 feet and shall have span/depth ratios less than 4-to-1. This statement shall apply to the Immediate Occupancy Performance Level only. (Tier 2: Sec. 4.5.3.1)

C NC N/A OTHER DIAPHRAGMS: The diaphragm shall not consist of a system other than wood, metal deck, concrete, or horizontal bracing. (Tier 2: Sec. 4.5.7.1)

Connections

C NC N/A UPLIFT AT PILE CAPS: Pile caps shall have top reinforcement and piles shall be anchored to the pile caps for Life Safety, and the pile cap reinforcement and pile anchorage shall be able to develop the tensile capacity of the piles for Immediate Occupancy. (Tier 2: Sec. 4.6.3.10)

APPENDIX B

CALCULATIONS

TENNEBAUM-MANHEIM ENGINEERS
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414 Mason Street, Suite 605 San Francisco. CA 94102 415.772.9891	VETS MEMORIAL BUILDING BUILDING WEIGHTS	Job No. 008-1 Date:	Sheet No. S1 By: NT

BUILDING WEIGHTS - ROOF

TRUSS A OR B WEIGHT (APPROXIMATE)

				W
⊤ 6×6×5/8	TOTAL LENGTH	TOP CHORD =	83.82' × 48.4#/'	= 4057#
"	"	" BOTT CHORD =	82' × 48.4#/'	= 3969#
⊤ 2½×2½×¼	TOTAL LENGTH	WEBS =	84' × 8.2#/'	= 689#
⊤ 3×3×⅜	"	" =	26' × 14.4#/'	= 374#
⊤ 4×3×⅜	"	" VERTS. =	60' × 17#	= 1020#

$W_{TRUSS\ A\&B}$ = 10,109#

TRUSS / SF = 10,109#/(82)(16.5') = 7.5 PSF

BRACING TRUSS

⊤ 4×3×⅜	LENGTH	TOP & BOTT. CHORD =	16.5' × 17#/' × 2	= 561#
⊤ 3½×2½×¼	WEB	=	57.36' × 9#/'	= 516#
				1077#

2×6 @ 24" o.c. = 1.89 PSF (@ CEILING & ROOF)

ROOF WT/SF

	ROOFING =	6.5 PSF
	SHEATHING =	3 PSF
	TRUSS B =	7.5 PSF
	BT TRUSS =	6 PSF
ROOF	2×6 @ 24" o.c. =	1.9 PSF
CEILING	2×6 @ 24" o.c. =	1.9 PSF
CEIL	=	10.0 PSF
	INSUL & MISC =	2.7 PSF
	DEAD LOAD =	39.5 PSF

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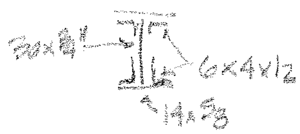
415.772.9891	BUILDING WEIGHTS	Date:	By: NT

SECOND FLOOR WEIGHT (AUDITORIUM)

WEIGHT

FLOORING 4 PSF ← ASSUMED
SHEATHING 3 PSF



2∠ 6x4x1/2 = 32.4 x 2 = 64.8 #/'
PL 30"x3/8" = 38.3 #/'
PL 2·14"x5/8" = 2x 29.8 = 59.6 #/'
162.7 #/' / 17 = 9.6 PSF

STEEL BEAMS = 9.6 PSF + 4 PSF = 11 PSF

JOISTS 2x14 @16" o.c. = 6.5 PSF

CEILING = = 10.0 PSF
MECH + MISC. = 2.5 PSF
37.0 PSF - ADD PARTION LOAD 10 PSF = 47 PSF

FIRST FLOOR WEIGHT

FLOORING = 4 PSF
SHEATHING = 3
STEEL FRAMING = 10 PSF (ASSUME)
2x14 @16" o.c. = 6.5
CEILING = 10.0
MISC. = 2.5
36 PSF + PARTITION ~10 PSF = 46 PSF

TO DETERMINE WT OF STL. FRAMING

$w = (40PSF + 100PSF)(16.5) = 2310^{\#}$
$M = 2310(26)^2/8 = 195^{'k}$
$S_{req'd} = \dfrac{195 \times 12}{21.78} = 107$ ~ W12x79
79/16.5 = 4.8 PSF x 2 = 6.8 PSF < 10 PSF OK

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415.772.9891		Date:	By: NT

BALCONY WT. (CANT READ DRAWING)

FLOORING	4 PSF
SHEATHING	3 PSF
MISC BUILT-UP FRAMING	7 PSF
2x14 @ 16" o.c.	6.5 PSF
MISC.	2.5
CEILING	10.0
	33.0 PSF + PARTITION

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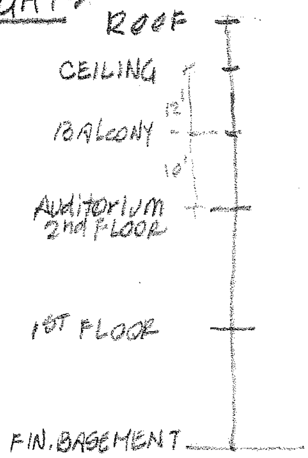
414 Mason Street, Suite 605 San Francisco. CA 94102	VETERAN'S MEMORIAL BLDG.	Job No. 008-1	Sheet No. S4
415.772.9891	WALL WEIGHTS	Date:	By: NT

WEIGHTS



WALLS NS (FOR EW DIRECTION)

$WT_{ROOF} = (75 psf)(86.5' \times 2)(14.75')$
$= (12975 plf)(14.75') = 191.4^k$

$WT_{2ND\,FLOOR} = (12975 plf)(19.5') = 253.0^k$

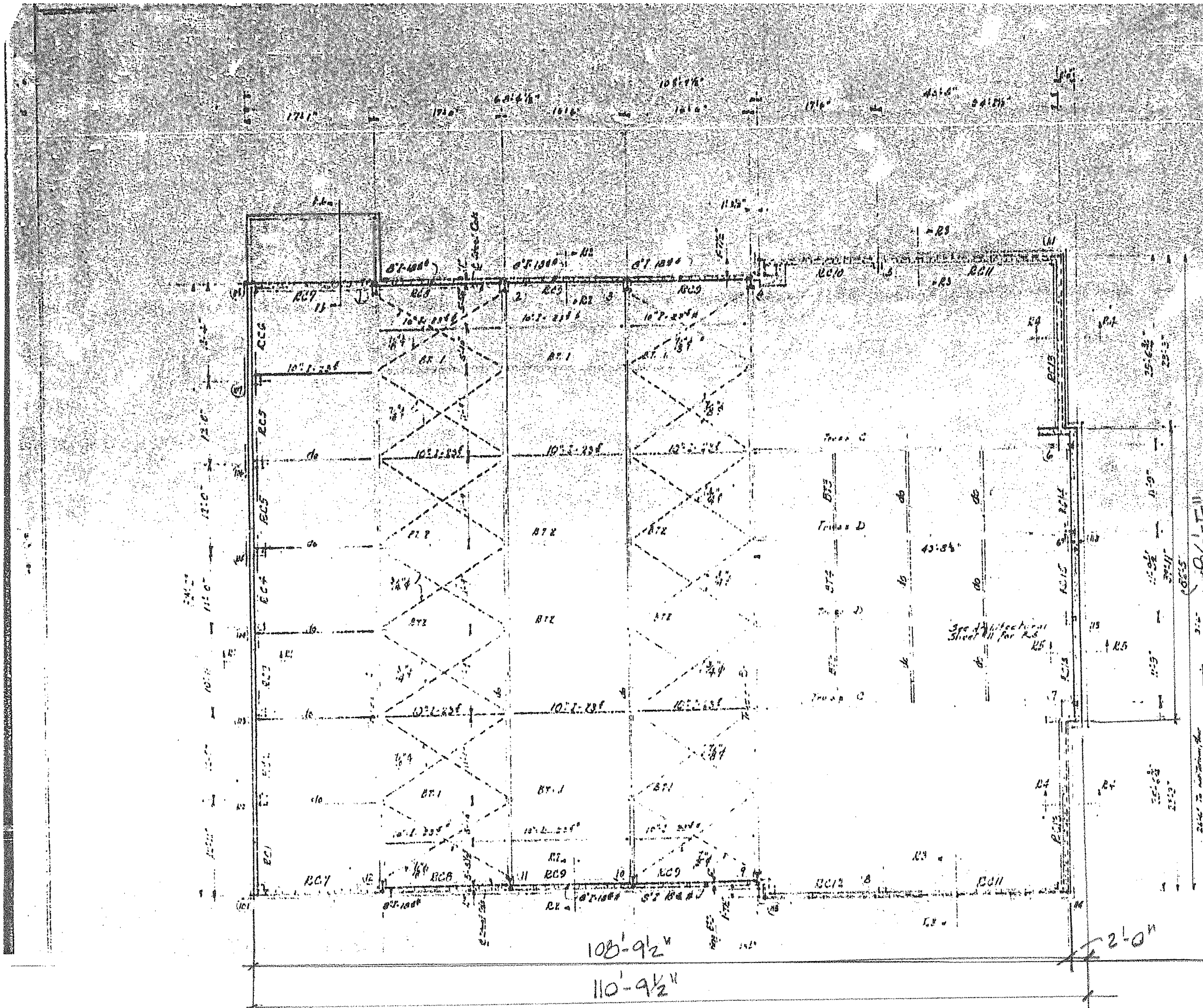
$WT_{1ST\,FLOOR} = (12975 plf)(14') = 181.7^k$

WALLS E-W (FOR NS DIRECTION)

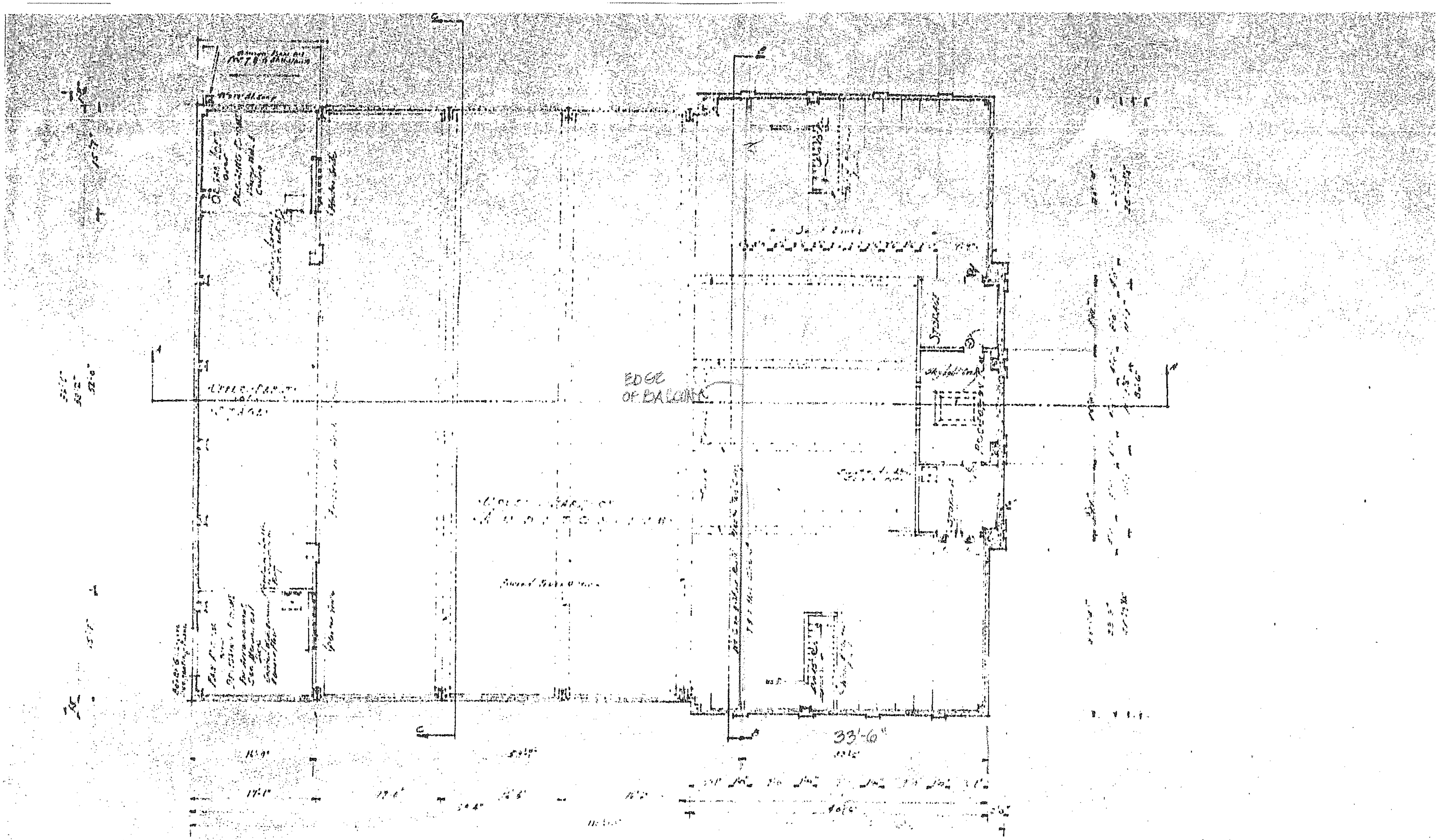
$WT_{ROOF} = (75 psf)(112.75')(2)(14.75)$
$= (16912.5 plf)(14.75') = 249.5^k$

$WT_{2ND\,FLR} = (16912.5 plf)(19.5') = 329.8^k$

$WT_{1ST\,FLR} = (16912.5 plf)(14') = 236.8^k$



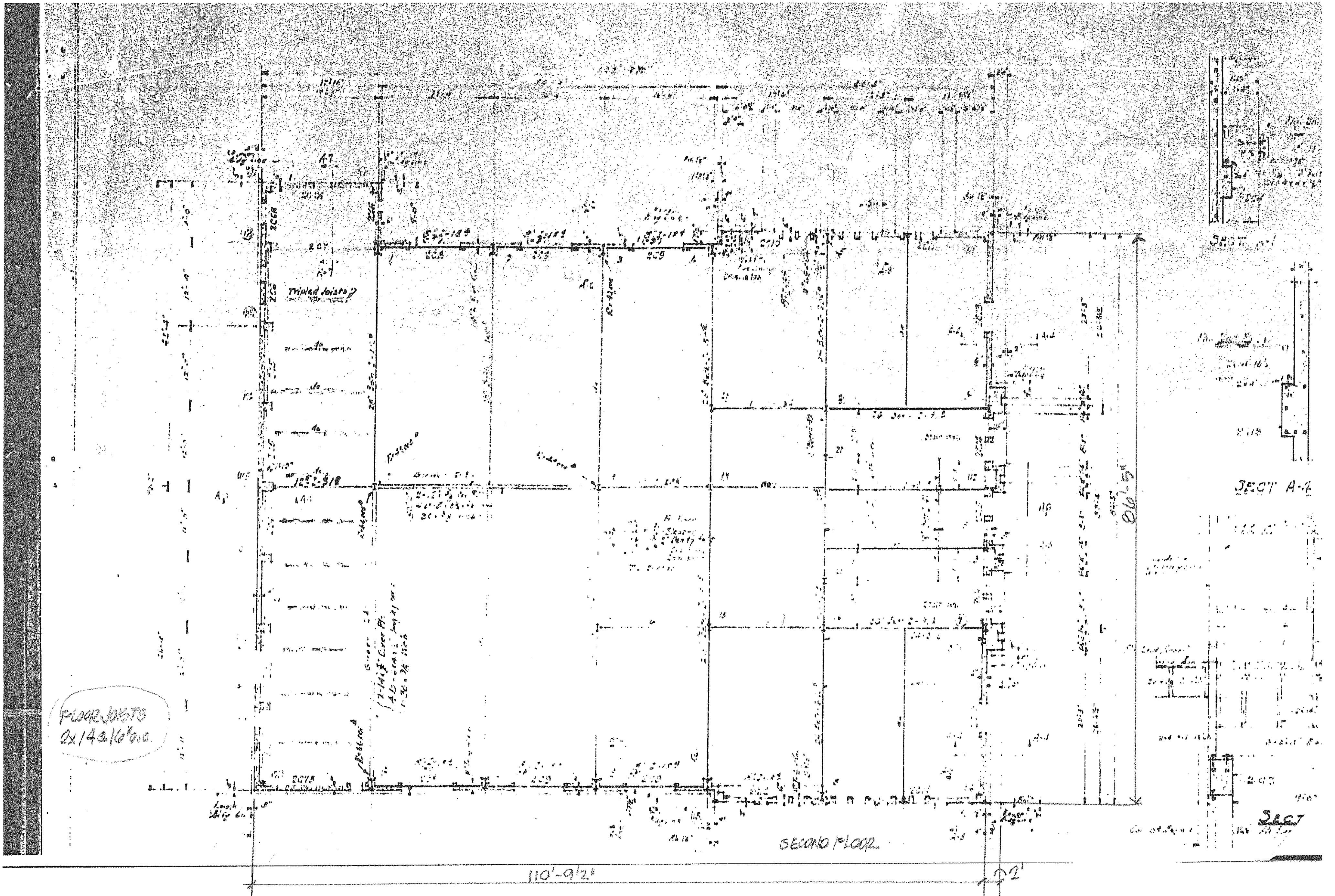
$$\text{ROOF WT}_{DL} = (39.5\,\text{PSF})(112.8)(86.42') + (39\,\text{PSF})(17')(9') = 391^{K}$$



BALCONY FLOOR PLAN

BALCONY WT_{DL} = (43 PSF)(33.5)(86.5) = 125^K

56



SECOND FLOOR WT$_{DL}$ = (40PSF)[9747+154 SF]
= 475.3^{k}

S

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SEISMIC

2007 CHBC - CHAPTER 8-7 SECT. 8-706.1

"...SEISMIC LOADS NEED NOT EXCEED .75 TIMES THE SEISMIC FORCES PRESCRIBED BY THE 1995 EDITION OF THE CALIFORNIA BUILDING CODE"

$V = \frac{ZIC}{R_N} W$ $\quad Z = .4 \quad I = 1.0 \quad C = 2.75$ max.
$R_W = 8$ (BUILDING FRAME SYSTEM)

$V = \frac{.4(1)(2.75)}{8} W = .1375W$

$.75V = .75(.1375W) = .103W$

NS DIRECTION

$W_{ROOF} = 391^k + 191^k = 582^k$
$W_{BALCONY} = 125^k$
$W_{2nd} = 475 + 253 = 728^k$
$W_1 = 455 + 182 = 637^k$
2072^k

EW DIRECTION

$W_{ROOF} = 391^k + 250 = 641^k$
$W_{BALCONY} = 125^k$
$W_2 = 475^k + 330^k = 805^k$
$W_1 = 455^k + 237^k = 692^k$
2263^k

$V_{NS} = .103(2072^k) = 213.4^k$

$V_{EW} = .103(2263^k) = 233.1^k$ = BASE SHEAR

CHECK E-W DIRECTION

LEVEL	h	WT	Wh	Wh/ΣWh	F	F_V
ROOF	56'	641^k	35896	.50	116.6^k	116.6
BALCONY	38'	125^k	4750	.077	16.3^k	132.9
2nd	28'	805^k	22,540	.32	74.6^k	207.5
1st	11'	692^k	7,612	.11	25.6^k	233.1
		2263^k	70,798		233.1^k	

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VETS MEMORIAL BLDG	Job No. 0004	Sheet No. S9
	Date:	By: NT

SEISMIC CONT.

CHECK N-S DIRECTION $V_{NS} = 213.4^k$

LEVEL	h	WT	Wh	Wh/ΣWh	F	F_V
ROOF	56'	582	32,592	.50	106.7	106.7
BALCONY	38'	125	4,750	.07	14.9	121.6
2nd	28'	728	20,384	.32	68.2	189.8
1st	11'	637	7,007	.11	23.5	213.3
			64,733			

DIAPHRAGM - USE $R_W = 6$ FOR WOOD DIAPHRAGM $F_{px} = \frac{\sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} w_i} w_{px}$

E-W DIRECTION

LEVEL	h	W	F(8/6)	F_V	F_{diaph}
ROOF	56'	641	155.5	155.5	155.5
BALCONY	38'	125	21.7	177.2	29.0
2nd	28'	805	99.5	276.7	141.8
1st	11'	692	34.2	310.9	95.1
		2263			

N-S DIRECTION

LEVEL	h	W	F(8/6)	F_V	F_{diaph}
ROOF	56'	582	142.3	142.3	142.3
BALCONY	38'	125	19.9	162.2	28.7
2nd	28'	728	90.9	253.1	128.4
1st	11'	637	31.3	284.4	80.1
		2263			

* F_{diaph} NEED NOT EXCEED $.75 Z I w_{px} = .75(.4)(1)(w_{px})$

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VETERANS MEMORIAL	Job No.	Sheet No. 59 r
CONCRETE WALLS	Date: 3-2013	By NT

CONCRETE STRENGTH (DETERMINED FROM CORING SAMPLES)

F_c = MEAN OF DATA - 1 STANDARD DEVIATION ASCE 41-06

BASEMENT CONCRETE STRENGTH

MATRISCOPE - SIX CORE SAMPLES (EXCLUDED HIGHEST STRENGTH SAMPLE)

MEAN OF DATA = 3023.4 PSI

$$S = \sqrt{\frac{1}{N-1}\sum_{i=1}^{N}(X_i - \bar{X})^2} = 379$$

STANDARD DEVIATION

$f'_{c_{\text{BASEMENT}}} = 3023.4 - 379 = \underline{2644 \text{ PSI}}$

FIRST FLOOR CONCRETE STRENGTH

MATRISCOPE - SIX CORE SAMPLES

MEAN OF DATA = 2324 PSI

$S_{\text{STD DEVIATION}} = 872$

$f'_{c_{\text{1st FLOOR}}} = 2324 - 872 = \underline{1452 \text{ PSI}}$

SECOND FLOOR CONCRETE STRENGTH

MATRISCOPE - SIX CORE SAMPLES

MEAN OF DATA = 2537 PSI

$S = 543$

$f'_{c_{\text{SECOND FLOOR}}} = 2537 - 543 = \underline{1994 \text{ PSI}}$

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VETERAN'S MEMORIAL | Job No. | Sheet No. 510 r
Date: 3, 2013 | By NT

BASEMENT CONC. WALL SHEAR CHECK

ALLOWABLE CONC. SHEAR STRESS: (CBC 26.1) HistoricCode

$V_c = 1.1\sqrt{f'_c} = 1.1\sqrt{2644} = 56.5$ psi $\rightarrow (.056)(\underset{t}{6''})(\underset{(1/FT)}{12''}) = 4^{K/FT}$
BASEMENT

IF WALL IS REINFORCED W/ 3/8" SQ BARS @ 24" o.c.

$V_s = (33\text{KSI}/2)(.38)^2 = 2.4^K/2' = 1.2^{K/FT}$

$V_c + V_s = 4^{K/FT} + 1.2^{K/FT} = 5.2^{K/FT}$

$V_{EW} = 116.6^{K/WALL}$

$V_{EW} = 116.6^K/52' = 2.24^{K/FT} < 5.2^{K/FT}$ OK IF 3/8" SQ. REINF @ 24" o.c.

IF NO REINF IN CONC. WALLS ADD FIBERWRAP

2 LAYERS FIBRWRAP GOOD FOR $2.3^{K/FT}$
ASSUME 40' LENGTH

$4(40') + 2.3(40) = 252^K > 116.6^K$ OK

IF NO REINF ADD 2LAYERS FIBRWRAP
(40' LENGTH OF WALL)

FIRST FLOOR CONC. WALL SHEAR CHECK

$V_c = 1.1\sqrt{f'_c} = 1.1\sqrt{1452} = 41.9$ psi
$(.0419)(6'')(12'') = 3^{K/FT}$

IF WALL IS REINFORCED W/ 3/8" SQ. BARS @ 24" o.c.

$V_s = (33\text{KSI}/2)(.38)^2 = 2.4/2 = 1.2^{K/FT}$

$V_c + V_s = 3^{K/FT} + 1.2^{K/FT} = 4.2^{K/FT}$

V_{EW} = /WALL 116.6^K $\ell_{WALL} = 40'$

$V_{EW} = 116/40 = 2.9^{K/FT} < 4.2^{K/FT}$ OK IF 3/8" SQ REINF @ 24" o.c.

IF NO REINF IN CONC. WALLS ADD FIBRWRAP
2 LAYERS FIBERWRAP GOOD FOR $2.3^{K/FT}$
$3(40) + (2.3)(40) = 212 > 116.6^K$

IF NO REINF. ADD 2LAYERS OF FIBRWRAP - 40'

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VETERAN'S MEMORIAL	Job No.	Sheet No. S11 r
CONC WALL ANCHORAGE	Date: 3.2013	By NT

ANCHORAGE OF CONCRETE WALLS - OUT OF PLANE

$F_p = 0.8\, S_{DS} W_p$

$W_p = 16' \times .5' \times 150$ pcf $= 1.2$ KLF

$S_{DS} = 2/3\, F_a S_S = 2/3\,(1.5) = 1.0$

$F_p = 0.8\,(1)(\;W_p\;) = .96$ K/ft $\times 4' = 3.8^K$

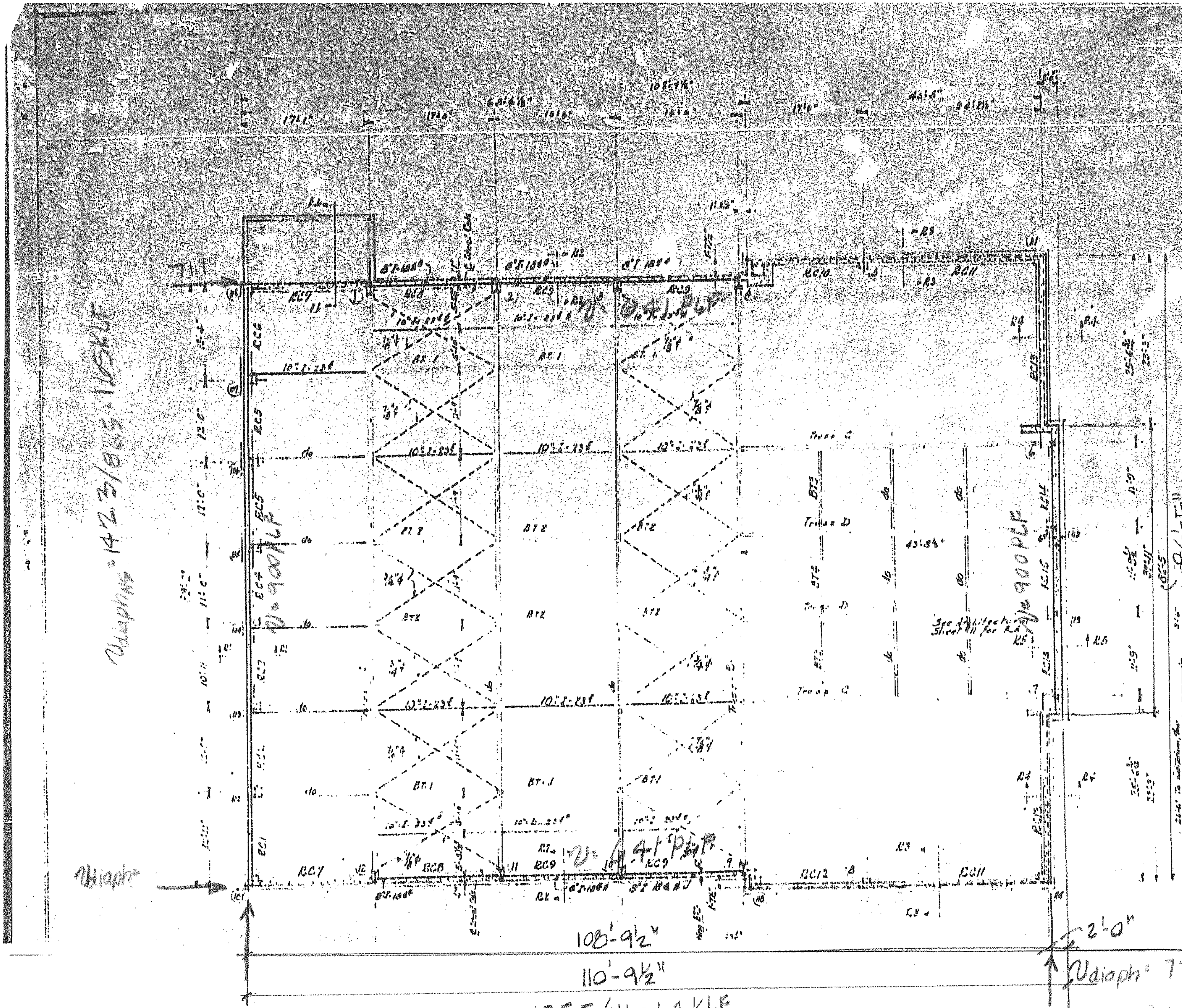
$F_{p\,min} = 400\, S_{DS} = 1.6^K$

USE SIMPSON SET-XP RE500

$V_{allow} = 300/1.4 = 214$ pl

(SEE S-15)

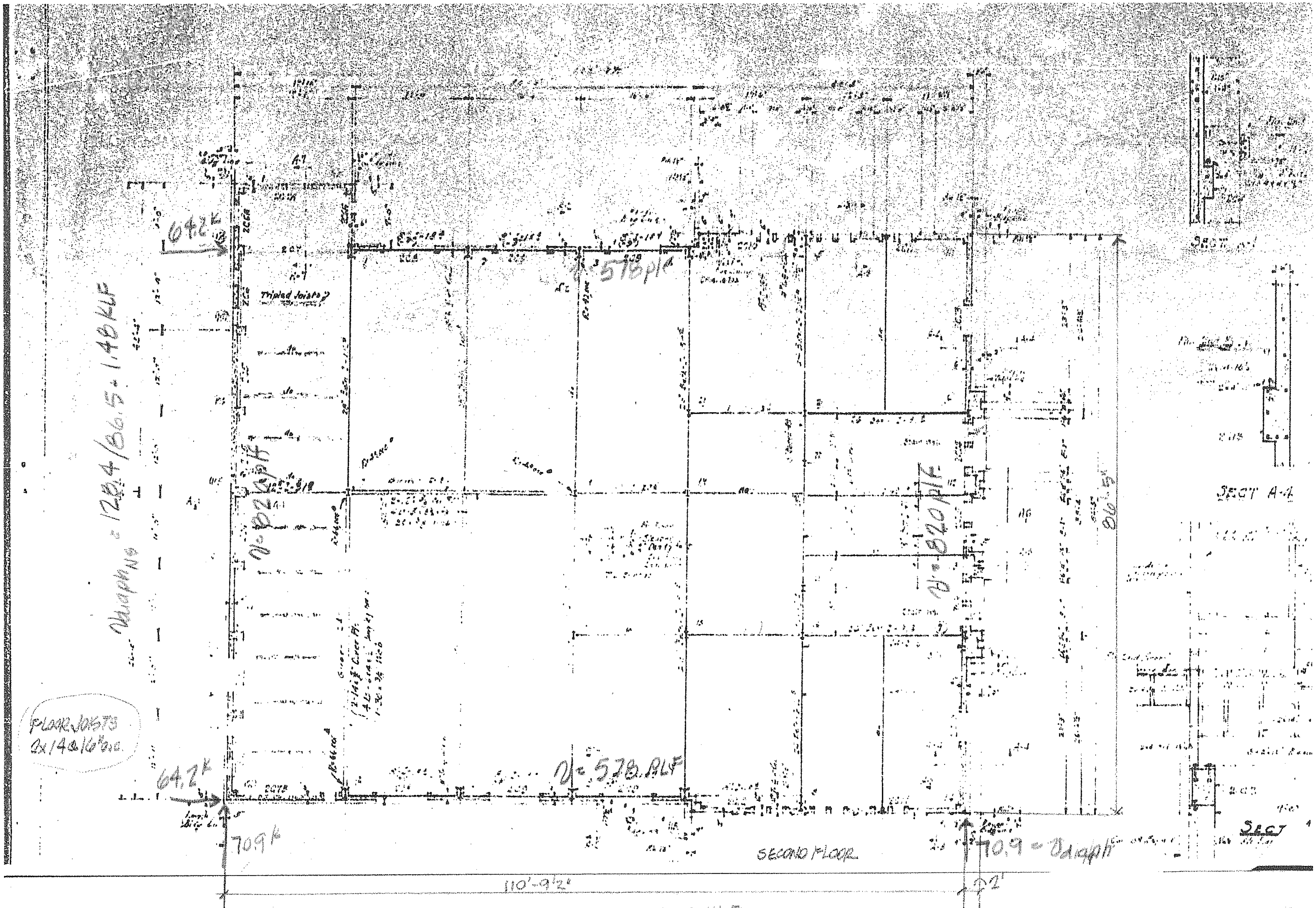
: REINF DIAPHRAGM WHERE SHEAR > 214



$V_{diaph} = 155.5/111 = 1.4$ KLF

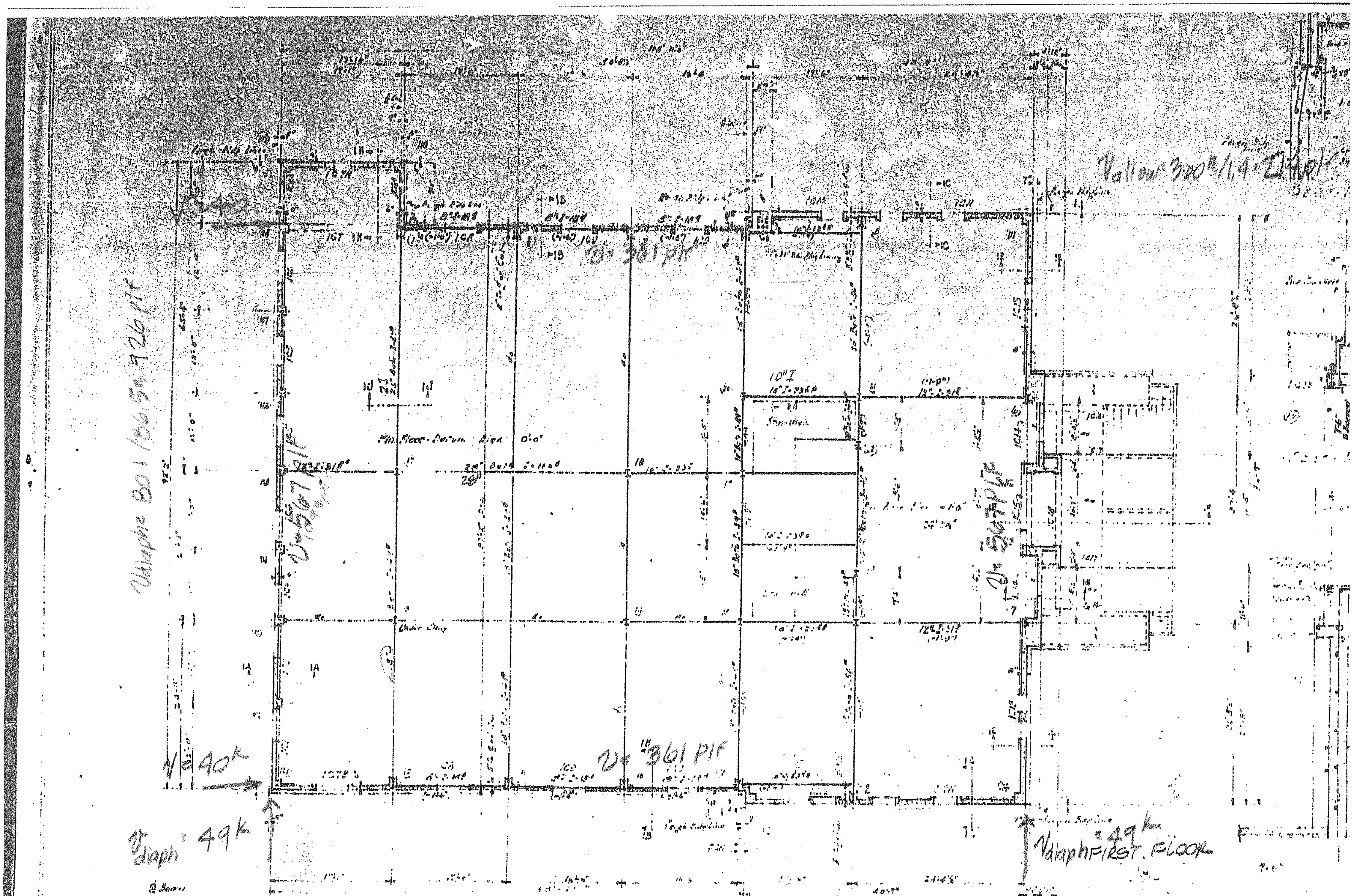
ROOF DIAPHRAGM LOADING

$$\text{ROOF WT}_{DL} = (39.5 PSF)(112.8)(86.42') + (39 PSF)(17')(9') = 391^k$$



SECOND FLOOR DIAPHRAGH LOADING

SECOND FLOOR WT_{DL} = (48 PSF)[9747 + 154 SF] = 475.3k



Vdiaph = 98.1/111 = 884 PLF

FIRST FLOOR WT_{DL} = (46 PSF) [9747 + 154 SF]

= 455.4k

FIRST FLOOR DIAPHRAGM LOADING

strengthened where necessary. Fixed conditions or midheight lateral loads on cast iron columns that could cause failure should be taken into account. Existing structural wrought, forged steel or grey iron may be assigned the maximum working stress prevalent at the time of original construction.

SECTION 8-810 HOLLOW CLAY TILE

The historical performance of hollow clay tile in past earthquakes shall be carefully considered in evaluating walls of hollow clay tile construction. Hollow clay tile bearing walls shall be evaluated and strengthened as appropriate for lateral loads and their ability to maintain support of gravity loads. Suitable protective measures shall be provided to prevent blockage of exit stairways, stairway enclosures, exit ways and public ways as a result of an earthquake.

SECTION 8-811 VENEERS

8-811.1 Terra cotta and stone. Terra cotta, cast stone and natural stone veneers shall be investigated for the presence of suitable anchorage. Steel anchors shall be investigated for deterioration or corrosion. New or supplemental anchorage shall be provided as appropriate.

8-811.2 Anchorage. Brick veneer with mechanical anchorage at spacings greater than required by the regular code may remain, provided the anchorages have not corroded. Nail strength in withdrawal in wood sheathing may be utilized to its capacity in accordance with code values.

SECTION 8-812 GLASS AND GLAZING

8-812.1 Glazing subject to human impact. Historical glazing material located in areas subject to human impact may be approved subject to the concurrence of the enforcing agency when alternative protective measures are provided. These measures may include, but not be limited to, additional glazing panels, protective film, protective guards or systems, and devices or signs which would provide adequate public safety.

8-812.2 Glazing in fire-rated systems. See Section 8-402.3.

**TABLE8-8A
ALLOWABLE VALUES FOR EXISTING MATERIALS**

EXISTING MATERIALS OR CONFIGURATIONS OF MATERIALS[1]	ALLOWABLE VALUES x14.594 for N/m
1. Horizontal diaphragms[2]	
1.1 Roofs with straight sheathing and roofing applied directly to the sheathing	100 lbs per foot for seismic shear
1.2 Roofs with diagonal sheathing and roofing applied directly to the sheathing	250 lbs per foot for seismic shear
1.3 Floors with straight tongue-and-groove sheathing	100 lbs per foot for seismic shear
1.4 Floors with straight sheathing and finished wood flooring with board edges offset or perpendicular	500 lbs per foot for seismic shear
1.5 Floors with diagonal sheathing and finished	600 lbs per foot for seismic shear
2. Crosswalls[2,3]	
2.1 Plaster on wood or metal lath	Per side: 200 lbs per foot for seismic shear
2.2 Plaster on gypsum lath	175 lbs per foot for seismic shear
2.3 Gypsum wallboard, unblocked edges	75 lbs per foot for seismic shear
2.4 Gypsum wallboard, blocked edges	125 lbs per foot for seismic shear
Existing footings, wood framing, structural steel and reinforced steel	
3.1 Plain concrete footings	f'_c = 1,500 psi (10.34 MPa) unless otherwise shown by tests[4]
3.2 Douglas fir wood	Allowable stress same as D.F. No. 1[4]
3.3 Reinforcing steel	f_t = 18,000 lbs per square inch (124.1 N/mm^2) maximum
3.4 Structural steel	f_t = 200,00 lbs per square inch (137.9 N/mm^2) maximum

[1]Material must be sound and in good condition.
[2]A one-third increase in allowable stress is not allowed.
[3]Shear values of these materials may be combined, except the total combined value shall not exceed 300 pounds per foot (4380 N/m).
[4]Stresses given may be increased for combinations of loads as specified in the regular code.

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414 Mason Street, Suite 605 San Francisco. CA 94102 415.772.9891	VETERAN'S MEMORIAL BLDG.	Job No. 008-1	Sheet No. S16
		Date: 2-08	By: NT

CHECK JOISTS FOR ADDITIONAL SOLAR PANEL WT.

$WT_{DL} = 9.5 + 2.7 = 12.2$ PSF $+ 1.9 = 14.1$ PSF

$W_{LL} = 30$ PSF

$S = \frac{(2)(6)^2}{6} = 12 \text{ IN}^3$

$I = 36 \text{ IN}^4$

$w = (44.1 \text{ PSF})(2') = 88.2$ PLF

$M = (88.2 \text{ PLF})(12' max)^2/8 = 1588'^{\#}$

$f_b = 1588 \times 12/12 = 1588$ HIGH

CHECK 25psf LL

$w = (39.1)(2') = 78.2$

$M = (78.2 \text{ PLF})(12' max)^2/8 = 1407.6'^{\#}$

$f_b = 1407 \times 12/12 = 1407 / 1.25 = 1126$ say OK

CHECK SHEAR

$w = 39.1(2') = 78.2$

$R = 78.2 \times 12/2 = 469^{\#}$

$f_v = 1.5(469)/(2)(6) = 58.6$ OK

$\Delta = \frac{5(78.2)(11.75)^4(1728)}{384(36)(1,600,000)} = .58''$ $l/240$ OK

APPENDIX C

PHOTOS

Front Entrance - South Elevation (March 2008)

West Elevation at parking lot (March 2008)

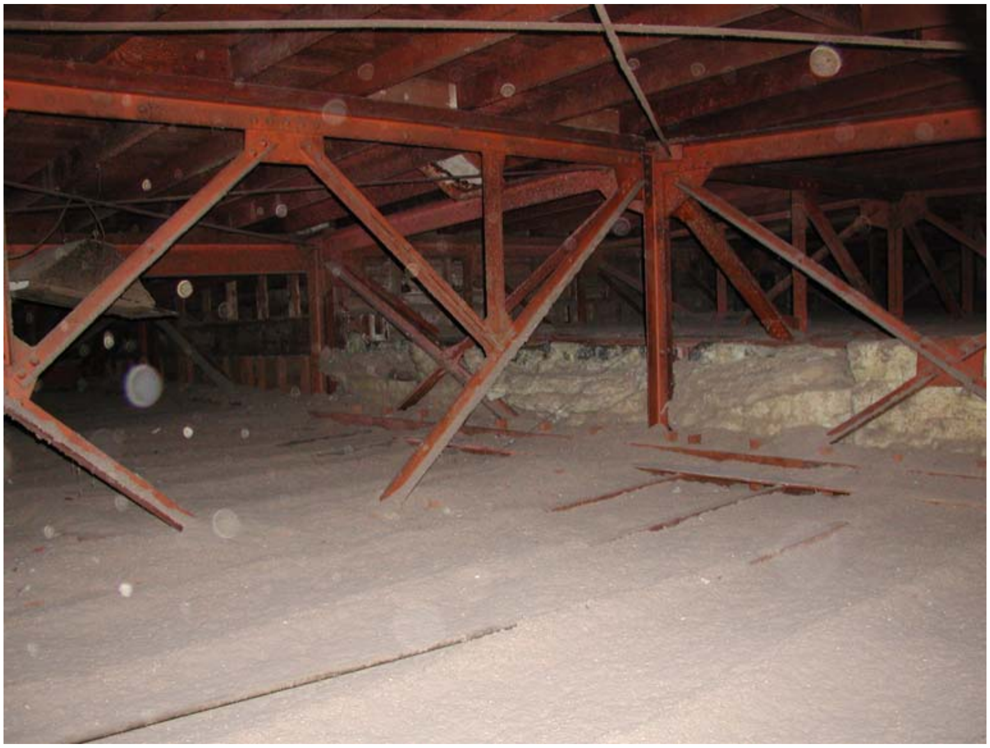
North Elevation – Back of Building (March 2008)

East Elevation (March 2008)

Concrete Spalling at Façade

Cracking at Ornamentation

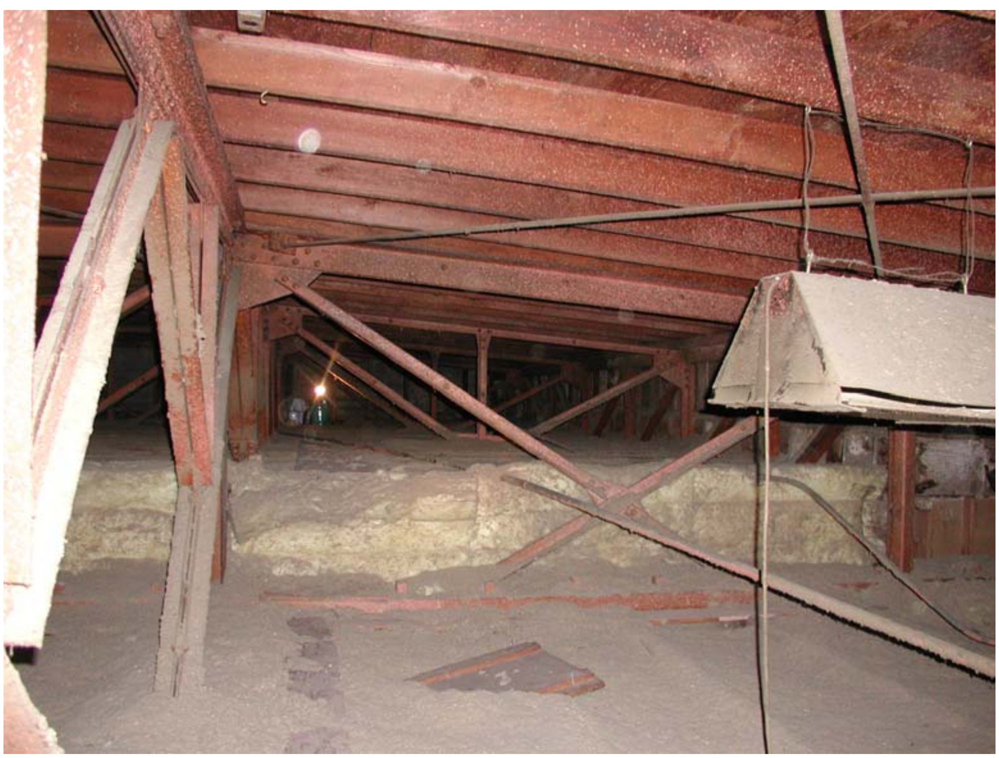
Concrete Spalling at Header
(March 2008)



Roof trusses, wood rafters, straight sheathing and ceiling joists

Roof trusses, rafters and straight sheathing
(March 2008)

Riveted connections at roof trusses



Roof framing
(March 2008)

Typical floor framing supported on steel beam and straight sheathing above joists

Typical floor framing
(March 2008)

Typical floor framing and steel beam

First floor joists (seen from basement level)
(March 2008)

Exposed second floor framing (auditorium)

Second floor framing from access hole
(March 2008)

Second floor framing looking toward beam to pilaster at access hole

Basement
(March 2008)

Auditorium

Auditorium stage
(March 2008)

PHOTOS – INVESTIGATION PHASE March 2012 and March 2013

Water Damage at 2nd Floor

Water Damage at 2nd Floor

PHOTOS – APRIL 10, 2013

Exposed reinforcing on North Façade

Exposed reinforcing on North Façade

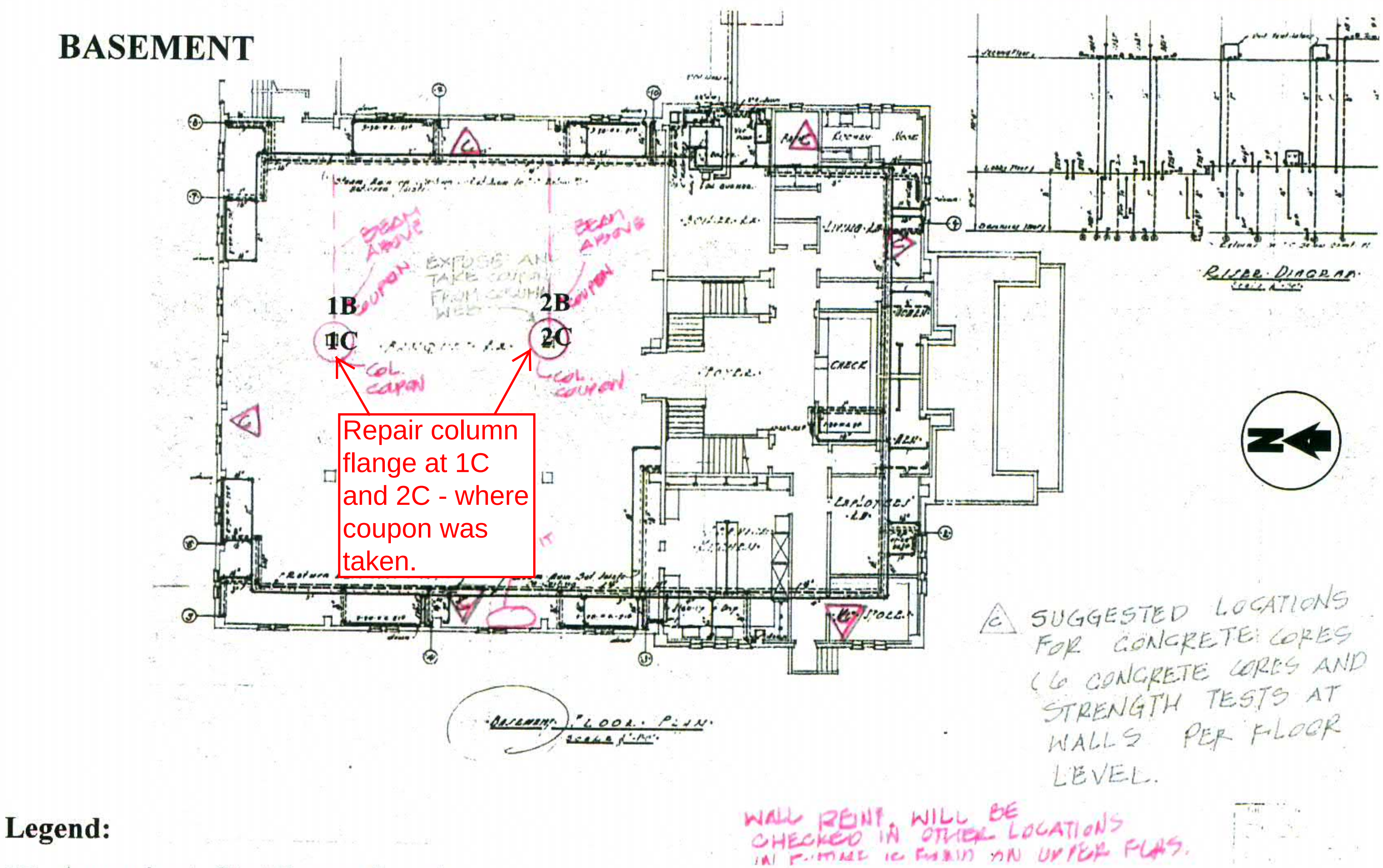
Exposed reinforcing on north façade of east stair tower

Column with coupon cut from flange
(Repair required)

Photos 4-10-13

APPENDIX D

COLUMN REPAIR PLAN LOCATION



Legend:

1C Approximate Steel Coupon Location

STEEL COUPON LOCATIONS MAP

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Project No.:	5456
Project Name:	Vallejo VA Memorial Building
Location:	444 Alabama Street, Vallejo, California
Date:	2/8/2013

Plate 1A