APPENDIX E:
Structural Seismic Risk Assessment/Feasibility Study

July 2, 2013

Mr. Alan Burks
Environ Architecture, Inc.
100 Oceangate, Suite P-200
Long Beach, CA 90802

Re: Structural Seismic Risk Assessment / Feasibility Study
East Division PD Sub-Station
3800 East Willow St.
Long Beach, CA 90815
MHP JN: 130232

Dear Mr. Burks:

At your request, MHP, Inc. conducted a seismic evaluation of the existing Schroeder Hall U.S. Army Reserve Center located at 3800 East Willow St. in Long Beach, California. The structure is to be converted from an Army office and classroom building to the new East Division PD Sub-Station for the Long Beach Police Department. The purpose of this review is to evaluate the expected seismic performance of the existing structure, determine the scope of strengthening work that is required to bring the existing structure into conformance with current code requirements for existing buildings, comment on the impact of proposed architectural improvements to the existing structure, and to provide schematic structural options for both required seismic retrofit and structural support of the proposed improvements.

Our review included a detailed analysis of the structural adequacy of the lateral force resisting systems of the building utilizing a three-dimensional computer model of the structure based on available record documents. Assessment of the structural performance results was performed in accordance with ASCE 41-06 and Chapter 34 Existing Structures of the 2010 California Building Code. No site specific geotechnical report was available at the time of our review. Our review did not include evaluation of any non-structural aspects of the design (such as existing non-structural partitions, ADA, etc.).

Additionally, a site visit was performed to verify existing conditions where possible. Terry Fernandez, S.E., and Kyle White, P.E., from our office completed the field investigation on May 9, 2013. Our visual investigation was limited to exposed surfaces accessible from interior spaces and the exterior of the structure at ground level.

Building Description

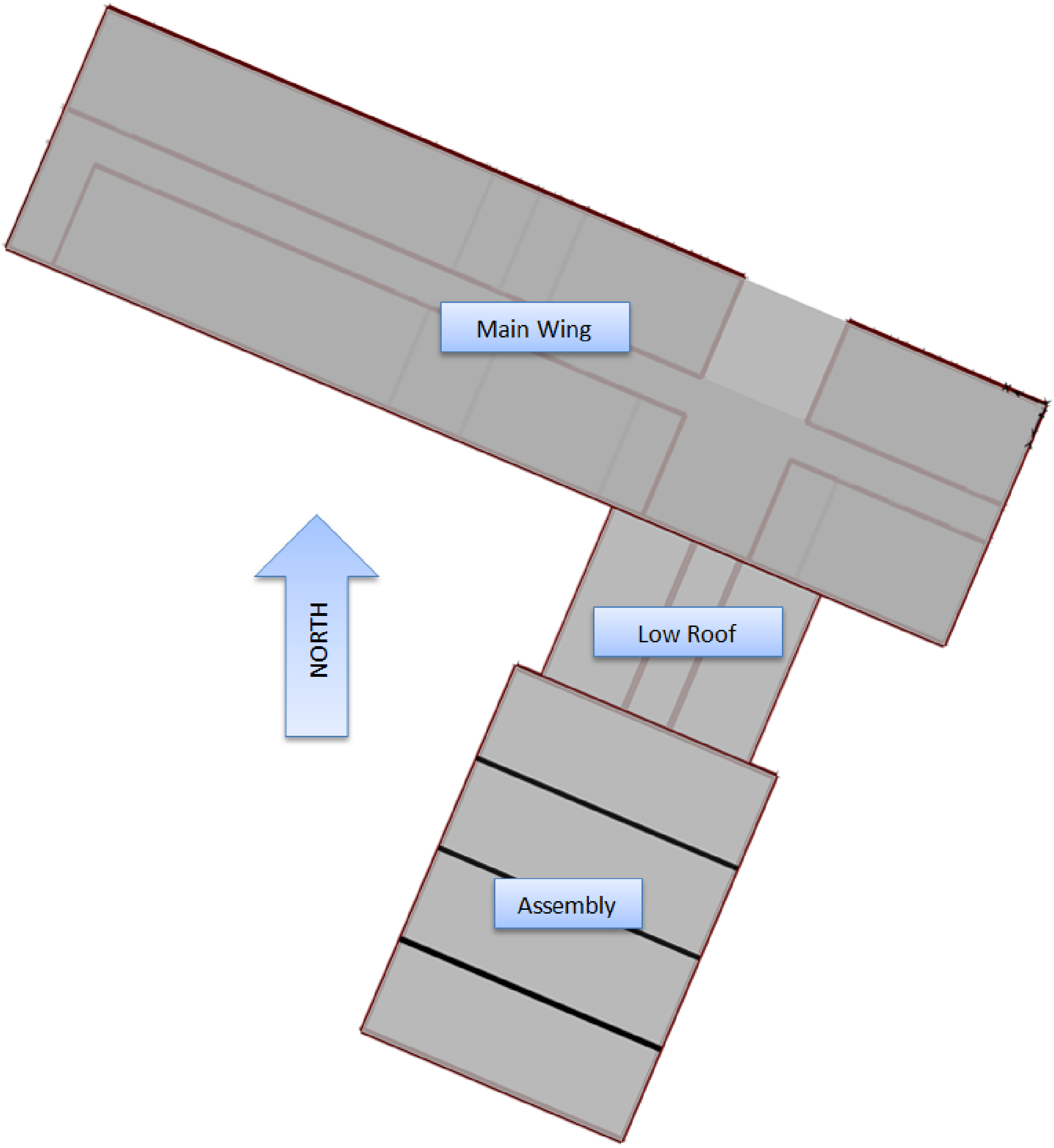
The existing Schroeder Hall Building, originally constructed in 1960, consists of three main components arranged in an L-shaped configuration with a total square footage of approximately 55,200 square feet. The two-story main wing is located at the north end. A high-roof assembly space which is connected to the main wing via a single-story low roof portion which extends to the south. The assembly space and the connecting low roof area are isolated from the main wing with respect to lateral loads in the north-south direction, but connected with respect to the east-west direction.

The structural framing of the 35,000 square foot main two-story wing incorporates bare metal deck at the roof diaphragm supported by structural steel open web joists which span between reinforced double-wythe brick masonry bearing walls. Second floor framing consists of a 3” thick reinforced concrete slab diaphragm supported by steel open web joists spanning between reinforced double-wythe brick masonry or one-story concrete masonry unit (CMU) bearing walls. Horizontal wind and seismic forces are resisted by the bare metal deck diaphragm at the roof and the concrete slab diaphragm at the second floor. The second floor diaphragm is stiffened in some regions by the addition of steel tie rods. These diaphragms behave as deep beams spanning between the masonry walls. Loads are ultimately transmitted to the soil at the foundation by conventional shallow concrete continuous footings.

The approximately 14,700 square foot assembly space to the south consists of a bare metal deck roof diaphragm supported by steel wide flange beams spanning between tapered steel girders and the masonry walls. The tapered steel girders span between reinforced brick masonry pilasters at the east and west walls of the space. Horizontal wind and seismic forces are resisted by the roof deck spanning between the masonry walls at the north and south faces of the space, and spanning between the masonry pilasters (behaving as cantilevered elements to either side of large window openings) at the east and west faces. The pilasters extend below the windows to the foundation and become integral with a high solid reinforced brick masonry wall. The roof diaphragm is stiffened by tie rod bracing in the East-west direction. Gravity and horizontal loads are ultimately transmitted to the soil at the foundation by conventional shallow concrete continuous footings.

The construction of the 5,500 square foot low roof space connecting the two-story main building to the high-roof assembly space matches that of the roof of the main wing. The north end of the low roof area is vertically supported by the two-story masonry wall of the main wing. Horizontal wind and seismic loads acting on the low roof area in the east-west direction are resisted by the masonry wall of the main wing, while the structures are isolated from one another in the north-south direction by a seismic separation which allows for 2” relative displacement between the two wings. Like the assembly and main wings, the low roof space is supported at the foundation by conventional concrete continuous footings.

It is our understanding that any required seismic upgrades are to be relegated to the interior spaces only in order to preserve the exterior due to its historical significance.



Schematic Plan View of Existing Schroeder Hall

Site Observations

A site observation was conducted on May 9, 2013. All three portions of the structure appeared to be in generally good condition. Items of note which were observed during the site walk are described below:

- Configuration and general dimensions of the structures appear to conform to those shown on the 1960 as-built plans provided for our review with the exception of the addition of two small wings of non-bearing masonry wall at main entry on the north façade.

- Diaphragm tie rod bracing at the underside of the existing second floor slab was observed as shown on the "New Tie Rod Bracing Installation" drawing dated February 8, 1960.
- Masonry and CMU walls were observed to be in excellent condition with little visible cracking or signs of distress.
- Existing second floor framing at the main wing and roof framing at the assembly space appeared to generally match the as-built documents.

Site Seismic Hazards

Design Basis Ground Motion

The design spectral response accelerations used for new design in the 2010 CBC are derived from the Maximum Considered Earthquake (MCE) event, having a return period of 2,475-years. The Seismic Site Coefficients (F_a and F_v), defined by the 2010 CBC considering the site classification and mapped spectral response accelerations (S_S and S_1), are used to adjust the mapped spectral accelerations to represent those reflective of the specific building site. The MCE spectral response accelerations (S_S and S_1), site coefficients (F_a and F_v), and design spectral response accelerations (S_{DS} and S_{D1}) which would be required by the 2010 CBC for design of a new building on the subject site are summarized in the following Table:

2010 CBC DESIGN SPECTRAL RESPONSE ACCELERATION PARAMETERS						
S_S	S_1	Site Class	F_a	F_v	S_{DS}	S_{D1}
1.734g	0.666g	D*	1.00	1.50	1.156g	0.666g

*Regional geologic information indicates that the site is underlain by Pleistocene (older) marine and non-marine terrace deposits. In the absence of site-specific information, this soil profile is consistent with Site Class D per the 2010 CBC.

Site-Specific Ground Motion

Based on published geologic reports and maps, strong ground shaking may affect the site as the result of earthquakes likely to occur on the following regional faults:

REGIONAL FAULTS			
Fault or Fault Zone	*Distance and Direction From Site*	*Recent Activity*	*Maximum Magnitude*
Newport-Inglewood (A)	1 miles SW	1933 M6.3	7.5
Palos Verdes (A)	8 miles SW	--	7.7
Puente Hills (A)	10 miles N	--	7.1
Elsinore (A)	14 miles SE	1910 M6.0	7.8
San Joaquin Hills (A)	14 miles SE	--	6.6

REGIONAL FAULTS			
Whittier (A)	14 miles NE	1987 M5.9	6.8
Upper Elysian (A)	17 miles NW	--	6.4
Hollywood-Raymond (A)	21 miles N	--	6.5
Verdugo (A)	22 miles NW	--	6.9
San Andreas (A)	46 miles NE	1857 M7.8	7.8

Active (A) or Potentially-Active (PA) Fault

Future earthquake ground motion at the site was estimated probabilistically as that level of ground motion having a probability of exceedance of 10 percent in a 50 year period (equivalent to an average return period of 475 years) and termed the Design Basis Earthquake (DBE). Site-specific ground motion, characterized in terms of peak ground acceleration and Modified Mercalli Intensity, and the estimated PGA and MMI at the site caused by previous instrumentally-recorded earthquake events likely to have affected the site are summarized in the following Table - Site-Specific Ground Motion:

SITE-SPECIFIC GROUND MOTION		
Ground Motion Level	**Peak Ground Acceleration (PGA)**	**Modified Mercalli Intensity (MMI)**
475-Year Event (DBE)	**0.36g**	**IX**
PRIOR EVENTS *		
Long Beach (1933/ M5.4/ 2)	0.35g	VIII
Torrance-Gardena (1941/ 5.4/ 6)	0.17g	VII
Whittier Narrows (1987/ M5.9/ 18)	<0.10g	VI
Northridge (1994/ M6.7/ 36)	<0.10g	VI

* Event (year/ magnitude/ epicentral distance)

Fault Rupture Hazard

California Earthquake Fault Zones (EFZs), established by the State of California under the Alquist-Priolo Earthquake Fault Zoning Act of 1973, are delineated around known surface traces of active faults. In accordance with State law, cities and counties must withhold development permits for new construction used for human occupancy and for extensive additions to or remodeling of existing structures until geologic investigations demonstrate that the proposed construction is not threatened by surface displacement from future faulting. If an active fault is found, a structure cannot be placed over the trace of the fault and must be set back from the fault (generally 50 feet). In addition, the effects of surface faulting are considered when estimating the degree of earthquake-related damage for existing structures located within the fault or drag zone.

The nearest mapped active fault is the Newport-Inglewood Fault at an approximate distance of one mile from the site. The site is not located within a California Earthquake Fault Zone (nearest EFZ is on the Newport-Inglewood Fault). Since no active or potentially active faults are known to cross the site, **the potential for ground surface rupture due to recognized faulting is considered to be low**.

Soil Liquefaction and Landslide Hazard

Seismically-induced soil settlement and liquefaction (loss of soil strength in saturated soil deposits during strong ground shaking), and slope failure (landslides or local failures triggered by earthquakes) may affect soils supporting foundations. The effects of these other earthquake hazards can lead to loss of bearing capacity and excessive settlement of foundations, resulting in increased seismic-related building damage. In California, Seismic Hazard Zone (SHZ) maps have been issued by the State Department of Conservation for some major urban areas showing areas prone to liquefaction and landslides. These maps show areas where investigations are required for liquefaction and landslide hazards before development and construction permits can be obtained.

Regional geologic maps indicate subsoils at the site consist of Pleistocene marine and non-marine terrace deposits with groundwater at a depth of greater than 30 feet below the ground surface. The site is not located within a California SHZ for liquefaction hazards (Newport Beach Quadrangle official map released April 15, 1998) and regional geologic information indicates a low potential for liquefaction. Based on this information, **the seismically-induced liquefaction potential at the site is considered low**.

The site consists of level ground with no adjacent slopes above or below the site; thus, **the potential for earthquake-induced landslide or slope stability failure is low**

Structural Assessment & Performance Objectives

The original building, constructed in 1960, is believed to have been designed under the jurisdiction of the 1958 Uniform Building Code (UBC). The configuration and construction of the building appears to be typical for buildings of similar age and construction. Since the 1960's, structural engineers have developed a significantly greater understanding of both seismic ground motions and building response to those ground motions. The current edition of the building code has significantly improved wind and seismic design standards, including a considerably greater lateral design force and strict standards for detailing and strength calculation of lateral force resisting elements, than those used for the project building. Therefore, compared to buildings built per 1976 or newer UBC and IBC standards, the subject building has significantly higher seismic risk. If this structure were to be designed per current code standards, the design lateral force for seismic considerations would be greater than twice that which would have been required at the time of original construction.

However, while code design issues have increased design forces and detailing requirements, the engineering community has developed increased tools and ability to better predict and understand existing building behavior for buildings that may not meet the present code. This increased knowledge base has resulted in guidelines and code basis for addressing existing structures.

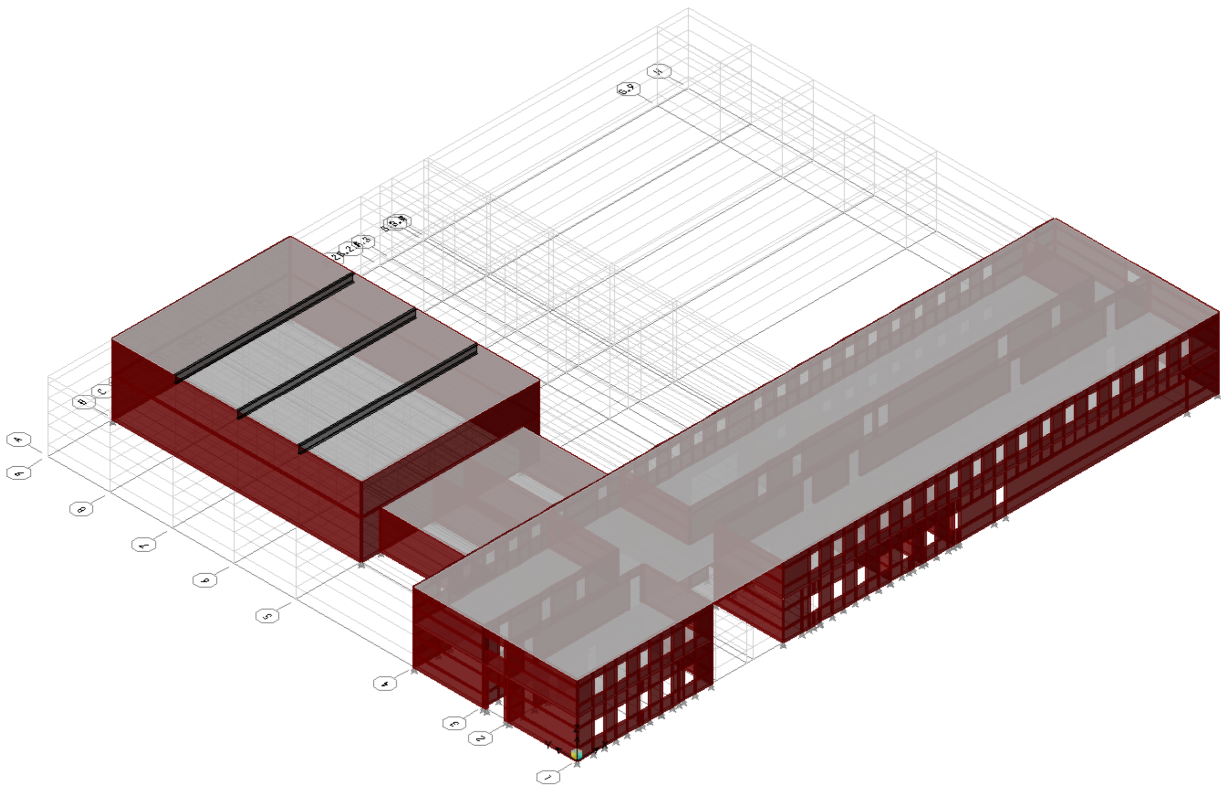
The existing structure was modeled and assessed based on the requirements of ASCE 41-06 as allowed by the 2010 California Building Code, Chapter 34 considering two separate structural performance objective levels. The two performance objectives are as follows:

1. Life Safety Structural Performance Level (S-3) for the BSE-1 Earthquake Hazard Level (corresponding to a ground motion with an approximately 10% probability of exceedance in 50 years, or a 475-year return period).

2. Collapse Prevention Structural Performance Level (S-5) for the BSE-2 Earthquake Hazard Level (corresponding to a ground motion with an approximately 2% probability of exceedance in 50 years, or a 2475-year return period).

Structural Analysis Results

The structural analysis for the evaluation of the project building consisted of both three-dimensional computer modeling and two-dimensional analysis methods to calculate seismic force demands based on the ground motions listed above. The evaluation determined the seismic lateral force distribution to the vertical lateral force resisting elements. The calculated elastic demand (D) for each critical structural element was compared to the capacity (C) of that element, multiplied by a component modification factor to account for permissible deformations beyond yield. These modification factors are defined in ASCE 41-06 and vary depending on the desired performance objective. Acceptable element performance is denoted to be when the element capacity multiplied by the appropriate modification factor is greater than or equal to the demand gathered from the analysis.



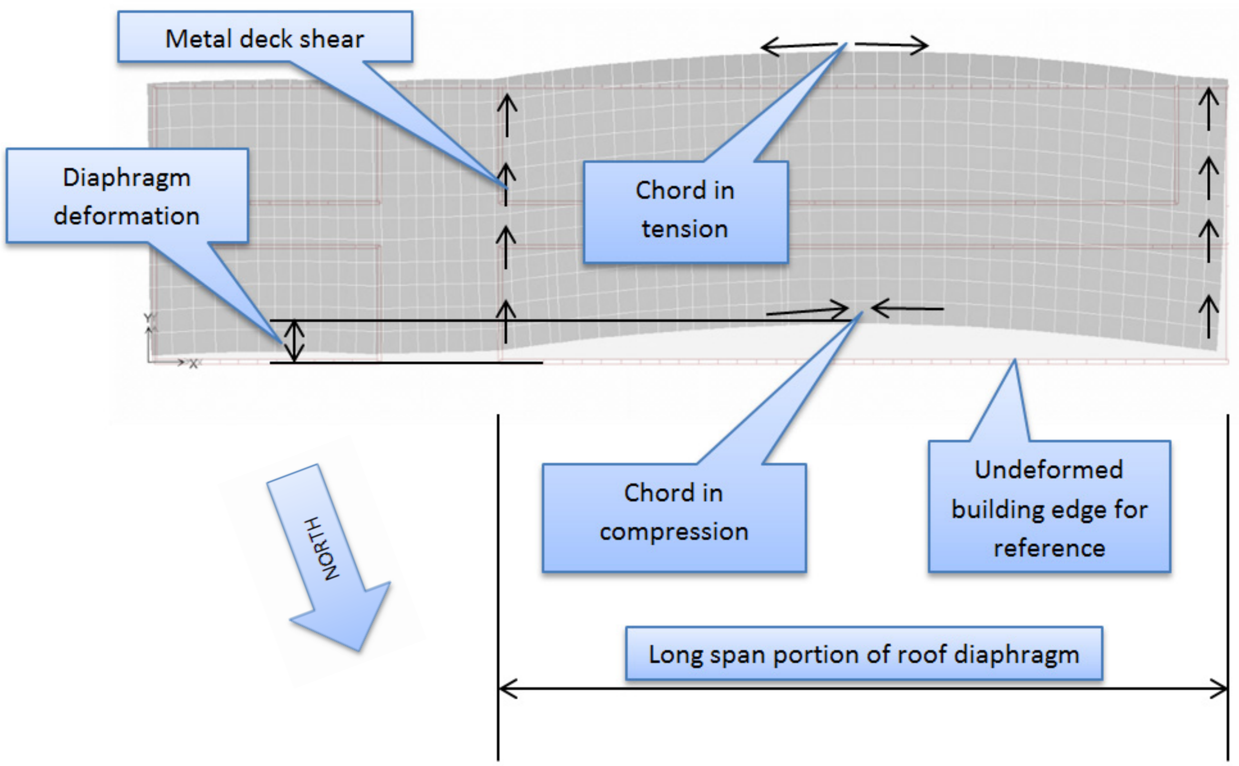
ETABS Computer Analysis Model of Existing Structure

The existing structure is in overall good condition and is expected to perform reasonably well when subjected to the design basis earthquake. However, the following concerns were noted as a result of our analysis of the building per ASCE/SEI 41-06 procedures for evaluating the structural performance per the criteria listed above:

- **Cantilever masonry pilasters at assembly:** of critical concern for structures of this era and construction type is adequate interconnection between the heavy masonry walls and the horizontal floor and roof diaphragms laterally support the out-of-plane (perpendicular to wall) seismic loads from the walls.
    - There is no adequate load path between the existing cantilevered masonry pilasters and the metal deck diaphragm for resisting out-of-plane (perpendicular to the wall) seismic forces. Existing wall anchor rods are approximately **133% overstressed** (DCR of 2.33) at the Collapse Prevention/BSE-2 level and approximately **85% overstressed** (DCR of 1.85) at the Life Safety/BSE-1 level.
    - The connection between the pilasters and the metal deck diaphragm for in-plane shear forces is as much as **321% overstressed** at the Collapse Prevention/BSE-2 level and as much as **149% overstressed** at the Life Safety/BSE-1 level.

  - Due to deficiencies at the pilaster to roof diaphragm connection in the assembly portion of the building, it is considered possible that portions of the roof framing will separate from the supporting walls during a future 475-Year seismic event, resulting in heavy localized structural damage and possible localized collapse of roof framing and adjacent walls.

- **Main wing roof diaphragm:** horizontal diaphragms resist lateral wind and seismic loads primarily through shear and flexure (bending). Shear forces at the main wing are resisted by the steel roof deck, while flexural stresses are resisted by chord reinforcing bars in the supporting masonry walls.



Main Wing Partial Roof Plan Showing Diaphragm Behavior

  - The metal roof deck at each end of the long span portion of the main wing roof diaphragm exhibits insufficient in-plane metal deck shear strength to satisfy the selected performance criteria for north-south motion. The deck is approximately **208% overstressed** at the Collapse Prevention/BSE-2 level and approximately **83% overstressed** at the Life Safety/BSE-1 level.
  - Wall reinforcing acting as diaphragm chords for north-south motion was determined to be inadequate to resist the prescribed loads. Roof diaphragm

chords are as much as **182% overstressed** at the Collapse Prevention/BSE-2 level and as much as **68% overstressed** at the Life Safety/BSE-1 level**.**

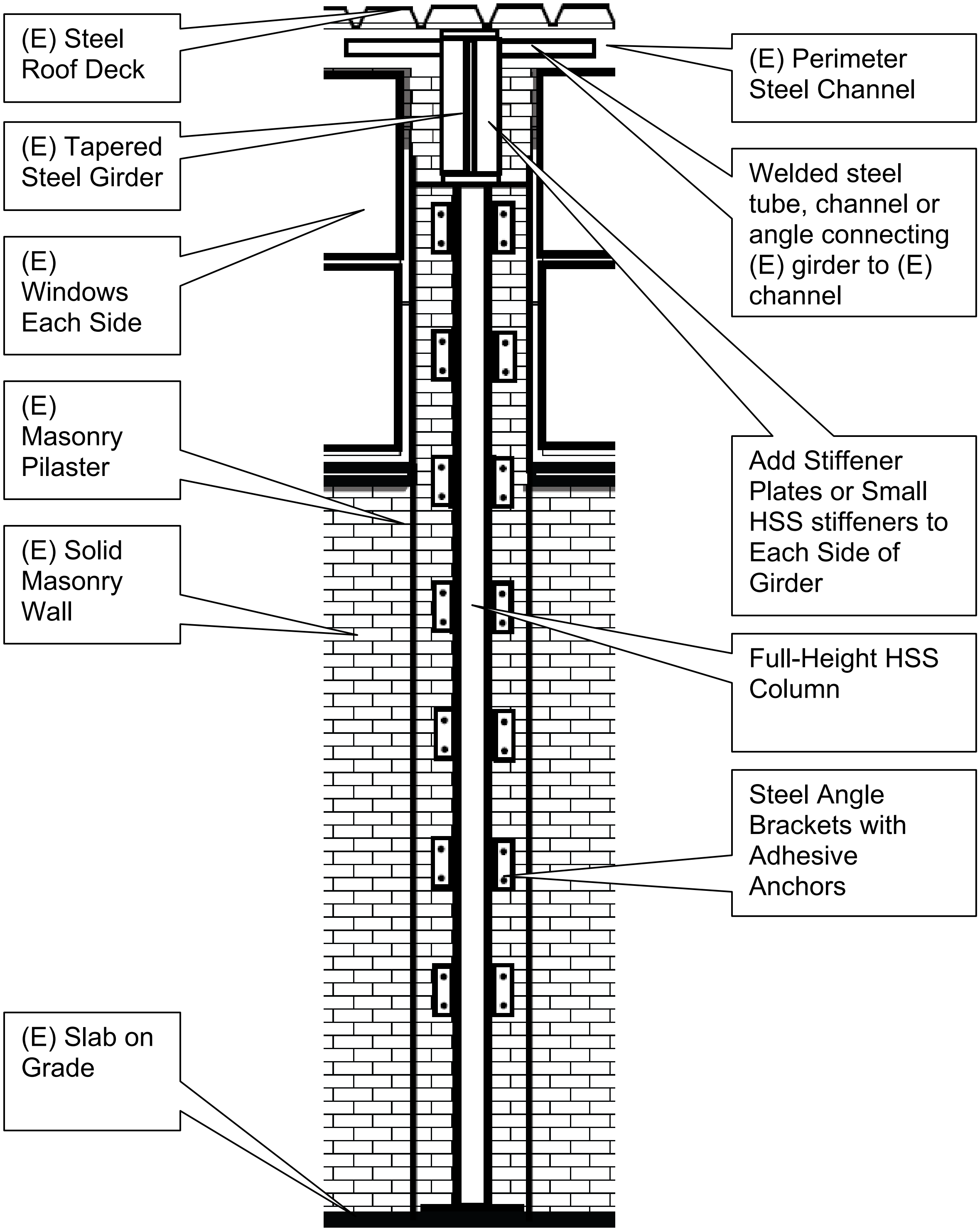
- **Main wing second floor diaphragm:** the long span portion to the west lacks sufficient diaphragm chord strength when subjected to north-south seismic motion. Chords are as much as **334% overstressed** at the Collapse Prevention/BSE-2 level and as much as **165% overstressed** at the Life Safety/BSE-1 level**.**

Conclusions and Recommendations

The existing structure is expected to generally perform adequately in a design basis event. However, based upon the results of the ASCE 41-06 analysis, the predicted structural performance of the existing structure **does not satisfy all provisions for Life Safety at BSE-1 or Collapse Prevention at BSE-2**. Seismic strengthening of multiple critical structural elements of the existing structural system would be required to reduce the life-safety hazards associated with the structures and to satisfy code compliance. Strengthening is of critical importance at the connections of the assembly masonry pilasters in order to reduce the likelihood of collapse.

Schematic recommendations for strengthening measures to address the previously discussed deficiencies are summarized below:

- **Cantilever masonry pilasters at assembly:** tube steel (HSS) columns aligned at the face of each existing pilaster running from the slab on grade up to the underside of the existing tapered steel girders would address both out-of-plane wall anchorage and in-plane load path concerns. The columns would be attached to the existing pilasters via adhesive anchors. These columns would span vertically from the slab on grade below to the underside of the tapered steel girders. A schematic sketch of this condition is provided below.



Interior Elevation of Schematic Pilaster Strengthening at Assembly

- **Main wing roof diaphragm:** to address the shear strength deficiency at the roof a new rod braced diaphragm system (very similar to what was added to the second floor level in 1960) will be required to provide needed strength to the existing metal deck diaphragm. A rod-bracing strengthening scheme is beneficial to this project in that the work may be performed from the underside of the roof, preserving the exterior condition. To address the diaphragm chord member deficiency new steel chord members in the form of steel channels will be provided. These new members will be anchored to the inside face of the existing masonry walls at the north and south walls using adhesive anchorage and will be continuous for the length of the long span portions of the diaphragm.

- **Main wing second floor diaphragm:** new steel chord members in the form of steel channels will be provided to address the inadequate existing chord strength. These new members will be anchored to the inside face of the existing masonry walls at the north and south walls using adhesive anchorage and will be continuous for the length of the long span portions of the diaphragm.

Architectural Improvements & Schematic Recommendations

A number of new architectural improvements are proposed to be performed as part of the renovation of the existing building. The tenant improvement scope includes:

- **New second story in assembly space:** a new two story locker room is proposed to be added within the existing assembly space. It is anticipated that the existing building walls will be utilized for vertical and lateral support of the new second story on three sides, with the free south end of the new second floor supported vertically and laterally by a new steel braced frame, concrete shear wall, or CMU shear wall and conventional concrete foundations as required. The floor construction will be determined in concert with the project architect in order to balance performance and cost.

- **New elevator tower:** to provide access to the new second floor space and ADA accessible service to the main wing a new elevator will be added. The elevator tower will be just east of the existing low roof area, between the main wing and assembly space, and will be seismically separate from the existing structure. It is anticipated that the tower will be constructed of CMU with a wood or steel roof structure. The new tower will bear on conventional concrete foundations.

- **New walkway:** In order to connect the new elevator tower with the adjacent existing buildings, a new walkway will be provided. The new walkway will be supported by the existing low roof structure. Due to the required elevation, slopes, and additional live loads associated with this new walkway, it is anticipated that significant roof structure demolition and reconstruction will be required. In order to minimize impact to the existing seismic diaphragm below it is recommended that where possible the new walkway be located over the existing hallway below, between the existing central masonry bearing walls. The new walkway enclosure will incorporate a seismic separation between the new enclosure and the existing main wing masonry wall.

- **New or expanded openings in walls:** the existing entry of the main wing will be reconfigured as part of the new tenant improvements, including the addition of or expansion of openings within the existing masonry bearing walls. These modifications as understood at this time do not appear to create new seismic deficiencies, but may require strengthening for vertical and/or out-of-plane lateral support. New support would be in the form of new steel channel or plate jambs and lintels spanning the width of the new opening.

- **Miscellaneous:** architectural, mechanical, electrical, and plumbing improvements are anticipated as would typically be part of a tenant improvement.

We trust this provides the information you require. Please do not hesitate to contact our office if you have questions regarding the above findings or if we may provide any additional assistance.

Sincerely,

Kyle White, P.E., CA C75960
Project Engineer

Terry Fernandez, S.E., CA S3256
Partner