ELIGIBILITY EVALUATION REPORT

School District: Beverly Hills Unified School District
School Campus: Beverly Hills High School
School Address: 241 Moreno Drive, Beverly Hills, CA 90212
Building Name/ID: Building E – East & West Wings

Original
Report Date: 12/12/2013
Last Revision
Date:

Report Outline
1. Eligibility check summary
2. Evaluation process
3. Site and building description
4. Deficiency list
5. ASCE 31 Evaluation statements

1. Eligibility Check Summary

1.1 Building Occupancy: The building’s current or planned use involves regular occupancy by students and staff, as detailed in Section 3.2.

1.2 Structural System: The building’s seismic force-resisting system includes at least one of the types listed in Section 3.5.

1.3 Collapse Potential: The building has deficiencies associated with a high potential for local or global collapse in the evaluation earthquake. See Sections 4 and 5 for a list of identified deficiencies. Among the identified deficiencies are the critical items checked in Section 1.3.3:

1.3.1 Collapse Potential Due to Ground Shaking: Ss = 1.241

1.3.2 Collapse Potential Due to One of the Following Geologic Hazards (CGS Approved Geologic Hazard Report Required):

<table>
<thead>
<tr>
<th>Geologic Hazard</th>
<th>Yes</th>
<th>No</th>
</tr>
</thead>
<tbody>
<tr>
<td>Liquefaction</td>
<td>☒</td>
<td>☐</td>
</tr>
<tr>
<td>Slope Stability Failure</td>
<td>☐</td>
<td>☒</td>
</tr>
<tr>
<td>Surface Fault Rupture</td>
<td>☐</td>
<td>☒</td>
</tr>
</tbody>
</table>

SAIFUL/BOUQUET STRUCTURAL ENGINEERS
SE Firm Name (Logo optional)
SE Address: 155 North Lake Avenue, 6th Floor
Pasadena, CA 91101

Phone: (626) 304-2616 www.saifulbouquet.com Saiful Islam, PhD, S.E.
(website or email address optional)
Name of SE whose stamp is above
1.3.3 Identified Deficiencies:

- Load Path
- Weak Story
- Soft Story
- Vertical Discontinuities
- Mass
- Torsion
- Adjacent Buildings
- Mezzanines
- Shear Stress Check (Column)
- Axial Stress Check
- Flat Slab Frames
- Captive Columns
- Beam Bars
- Deflection Compatibility
- Flat Slabs
- Redundancy
- Unreinforced Masonry Bearing Walls
- Shear Stress Check (Shear Wall or Infill)
- Redundancy (Shear Wall)
- Openings at Shear Walls
- Topping Slab
- Wall Anchorage
- Other *

SE Firm Name: Saiful/Bouquet Structural Engineers
SE Firm Address: 155 North Lake Avenue, 6th Floor, Pasadena, CA 91101
SE Firm Phone #: (626) 304-2616

Original Report Date: 12/12/2013
Last Revision Date: 

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2. Evaluation Process

2.1 Purpose and Scope

As described in DSA Procedure 08-03, the primary purpose of this evaluation is to confirm the subject building’s eligibility for Proposition 1D (AB 127, 2006) retrofit funding.

As noted in DSA Procedure 08-03, the intent of this evaluation is to identify conditions that represent “a high potential for catastrophic collapse.” As described further in Sections 2.2 through 2.4, the evaluation includes:

- Completion of a standardized checklist developed specially for this project (Section 2.2). As described in Section 2.2, once a critical deficiency is confirmed, the balance of the checklist need not be completed.
- A site visit (Section 2.3)
- Document review (Section 2.4)

It is not the intent of this evaluation to provide a complete Life Safety evaluation; earthquake safety hazards other than those listed in this report might exist. Further, it is not the intent of this evaluation to identify deficiencies with respect to post-earthquake use or recovery feasibility. In particular, except where specifically noted, the scope of this evaluation does not include:

- Material testing or destructive investigation
- Comprehensive condition assessment or verification of construction documents
- Assessment of code compliance, either at present or at the time of construction
- Assessment for load combinations not including earthquake effects
- Consideration of Life Safety hazards related to egress
- Consideration of Life Safety hazards related to hazardous materials
- Consideration of the effects of damage to nonstructural components or contents.

Building located on sites with geologic hazards (liquefaction, slope failure, faulting) may be eligible for the Proposition 1D funding if it can be demonstrated that the geologic hazard may cause the building to have a high potential for catastrophic collapse. In this case, a geologic hazard report shall be prepared and submitted to CGS for approval and a copy included with evaluation report. The geologic hazard report shall identify the resulting displacements that will be imposed on the structure so a structural analysis can be performed. If eligibility is being sought for a deficiency that is not related to geologic hazards, then a geologic hazard report does not need to be prepared for the purpose of this evaluation report.

With respect to DSA Procedure 08-03, this report fulfills the intent of its Section 1. The remaining sections of Procedure 08-03 are outside the scope of this evaluation and report:

2.2 Evaluation criteria: Modifications to ASCE 31

As noted in DSA Procedure 08-03, the evaluation applies ASCE 31, an engineering standard that allows the user to choose a performance level of either Life Safety or Immediate Occupancy. Procedure 08-03 suggests that Life Safety is the performance level of interest, but the Procedure also focuses on collapse, a lesser performance level not explicitly addressed by ASCE 31. For this evaluation, DSA has clarified that only collapse-prone conditions need to be identified. Further, because the focus of this evaluation is on checking eligibility for retrofit funding, as opposed to producing a comprehensive list of potential deficiencies, the full evaluation need not be completed once a critical deficiency is identified.
ASCE 31 involves three “tiers” of evaluation. Tier 1 uses a set of generic, mostly qualitative “evaluation statements” (also called checklists) to identify potential deficiencies. Tier 2 applies more quantitative checks to confirm or correct the Tier 1 findings. Tier 3 involves a more thorough structural analysis. For this evaluation, DSA has clarified that only Tier 1 is required for most issues, with Tier 2 evaluation for specific issues.

The criteria used for this evaluation therefore are based on the ASCE 31 Tier 1 checklists, with the following modifications:

- Basic Structural, Supplemental Structural, and Foundations checklists are considered.
- Nonstructural checklists are excluded. While some issues addressed by these checklists are relevant to nonstructural collapse potential, their completion is beyond the scope of this evaluation. While not considered for purposes of establishing funding eligibility, relevant deficiencies will be investigated and addressed during a retrofit design phase.
- Evaluation statements required by ASCE 31 for Immediate Occupancy only are excluded.
- Evaluation statements not associated with one of the eligible structure types are excluded.
- Certain evaluation statements related to “critical deficiencies” indicative of a high potential for structural collapse are identified. If a critical deficiency is confirmed, the balance of the evaluation need not be completed. The critical deficiencies are those listed in Section 1. They were selected by DSA for this project based in part on precedents set by the California Office of Statewide Health Planning and Development.
- For Quick Checks and Tier 2 evaluations, the ASCE 31 criteria for Life Safety performance are used, except that $m$ values, where needed, are increased by an additional factor of 1.33.
- The Tier 1 evaluation statements are modified to reflect emphasis on collapse-level performance:
  - Since the presence of an unreinforced masonry bearing wall system is deemed a critical deficiency, an evaluation statement to that effect is added, and detailed ASCE 31 evaluation statements specific to that system are omitted.
  - Condition of Materials: Evaluation statements are edited to focus less on presence of damage and more on significance of damage. Note that Masonry Lay-up and Foundation Performance evaluation statements are relocated to the Condition of Materials subsection of Section 5.
  - Except for cracks in certain concrete members, Condition of Materials evaluation statements related to existing cracks are omitted.
  - Beam Bars: The requirement for 25 percent of the joint bars to be continuous for the length of the member is removed.
  - Redundancy (Moment frame and Braced frame): The requirement for two bays per frame line is removed.
  - Stiffness of wall anchors: The limitation of 1/8-inch gap prior to anchor engagement is removed.
  - Overturning: This statement is removed.
  - In general, statements are modified for clarity and consistency with this DSA program.
- Tier 2 evaluation is required for any critical item (see Section 1) found to be non-compliant by Tier 1. The potential requirement for full-building Tier 2 evaluation found in ASCE 31 Table 3-3 is waived.

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2.3 Document review
The following documents were provided for use in completing the evaluation, in general compliance with ASCE 31, Section 2.2. The Set ID is used to identify the documents cited in Section 5 of this report.

<table>
<thead>
<tr>
<th>SET ID</th>
<th>DATE</th>
<th>DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>5/22/1967</td>
<td>BEVERLY HILLS HIGH SCHOOL</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Additions and Alterations</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Rowland H. Crawford A.I.A. Architect</td>
</tr>
<tr>
<td></td>
<td></td>
<td>S.B. Barnes &amp; Associates, Structural Engineers</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Architectural sheets E-1 through E-17</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Structural sheets E-S1 through E-S6 plus sheets X-S1 and X-S2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>DSA Application # 28655</td>
</tr>
</tbody>
</table>

2.4 Site visit
In general compliance with ASCE 31, Sections 2.2 and 2.3, a site visit shall be made to verify the building configuration and conditions and to assist in completing the evaluation.

Dates of site visit: 4/12/2012, 5/10/2012
Visiting engineer(s) and staff: Robert Randall, S.E.
School district contact person: Gary Woods (Superintendent)
School campus representative (if different than above): Charlotte Clement (BHUSD)

The scope of the site visit was based on our judgment, accessibility of certain areas, and convenience of the school on-site liaison. The purpose of the following list is merely to record the work that was done. The site visit included (check all applicable boxes):

- INTERVIEW W/ ON-SITE LIAISON
- GROUNDS, FOR OBSERVATION OF SOIL, SLOPES, DRAINAGE, GENERAL CONDITION
- EXTERIOR OBSERVATION TO VERIFY BASIC MASSING, CONFIGURATION, GENERAL CONDITION
- INTERIOR OBSERVATION TO VERIFY USE, WALL LINE CONFIGURATION, GENERAL CONDITION
- ROOF
- BASEMENT
- CEILING PLENUM
- UNFINISHED SPACES (MECHANICAL ROOMS, CLOSETS, CRAWL SPACES, ETC.)
- DETAILS OF STRUCTURE-ARCHITECTURE INTERACTION
- ROOF-TO-WALL CONNECTIONS
- GRAVITY SYSTEM FRAMING
- SEISMIC FORCE RESISTING SYSTEM ELEMENTS OR COMPONENTS
- ADJACENT BUILDINGS SUBJECT TO POUNDING
- DESTRUCTIVE VALIDATION OF EXISTING BRICK/CONCRETE WALL CONSTRUCTION.
- OTHER:

The site visit confirmed that the existing structure generally conforms to the available drawings listed in Section 2.3.
### 3. Site and Building Description

#### 3.1 Building description

**General**

<table>
<thead>
<tr>
<th>Year originally built:</th>
<th>1967</th>
</tr>
</thead>
<tbody>
<tr>
<td>DSA Application number</td>
<td>28655</td>
</tr>
<tr>
<td>Original Construction</td>
<td>☑</td>
</tr>
<tr>
<td>Work done pursuant to the Garrison Act (Ed Code 17367)</td>
<td>☐</td>
</tr>
</tbody>
</table>

- Number of stories above grade: 2
- Number of stories below grade: 0
- Total floor area (sq ft, approx): 22,000
- Other essentially identical buildings on this campus? ☑ Yes ☐ No

There are no similar buildings on this campus.

**Photographs**

Exterior elevation photograph, looking, **Southwest** taken: 4/12/12

#### Ground floor plan

![Ground floor plan](image-url)

**SE Firm Name:** Saiful/Bouquet Structural Engineers  
**SE Firm Address:** 155 North Lake Avenue, 6th Floor, Pasadena, CA 91101  
**SE Firm Phone #:** (626) 304-2616
3.2 Building Occupancy

Original, current, and planned uses of the building include those indicated here:

<table>
<thead>
<tr>
<th>Original Use</th>
<th>Current Use</th>
<th>Planned Future Use</th>
</tr>
</thead>
<tbody>
<tr>
<td>Office / Administration</td>
<td>☒</td>
<td>☒</td>
</tr>
<tr>
<td>Classrooms / Instruction Areas</td>
<td>☒</td>
<td>☒</td>
</tr>
<tr>
<td>Kitchen</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Assembly: Dining</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Assembly: Auditorium</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Assembly: Gymnasium</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Locker Rooms</td>
<td>☒</td>
<td></td>
</tr>
<tr>
<td>Patio cover / bus shelter / walkway cover</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bleachers / stadium structure</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Other occupied: None</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mechanical / utility rooms</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bulk storage</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Vacant / unused</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Other unoccupied:</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

3.3 Seismicity

Latitude: 34.061970
Longitude: -118.412425

Site Class per ASCE 31, Section 3.5.2.3: Class D

Basis for Site Class determination: Default per 3.5.2.3.1

<table>
<thead>
<tr>
<th>Period [sec]</th>
<th>Mapped MCE values from ASCE 7-05 [g]</th>
<th>Site Coefficients from ASCE 31 Tables 3-5, 3-6</th>
<th>Design values per ASCE 31 section 3.5.2.3.1 [g]</th>
<th>$S_a$ per ASCE 31 section 3.5.2.3.1, [g]</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.2</td>
<td>$S_d = 1.860$</td>
<td>$F_a = 1.0$</td>
<td>$S_{DS} = (2/3) S_d F_a = 1.240$</td>
<td>$S_{a,0.2} = S_{DS} = 1.240$</td>
</tr>
<tr>
<td>1.0</td>
<td>$S_i = 0.633$</td>
<td>$F_v = 1.5$</td>
<td>$S_{DS} = (2/3) S_i F_v = 0.633$</td>
<td>$S_{a,1.0} = \min (S_{DS}, S_{DS}/T) = 1.240$</td>
</tr>
</tbody>
</table>
### 3.4 Gravity System

**Roof diaphragm and framing:** The roof framing consists of ½” plywood sheathing over wood joists which span to tapered steel girders.

**Typical floor diaphragm and framing:** The 2nd floor consists of a built-up wood flooring system with ½” plywood as the diaphragm, which sits on top of wood joists that span to steel wide flange girders.

**Ground floor framing:** The first floor consists of a concrete slab-on-grade.

**Vertical load-bearing elements:** Exterior masonry bearing walls and interior steel columns.

**Basement walls:** No basement.

**Foundation:** The foundation consists of shallow continuous footings underneath bearing and shear walls with isolated shallow spread footings underneath interior columns.

**Snow load for use in load combinations involving earthquake:** Snow load not required.

### 3.5 Structural System per ASCE 31 Classifications (Category 2 Buildings Types per AB 300 Report)

<table>
<thead>
<tr>
<th>Classification</th>
<th>North-South</th>
<th>East-West</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>C1B*</td>
<td></td>
<td></td>
</tr>
<tr>
<td>C2A</td>
<td></td>
<td></td>
</tr>
<tr>
<td>C3A</td>
<td></td>
<td></td>
</tr>
<tr>
<td>PC1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>PC1A</td>
<td></td>
<td></td>
</tr>
<tr>
<td>PC2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>PC2A</td>
<td></td>
<td></td>
</tr>
<tr>
<td>RM1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>S1B*</td>
<td></td>
<td></td>
</tr>
<tr>
<td>S3</td>
<td></td>
<td></td>
</tr>
<tr>
<td>URM</td>
<td></td>
<td></td>
</tr>
<tr>
<td>URMA</td>
<td></td>
<td></td>
</tr>
<tr>
<td>M*</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**List the structural system(s) here.**

None of the above

**List the present structural system(s) here.**

* These structural systems are a subset of the classification in ASCE 31 and are defined in the Category 2 building types in the AB 300 Seismic Safety Inventory of California Public Schools report (2002).
### Horizontal system combinations
The typical roof and floor diaphragms consist of $\frac{1}{2}''$ structural plywood sheathing supported vertically by wood joists spanning between steel framing.

### Vertical system combinations
The typical vertical elements consist of 10" thick (nominal) reinforced brick masonry walls.

### SFRS foundation
Shallow continuous footings under walls.

### Gravity loading
Both the interior and exterior masonry shear walls are bearing.

### System details
The roof and floor diaphragms consist of $\frac{1}{2}''$ plywood (blocked) and have cross ties and joist anchors spaced at 4'-0" o.c. Typical bearing and shear walls are 10” brick masonry reinforced with a single curtain of #4 @ 13" o.c.

### Structural materials
**Set 1:**
- Concrete ultimate compressive strength = 3,000 psi.
- Masonry strength not specified, use $f_m=1,000$ psi per ASCE 31.
- Rebar strength not specified, use $f_y=33$ ksi per ASCE 31.
- Wood grade not specified.

### Original design code
Design code not specified, assume 1964 UBC.

### History of seismic retrofit or significant alteration
These buildings have not had any seismic retrofit or significant alterations.

### Benchmark year check
This building does not meet the benchmark criteria.
4. Deficiency list
The following table summarizes the potential deficiencies identified in Section 5 of this report.

<table>
<thead>
<tr>
<th>Non-compliant condition</th>
<th>Discussion</th>
<th>Additional evaluation recommended</th>
</tr>
</thead>
</table>
| ADJACENT BUILDINGS               | • The gap between these buildings and the adjacent gymnasium building is less than 4% of the height of the shorter building.  
  • Floors align with adjacent building floors, so this is not expected to be a critical deficiency. | None.                            |
| WALL ANCHORAGE                   | • The wall anchors of the wood diaphragms to the concrete walls are not adequate to resist the force required per ASCE 31. This is typical for the entire building.  
  • This deficiency can cause both local and global collapse in a large earthquake and needs to be addressed. | None.                            |
## 5. ASCE 31 Evaluation Statements

Evaluation statements provided in this section are from ASCE 31. They have been modified for this project with DSA approval as described in Section 2.2 of this report. References within the evaluation statements to other section numbers are generally to sections of ASCE 31.

- **C** = Compliant
- **NC** = Non-compliant
- **U** = Unknown or not investigated
- **NA** = Not applicable to this building

Items marked NC or U are summarized in Section 4 of this report.

### CONDITION OF MATERIALS

<table>
<thead>
<tr>
<th></th>
<th>NC</th>
<th>U</th>
<th>NA</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>DETERIORATION OF WOOD. There shall be no evidence of or reason to suspect structural capacity loss due to decay, shrinkage, splitting, fire damage, or sagging in wood members or deterioration, damage, or loosening in metal connection hardware.</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>DETERIORATION OF CONCRETE. There shall be no evidence of or reason to suspect structural capacity loss due to cracking of concrete or deterioration of concrete or reinforcing steel in gravity or seismic force-resisting elements.</td>
<td>C</td>
<td></td>
</tr>
<tr>
<td></td>
<td>DETERIORATION OF STEEL. There shall be no evidence of or reason to suspect structural capacity loss due to rusting, corrosion, cracking, or other deterioration in any of the steel elements or connections in the gravity or seismic force-resisting elements.</td>
<td>C</td>
<td></td>
</tr>
<tr>
<td></td>
<td>POST-TENSIONING ANCHORS. There shall be no evidence of or reason to suspect structural capacity loss due to corrosion or spalling in the vicinity of post-tensioning or end fittings. Coil anchors shall not have been used.</td>
<td>C</td>
<td></td>
</tr>
<tr>
<td></td>
<td>PRECAST CONCRETE WALLS. There shall be no evidence of or reason to suspect structural capacity loss due to deterioration of concrete or reinforcing steel or distress, especially at connections.</td>
<td>C</td>
<td></td>
</tr>
<tr>
<td></td>
<td>MASONRY UNITS. There shall be no evidence of or reason to suspect structural capacity loss due to deterioration of masonry units.</td>
<td>C</td>
<td></td>
</tr>
<tr>
<td></td>
<td>MASONRY JOINTS. The mortar shall not be easily scraped away from the joints by hand with a metal tool, and there shall be no evidence of or reason to suspect structural capacity loss due to eroded mortar.</td>
<td>C</td>
<td></td>
</tr>
<tr>
<td></td>
<td>MASONRY LAY-UP. Filled collar joints of multi-wythe masonry infill walls shall have negligible voids.</td>
<td>C</td>
<td></td>
</tr>
<tr>
<td></td>
<td>FOUNDATION PERFORMANCE. There shall be no evidence of or reason to suspect existing foundation movement (due to settlement, heave, or other causes) that would affect the integrity or strength of the structure.</td>
<td>C</td>
<td></td>
</tr>
</tbody>
</table>
# BUILDING CONFIGURATION

<table>
<thead>
<tr>
<th>Critical Item</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>LOAD PATH.</td>
<td>The structure shall contain a minimum of one complete load path for seismic force effects from any horizontal direction that serves to transfer the inertial forces from the mass to the foundation.</td>
</tr>
<tr>
<td>WEAK STORY.</td>
<td>The strength of the seismic force-resisting system in any story shall not be less than 80% of the strength in an adjacent story, above or below.</td>
</tr>
<tr>
<td>SOFT STORY.</td>
<td>The stiffness of the seismic force-resisting system in any story shall not be less than 70% of the seismic force-resisting system stiffness in an adjacent story above or below, or less than 80% of the average seismic force-resisting system stiffness of the three stories above or below.</td>
</tr>
<tr>
<td>GEOMETRY.</td>
<td>There shall be no changes in 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.</td>
</tr>
<tr>
<td>VERTICAL DISCONTINUITIES.</td>
<td>All vertical elements of the seismic force-resisting system shall be continuous to the foundation.</td>
</tr>
<tr>
<td>MASS.</td>
<td>There shall be no change in effective mass more than 50% from one story to the next. Light roofs, penthouses and mezzanines need not be considered.</td>
</tr>
<tr>
<td>TORSION.</td>
<td>The estimated distance between the story center of mass and the story center of rigidity shall be less than 20% of the building width in either plan dimension.</td>
</tr>
<tr>
<td>ADJACENT BUILDINGS.</td>
<td>The clear distance between the building being evaluated and any adjacent building shall be greater than 4% of the height of the shorter building. Alternatively, if the 4% separation does not exist, the two buildings shall be configured such that pounding would not damage the columns of the subject building within the clear span of the columns. <em>The buildings are configured such that the floors do align and will not pound any adjacent columns or bearing walls.</em></td>
</tr>
<tr>
<td>MEZZANINES.</td>
<td>Interior mezzanine levels shall be braced independently from the main structure, or shall be anchored to the seismic force-resisting elements of the main structure.</td>
</tr>
</tbody>
</table>

# MOMENT FRAMES

<table>
<thead>
<tr>
<th>Critical Item</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>SHEAR STRESS CHECK (Columns).</td>
<td>The shear stress in concrete columns of the seismic force-resisting system, calculated using the Quick Check procedure of Section 3.5.3.2, shall be less than the greater of 100 psi or 2\sqrt{f'_c}.</td>
</tr>
</tbody>
</table>
### Critical Item

**AXIAL STRESS CHECK (Concrete columns).** The axial stress due to gravity loads in columns subjected to seismic overturning forces shall be less than $0.10\sigma_c$. Alternatively, the axial stresses due to overturning forces alone, calculated using the Quick Check procedure of Section 3.5.3.6, shall be less than $0.30\sigma_c$.

**AXIAL STRESS CHECK (Steel columns).** The axial stress due to gravity loads in steel columns subjected to seismic overturning forces shall be less than $0.10F_y$. Alternatively, the axial stresses due to overturning forces alone, calculated using the Quick Check procedure of Section 3.5.3.6, shall be less than $0.30F_y$.

**FLAT SLAB FRAMES.** The seismic force-resisting system shall not be a frame consisting of columns and a flat slab/plate without beams.

**PRESTRESSED FRAME ELEMENTS.** The seismic force-resisting frames shall not include any prestressed or post-tensioned elements where the average prestress exceeds the lesser of 700 psi or $F_c/6$ at potential hinge locations. The average prestress shall be calculated in accordance with the Quick Check Procedure of Section 3.5.3.8.

**CAPTIVE COLUMNS.** There shall be no columns at a level with height/depth ratios less than 50% of the nominal height/depth ratio of the typical columns at that level.

**NO SHEAR FAILURES.** The shear capacity of frame members in the seismic force-resisting system shall be able to develop the moment capacity at the ends of the members.

**STRONG COLUMN/WEAK BEAM.** The sum of the moment capacity of the columns shall be 20% greater than that of the beams at concrete frame joints.

**STRONG COLUMN/WEAK BEAM.** The percent of strong column/weak beam joints in each story of each line of steel moment-resisting frames shall be greater than 50%. This check need not apply for 1-story structures.

**BEAM BARS.** At least two longitudinal top and two longitudinal bottom bars shall extend continuously throughout the length of each frame beam.

**COLUMN BAR SPLICES.** All column bar lap splice lengths shall be greater than $35d_0$, and shall be enclosed by ties spaced at or less than $8d_0$. Alternatively, column bars shall be spliced with mechanical couplers with a capacity of at least 1.25 times the nominal yield strength of the spliced bar.

**BEAM BAR SPLICES.** The lap splices or mechanical couplers for longitudinal beam reinforcing shall not be located within $l_b/4$ of the joints and shall not be located in the vicinity of potential plastic hinge locations.

**COLUMN TIE SPACING.** Frame columns shall have ties spaced at or less than $d/4$ throughout their length and at or less than $8d_b$ at all potential plastic hinge locations.

---

<table>
<thead>
<tr>
<th>SE Firm Name:</th>
<th>Saiful/Bouquet Structural Engineers</th>
</tr>
</thead>
<tbody>
<tr>
<td>SE Firm Address:</td>
<td>155 North Lake Avenue, 6th Floor, Pasadena, CA 91101</td>
</tr>
<tr>
<td>SE Firm Phone #:</td>
<td>(626) 304-2616</td>
</tr>
<tr>
<td>Critical Item</td>
<td>Description</td>
</tr>
<tr>
<td>---------------</td>
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</tr>
<tr>
<td><strong>STIRRUP SPACING.</strong> All beams shall have stirrups spaced at or less than ( \frac{d}{2} ) throughout their length. At potential plastic hinge locations stirrups shall be spaced at or less than the minimum of ( 8d_b ) or ( \frac{d}{4} ).</td>
<td></td>
</tr>
<tr>
<td><strong>JOINT REINFORCING.</strong> Beam-column joints shall have ties spaced at or less than ( 8d_b ).</td>
<td></td>
</tr>
<tr>
<td><strong>COMPLETE FRAMES.</strong> Concrete frames that are not part of the seismic force-resisting system shall form a complete gravity load carrying system.</td>
<td></td>
</tr>
<tr>
<td><strong>DEFLECTION COMPATIBILITY.</strong> Elements of concrete frames that are not part of the seismic force-resisting system shall have the shear capacity to develop the flexural strength of the components.</td>
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<tr>
<td><strong>FLAT SLABS.</strong> Flat slabs/plates that are not part of the seismic force-resisting system shall have continuous bottom steel through the column joints.</td>
<td></td>
</tr>
<tr>
<td><strong>REDUNDANCY (Moment frame).</strong> The number of lines of moment frames in each principal direction shall be greater than or equal to 2.</td>
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</tr>
<tr>
<td><strong>INTERFERING WALLS.</strong> All concrete and masonry infill walls placed in moment frames shall be isolated from structural elements. (This evaluation statement does not apply to seismic force-resisting system type C3A or others where the infill is being evaluated as a shear wall or force-resisting element.)</td>
<td></td>
</tr>
<tr>
<td><strong>PRECAST CONNECTION CHECK.</strong> The connections at joints of precast concrete frames shall have the capacity to resist the shear and moment demands calculated using the Quick Procedure of Section 3.5.3.5.</td>
<td></td>
</tr>
<tr>
<td><strong>PRECAST FRAMES.</strong> For buildings with concrete shear walls, precast concrete frame elements shall not be necessary as primary components for resisting seismic forces.</td>
<td></td>
</tr>
<tr>
<td><strong>PRECAST CONNECTIONS.</strong> For buildings with concrete shear walls, the connections between precast frame elements such as chords, ties, and collectors in the seismic force-resisting system shall develop the capacity of the connected members.</td>
<td></td>
</tr>
<tr>
<td><strong>DRIFT CHECK:</strong> The drift ratio of the steel moment frames, calculated using the Quick Check procedure of Section 3.5.3.1, shall be less than 0.025.</td>
<td></td>
</tr>
<tr>
<td><strong>MOMENT-RESISTING CONNECTIONS:</strong> All moment connections shall be able to develop the strength of the adjoining members or panel zones.</td>
<td></td>
</tr>
<tr>
<td><strong>PANEL ZONES:</strong> All panel zones shall have the shear capacity to resist the shear demand required to develop 0.8 times the sum of the flexural strengths of the girders framing in at the face of the column.</td>
<td></td>
</tr>
<tr>
<td><strong>COLUM SPLICES:</strong> All column splice details located in moment-resisting frames shall include connection of both flanges and the web.</td>
<td></td>
</tr>
<tr>
<td><strong>COMPACT MEMBERS:</strong> All frame elements shall meet section requirements set forth by Table I-9-1 of Seismic Provisions for Structural Steel Buildings (AISC, 1997).</td>
<td></td>
</tr>
</tbody>
</table>
## SHEAR WALLS

### Critical Item
- **UNREINFORCED MASONRY BEARING WALLS.** The seismic force-resisting system in any direction shall not rely on or consist primarily of unreinforced masonry bearing walls.

### Critical Item
- **SHEAR STRESS CHECK (Shear wall).** 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 \(2\sqrt{f_c}\).

### Critical Item
- **REINFORCING STEEL.** In concrete or precast shear walls, the ratio of reinforcing steel area to gross concrete area shall not be less than 0.0015 in the vertical direction and 0.0025 in the horizontal direction. The spacing of reinforcing steel shall be equal to or less than 18 inches.

### Critical Item
- **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.

### Critical Item
- **REDUNDANCY (Shear wall).** The number of lines of shear walls in each principal direction shall be greater than or equal to 2.

### Critical Item
- **PROPORTIONS.** The height-to-thickness ratio of masonry infill walls at each story shall be less than 9. (This evaluation statement applies only to seismic force-resisting system type C3A and others where the infill is being evaluated as a shear wall or force-resisting element.)

### Critical Item
- **SOLID WALLS.** The masonry infill walls shall not be of cavity construction. (This evaluation statement applies only to seismic force-resisting system type C3A and others where the infill is being evaluated as a shear wall or force-resisting element.)

### Critical Item
- **INFILL WALLS.** The infill walls shall be continuous to the soffits of the frame beams and to the columns to either side. (This evaluation statement applies only to seismic force-resisting system type C3A and others where the infill is being evaluated as a shear wall or force-resisting element.)

### Critical Item
- **SHEAR STRESS CHECK (Precast concrete shear walls).** The shear stress in the precast panels, calculated using the Quick Check procedure of Section 3.5.3.3, shall be less than the greater of 100 psi or \(2\sqrt{f_c}\).

### Critical Item
- **WALL OPENINGS.** The total width of openings along any perimeter wall line shall constitute less than 75% of the length of any perimeter shear wall, with the wall piers having height-to-width ratios of less than 2 to 1.

### Critical Item
- **CORNER OPENINGS.** Walls with openings at a building corner larger than the width of a typical panel shall be connected to the remainder of the wall with collector reinforcing.

### Critical Item
- **SHEAR STRESS CHECK (Brick or hollow clay masonry infill).** The shear stress in the masonry shear walls, calculated using the Quick Check procedure of Section 3.5.3.3, shall be less than 30 psi for clay units.
**ELIGIBILITY EVALUATION REPORT**

<table>
<thead>
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<tr>
<td>School District:</td>
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</tr>
<tr>
<td>School Campus:</td>
<td>Beverly Hills High School</td>
</tr>
<tr>
<td>School Address:</td>
<td>241 Moreno Drive, Beverly Hills, CA 90212</td>
</tr>
<tr>
<td>Building Name/ID:</td>
<td>Building E – East &amp; West Wings</td>
</tr>
<tr>
<td>Project Tracking No.:</td>
<td>Page 16 of 25</td>
</tr>
</tbody>
</table>

**Critical Item**

1. **SHEAR STRESS CHECK** (Concrete block infill and reinforced masonry shear walls). The shear stress in the masonry shear walls, calculated using the Quick Check procedure of Section 3.5.3.3, shall be less than 70 psi for concrete units.

2. **PROPORTIONS.** The height-to-thickness ratio of unreinforced masonry infill shear walls shall be less than the following: Top story of multi-story building: 9, First story of multi-story building: 15, All other conditions: 13

3. **REINFORCING STEEL.** In reinforced masonry shear walls, the total vertical and horizontal reinforcing steel ratio shall be greater than 0.002 of the wall with the minimum of 0.0007 in either of the two directions; the spacing of reinforcing steel shall be less than 48”; and all vertical bars shall extend to the top of the walls.

**BRACED FRAMES**

1. **REDUNDANCY:** The number of lines of braced frames in each principal direction shall be greater than or equal to 2.

2. **AXIAL STRESS CHECK:** The axial stress in the diagonals, calculated using the Quick Check Procedure of Section 3.5.3.4, shall be less than 0.50Fy.

3. **SLENDERNESS OF DIAGONALS:** All diagonal elements required to carry compression shall have KI/r ratios less than 120.

4. **CONNECTION STRENGTH:** All the brace connections shall develop the yield capacity of the diagonals.

5. **K-BRACING:** The bracing system shall not include K-braced bays.

**DIAPHRAGMS**

1. **DIAPHRAGM CONTINUITY.** The diaphragm shall not be composed of split-level floors and shall not have expansion joints.

2. **CROSS TIES.** There shall be continuous cross ties between diaphragm chords.

3. **ROOF CHORD CONTINUITY.** All roof chord elements shall be continuous, regardless of changes in roof elevation.

4. **OPENINGS AT SHEAR WALLS.** Diaphragm openings immediately adjacent to the shear walls shall be less than 25% of the wall length, and diaphragm openings immediately adjacent to exterior masonry shear walls shall not be greater than 8 ft long.

5. **OPENINGS AT BRACED FRAMES.** Diaphragm openings immediately adjacent to the braced frames shall extend less than 25% of the frame length.

**SE Firm Name:** Saiful/Bouquet Structural Engineers

**SE Firm Address:** 155 North Lake Avenue, 6th Floor, Pasadena, CA 91101

**SE Firm Phone #:** (626) 304-2616

**PR 08-03**

**SMP Template (iss 09-15-11)**

**errata 10-11-11**
**ELIGIBILITY EVALUATION REPORT**

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<td>Beverly Hills High School</td>
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<td></td>
</tr>
<tr>
<td>School Address:</td>
<td>241 Moreno Drive, Beverly Hills, CA 90212</td>
<td></td>
<td></td>
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</table>

| C NC U NA | OTHER DIAPHRAGMS. The diaphragm shall not consist of a system other than wood, metal deck, concrete or horizontal bracing. |
| Critical Item | TOPPING SLAB. Precast concrete diaphragm elements shall be interconnected by a continuous reinforced concrete topping slab. |
| C NC U NA | STRAIGHT SHEATHING. All straight sheathed diaphragms shall have aspect ratios less than 2 to 1 in the direction being considered. |
| C NC U NA | SPANS. All wood diaphragms with spans greater than 24 ft shall consist of wood structural panels or diagonal sheathing. |
| C NC U NA | UNBLOCKED DIAPHRAGMS. All diagonally sheathed or unblocked wood structural panel diaphragms shall have horizontal spans less than 40 ft and shall have aspect ratios less than or equal to 4 to 1. |

**CONNECTIONS**

| C NC U NA | WALL ANCHORAGE. Exterior concrete or masonry walls shall be anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps 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 1 calculation is provided in Appendix A.2. By inspection, the force level required for the Tier 2 check per 4.2.6.3.5 is greater than the force level for the Tier 1 check. Therefore the wall anchorage does not pass a Tier 2 evaluation. Typical anchorage detail is provided in Appendix A.3.*

| C NC U NA | WOOD LEDGERS. The connection between the wall panels and the diaphragm shall not induce cross-grain bending or tension in the wood ledgers. |
| C NC U NA | PRECAST PANEL CONNECTIONS. There shall be at least two anchors from each precast wall panel into the diaphragm elements. |
| C NC U NA | STIFFNESS OF WALL ANCHORS. Anchors of concrete or masonry walls to wood structural elements shall be installed taut and shall be stiff enough to limit the relative movement between the wall and the diaphragm prior to engagement of the anchors, as needed for reliable bearing. |
| C NC U NA | GIRDER/COLUMN CONNECTION. There shall be a positive connection utilizing plates, connection hardware, or straps between girders and their supporting columns. (This evaluation statement applies primarily to precast concrete and masonry systems.) |
| C NC U NA | GIRDERS. Girders supported by walls or pilasters shall have at least two additional column ties securing the anchor bolts. (This evaluation statement applies primarily to precast concrete systems.) |

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<tr>
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<td>(626) 304-2616</td>
</tr>
</tbody>
</table>
CORBEL BEARING. If precast concrete frame girders bear on column corbels, the length of bearing shall be greater than 3”.

CORBEL CONNECTIONS. Precast concrete frame girders shall not be connected to corbels with welded elements.

TRANSFER TO SHEAR WALLS. Diaphragms shall be connected for transfer of loads to shear walls.

TRANSFER TO STEEL FRAMES. Diaphragms shall be connected for transfer of loads to the steel frames.

TOPPING SLAB TO WALLS OR FRAMES. Reinforced concrete topping slabs that interconnect the precast concrete diaphragm elements shall be doweled for transfer of forces into shear wall or frame elements.

CONCRETE COLUMNS. All concrete columns shall be doweled into the foundation.

FOUNDATION DOWELS. Wall reinforcement shall be doweled into the foundation.

PRECAST WALL PANELS. Precast wall panels shall be connected to the foundation.

UPLIFT AT PILE CAPS. Pile caps shall have top reinforcement and piles shall be anchored to the pile caps.

STEEL COLUMNS: The columns in lateral-force-resisting frames shall be anchored to the building foundation.

WALL PANELS: Metal, fiberglass or cementitious wall panels shall be positively attached to the foundation.

ROOF PANELS: Metal, plastic, or cementitious roof panels shall be positively attached to the roof framing to resist seismic forces.

WALL PANELS: Metal, fiberglass or cementitious wall panels shall be positively attached to the framing to resist seismic forces.

POLE FOUNDATIONS. Pole foundations shall have a minimum embedment depth of 4 ft.

TIES BETWEEN FOUNDATION ELEMENTS. The foundation shall have ties adequate to resist seismic forces where footings, piles, and piers are not restrained by beams, slabs, or soils in Site Class A, B, or C.
# GEOLOGIC SITE HAZARDS

<table>
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<tr>
<th>Critical Item</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>LIQUEFACTION:</td>
<td>Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building’s seismic performance shall not exist in the foundation soils at depths within 50 feet.</td>
</tr>
<tr>
<td>SLOPE FAILURE:</td>
<td>The building site shall be sufficiently remote from potential earthquake-induced slope failures or rockfalls to be unaffected by such failures or shall be capable of accommodating any predicted movements without failure.</td>
</tr>
<tr>
<td>SURFACE FAULT RUPTURE:</td>
<td>Surface fault rupture and surface displacement at the building site is not anticipated.</td>
</tr>
</tbody>
</table>
Appendices

A.1 Structural calculations

**LOADING CRITERIA**

**EAST & WEST WINGS ROOF**
- Composition Roof: 6.5 psf
- 1/2" plywood sheathing: 1.6 psf
- Batt Insulation: 1.0 psf
- 2x10 joists @ 16" o.c.: 2.8 psf
- Tapered Steel Girders: 4.9 psf
- MEP: 1.0 psf
- Acoustic Tile Ceiling: 1.0 psf
- Misc: 1.2 psf

**TOTAL DL = 20.0 psf**

**EAST & WEST WINGS 2ND FLOOR**
- Hardwood Floor (3/4" plywood+vinyl): 2.4 psf
- 1/2" plywood: 1.6 psf
- 2"x3" @ 16" o.c. flat: 1.1 psf
- 1" rigid insulation: 1.0 psf
- 1/2" plywood (structural): 1.6 psf
- 2x12 joists @ 16" o.c.: 3.3 psf
- MEP: 2.0 psf
- Acoustic Tile Ceiling: 1.0 psf
- Misc: 1.0 psf

**TOTAL DL = 15.0 psf**

**TYPICAL EAST & WEST WING WALLS**
- 10" Brick: 100 psf
Conterminous 48 States
Latitude = 34.060953
Longitude = -118.411857
Spectral Response Accelerations Ss and S1
Ss and S1 = Mapped Spectral Acceleration Values
Site Class B - Fa = 1.0, Fv = 1.0
Data are based on a 0.01 deg grid spacing

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<td>1.0</td>
<td>0.636 (S1, Site Class B)</td>
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</tbody>
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Conterminous 48 States
Latitude = 34.060953
Longitude = -118.411857
Spectral Response Accelerations SMs and SM1
SMs = Fa x Ss and SM1 = Fv x S1
Site Class D - Fa = 1.0, Fv = 1.5

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<td>0.954 (SM1, Site Class D)</td>
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Conterminous 48 States
Latitude = 34.060953
Longitude = -118.411857
Design Spectral Response Accelerations SDs and SD1
SDs = 2/3 x SMs and SD1 = 2/3 x SM1
Site Class D - Fa = 1.0, Fv = 1.5

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<tr>
<td>1.0</td>
<td>0.636 (SD1, Site Class D)</td>
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</tbody>
</table>
SHEAR STRESS CHECK

USE Eq 3.12 FROM SECTION 3.5.33

\[ E_\text{q} 3.12: V_s^{\max} = \frac{1}{m} \left[ \frac{V_s^*}{A_s} \right] \]

\[ m = 4.0 \quad \text{(TABLE 2.7, DEWF. WNC., LS)} \]

FIND \( V_s^* \)

FIND \( V_s \)

\[ V = CS_m W \]

\[ C = 1.0 \quad \text{(TABLE E-4, 2故事, C2A) } \]

\[ S_m = \frac{C_{d1}}{T} \]

\[ T = \frac{C_e W_n}{V_s^{\max} \text{ ch. } 3.7-8} \]

\[ T = 0.242 \quad \text{ (ch. 3.7-8)} \]

\[ W_n = 44' \quad \text{ from ENV. LOADS} \]

\[ f_{y} = 0.75 \]

\[ C_{d1} = 0.052 \quad \text{ from HAZARD HAZ} \]

\[ S_m = \frac{W_n Q_{s2}}{0.392} = 1.86 \quad \text{ > 1.241. } \]

\[ S_m = 1.241 \]

\[ V = 1.241 W \]

\[ V = 1.241 W \]
A.2 Evaluation statement notes

ASCE 31 § 3.5.3.7

\[ T_c = \frac{1}{2} S_{p_2} \omega_f A_p \]
\[ \omega_f = 1.2 \text{sf} \]

\[ S_{p_2} = 1.241 \]

\[ \omega_f = 10 \text{psf/ln x 10}^5 = 100 \text{ psf} \]
\[ A_p = 2.6' x 4' = 52 \text{ ft}^2 \]
\[ T_c = 0.9 x 1.24 x 100 \text{ psf x 52 ft}^2 = 5808 \text{ lbs} \]

Strip based on steel 13/16 X-51 Type C

2-3/8" bolts \( \leq 2x \) capacity = \( 2 \times 730 \text{ lbs} = 1460 \text{ lbs} \) (ASCE)

\[ \text{ASCE 31} \text{ capacity} = 1460 \text{ lbs x 2.14 x 1.4} = 5045 \text{ lbs} \]

\[ 5045 < 5808 \text{ \_\_\_N,6,} \]

\[ \% = 1.16 \]

WALL ANCHORAGE N,6.

SE Firm Name: Saiful/Bouquet Structural Engineers
SE Firm Address: 155 North Lake Avenue, 6th Floor, Pasadena, CA 91101
SE Firm Phone #: (626) 304-2616

PR 08-03
SMP Template (iss 09-15-11)
(errata 10-11-11)
**Base Shear V**

- **Roof Area = 4918 ft²**
  - **Roof Unit Weight = 20 psf**
  - Weight = 98.4 kips

- **2nd Floor Area = 4918 ft²**
  - **2nd Floor Unit Weight = 15 psf**
  - Weight = 73.8 kips

- **N-S wall length = 196'**
  - **height = 32'**
  - Weight = 184 kips

- **E-W wall length = 123'**
  - **height = 32'**
  - Weight = 516.8 kips

**Seismic Mass = 1173 kips**

\[ V = 1.241 \times W = 1456.3 \text{ kips} \]

**Wall Shear**

\[ V_{max} = \frac{1}{n_0} \frac{V_j}{A_w} \]

- **m = 4.0**
- **V_j = 1456 kips**
- **A_w = 156 \times 12 \times 10^{-4} = 18960 \text{ in}^2**

\[ V_{max} = 20 \text{ psi} < 70 \text{ psi} \quad \therefore \text{OK} \]

**WALL SHEAR OK**

**BY INSPECTION, EAST WING WALL SHEAR OK**
A.3 Photographs and details

Typical Joist Anchor Detail. Detail 15/S-X1 on Set 1.

![Diagram of Joist Anchor Details]

**Typical Joist Anchor Straps 15**

**Notes:**
1. All straps to be 3/4" x 1-3/4" wide. All holes to be drilled for 4-1/2" C.R. x 2-1/2" LG. Wood screws each strap unless otherwise noted.
2. Joist anchor strap for concrete walls and straps connecting rafters across steel frames shall be placed on the same rafter alignment to provide a continuous tie across the building.