This is a template document intended to ensure complete and consistent reports. Use of this template is mandatory for application to the SFP Seismic Mitigation Program, per DSA Procedure 08-031.

The purpose of this evaluation report is to establish eligibility for retrofit funding under Proposition 1D (AB 127, 2006). It is not the intent of this evaluation to provide a complete Life Safety evaluation. The evaluation is complete when eligibility has been determined.

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

In addition provide the following supporting documentation as applicable and use the following references:

1. Eligibility check summary
   - Appendix A.1. Structural calculations
2. Evaluation process
   - Appendix A.2. Evaluation statement notes
3. Site and building description
   - Appendix A.3 Photographs and details
4. Deficiency list
5. ASCE 31 Evaluation statements

MHP, Inc.

SE Firm Name (Logo optional)
SE Address: 3900 Cover St.
Phone: 562-985-3200
Terry Fernandez
Name of SE whose stamp is above

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 = 1.698

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

---

1 “DSA Procedure 08-03,” California Department of General Services, Division of the State Architect, latest edition.

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

---

SE Firm Name: MHP, Inc.
SE Firm Address: 3900 Cover St., Long Beach, CA 90808
SE Firm Phone #: 562-985-3200

---

PR 08-03
SMP Template (iss 09-15-11)
(errata 10-11-11)
ELIGIBILITY EVALUATION REPORT

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>School Campus:</td>
<td>Horace Mann School</td>
<td>Last Revision Date:</td>
</tr>
<tr>
<td>School Address:</td>
<td>8701 Charleville Blvd., Beverly Hills, CA 90211</td>
<td></td>
</tr>
<tr>
<td>Building Name/ID:</td>
<td>Building A</td>
<td></td>
</tr>
<tr>
<td>Project Tracking No.:</td>
<td>64311-39</td>
<td></td>
</tr>
</tbody>
</table>

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
- 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 *

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\(^2\), 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.\(^3\)
- 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 a 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.


o 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.

o Except for cracks in certain concrete members, Condition of Materials evaluation statements related to existing cracks are omitted.

o Beam Bars: The requirement for 25 percent of the joint bars to be continuous for the length of the member is removed.

o Redundancy (Moment frame and Braced frame): The requirement for two bays per frame line is removed.

o Stiffness of wall anchors: The limitation of 1/8-inch gap prior to anchor engagement is removed.

o Overturning: This statement is removed.

o 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.
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>1/24/1929</td>
<td>Beverly Hills School No. 4, Roy Seldon Price, Edward Cray Taylor, Ellis Wing</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Taylor Associated Architects, Sheets E1 through E8, Original Construction</td>
</tr>
<tr>
<td>2</td>
<td>1/15/1999</td>
<td>Modernization of Beverly Hills Unified School District Horace Mann School,</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Langdon Wilson Architects, Sheets ST-1 through ST-4 and S-2.1.1, S-2.4.1, S-4.1,</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Retrofit drawings A# 03-101858</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.

- Date of site visit: October 24, 2008, June 19, 2009
- Visiting engineer(s) and staff: Lance Kenyon, S.E., Dan Fox (respectively)
- Site visit authorized by: Brian Daugherty of Daugherty & Daugherty Architects

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**
- **GROUND, 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 (UNDERSIDE ONLY)**
- **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**
- **OTHER:**

The site visit confirmed that the existing structure generally conforms to the available drawings listed in Section 2.3, with the following exceptions:

<table>
<thead>
<tr>
<th>SET ID</th>
<th>CONDITION SHOWN ON PLANS</th>
<th>CONDITION OBSERVED AT SITE VISIT</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td></td>
<td>No exceptions noted.</td>
</tr>
<tr>
<td>2</td>
<td></td>
<td>No exceptions noted.</td>
</tr>
</tbody>
</table>
## Site and Building Description

### 3.1 Building description

**General**

- **Year originally built:** 1929
- **DSA Application number:** N/A
- **Original Work done pursuant to the Construction Garrison Act (Ed Code 17367):**
- **Number of stories above grade:** 2
- **Number of stories below grade:** 1 (partial)
- **Total floor area (sq ft, approx):** 34,700 sf at first floor, 8,500 sf at second
- **Other essentially identical buildings on this campus?** Yes ☑️ No

**Photographs**

Exterior elevation photograph, looking, Northeast : 6/19/2009
<table>
<thead>
<tr>
<th>Item</th>
<th>Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>School District</td>
<td>Beverly Hills Unified School District</td>
</tr>
<tr>
<td>School Campus</td>
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<td>8701 Charleville Blvd., Beverly Hills, CA 90211</td>
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<td>Building A</td>
</tr>
<tr>
<td>Project Tracking No.</td>
<td>64311-39</td>
</tr>
<tr>
<td>Original Report Date</td>
<td>12/20/13</td>
</tr>
<tr>
<td>Last Revision Date</td>
<td></td>
</tr>
</tbody>
</table>

**Exterior elevation photograph, looking, West taken: 6/19/2009**
BUILDING A OVERAL FIRST FLOOR PLAN CONFIGURATION
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:</td>
<td>complete as appropriate</td>
<td>☒</td>
</tr>
<tr>
<td>Mechanical / Utility Rooms or Enclosures</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: 34.06531
Longitude: -118.38208

Site Class per ASCE 31, Section 3.5.2.3: D

Basis for Site Class determination: Default Properties

<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>$S_x = 1.698$</td>
<td>$F_a = 1.0$</td>
<td>$S_{DS} = (2/3) S_x F_a = 1.132$</td>
<td>$S_{a,0.2} = S_{DS} = 1.132$</td>
</tr>
<tr>
<td>1.0</td>
<td>$S_I = 0.652$</td>
<td>$F_v = 1.5$</td>
<td>$S_{DI} = (2/3) S_I F_v = 0.652$</td>
<td>$S_{a,1.0} = \min (S_{DS}, S_{DI}/T) = 1.132$</td>
</tr>
</tbody>
</table>
3.4 Gravity System

Roof diaphragm and framing: Plywood sheathing, dimensional lumber trusses.

Typical floor diaphragm and framing: Diagonal sheathing over 2x purlins and steel trusses at classrooms, concrete pan joist at corridor.

Ground floor framing: Diagonal sheathing over 2x purlins and steel trusses.

Vertical load-bearing elements: Cast-in-place concrete walls and columns, typical.

Basement walls: Cast-in-place concrete walls, typical.

Foundation: Conventional shallow concrete cast-in-place footings, typical.

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>North-South</th>
<th>East-West</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1</td>
<td></td>
</tr>
<tr>
<td>C1B*</td>
<td></td>
</tr>
<tr>
<td>C2A</td>
<td></td>
</tr>
<tr>
<td>C2A</td>
<td></td>
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<tr>
<td>C2A</td>
<td></td>
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<tr>
<td>S1B*</td>
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<tr>
<td>S3</td>
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<tr>
<td>URM</td>
<td></td>
</tr>
<tr>
<td>URM</td>
<td></td>
</tr>
<tr>
<td>M*</td>
<td></td>
</tr>
<tr>
<td>None of the above</td>
<td></td>
</tr>
</tbody>
</table>

* 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).

For each item below, give a brief response or description.

- Horizontal system combinations
  - Combination at second floor – concrete slab diaphragm at corridors, diagonal sheathing at classrooms. Both span between concrete shear walls and piers.
Vertical system combinations
No combinations – concrete shear walls and piers.

SFRS foundation
Conventional shallow concrete footings.

Gravity loading
All concrete walls carry gravity loads and function as shear walls.

System details
Plywood roof sheathing and shear transfer added during limited retrofit in early 2000’s (no wall anchorage retrofit). Diagonally sheathed diaphragm at second floor with limited to no shear transfer to supporting concrete shear walls or trusses. Wall anchorage at second floor consists of cast-in ½” diameter rods with approximately 4” clear to surface of concrete at 4 feet on center, run through each truss top chord. Similar condition at ground floor.

Structural materials
No material strengths listed on original construction drawings. Default material properties assumed from ASCE 31 section 2.2. Concrete f’c = 2000 psi, reinforcing steel Fy = 33 ksi.

Original design code
Assumed 1927 UBC.

History of seismic retrofit or significant alteration
Significant alteration/retrofit in 1999 (set ID 2). Addition of concrete shear walls and braced frames at select locations on ground floor level. Addition of plywood sheathing to existing wood trusses at roof level to provide shear transfer from roof diaphragm to corridor concrete shear walls at second floor. Plywood roof sheathing observed during site observation appears to have been added prior to this retrofit.

Benchmark year check
Not a benchmark building.
4. Deficiency list

The following table summarizes the potential deficiencies identified in Section 5 of this report.

Other deficiencies might exist. The evaluation was stopped once critical deficiencies were identified.

<table>
<thead>
<tr>
<th>Non-compliant condition</th>
<th>Discussion</th>
<th>Additional evaluation recommended</th>
</tr>
</thead>
<tbody>
<tr>
<td>LOAD PATH</td>
<td>Raised first floor and second floor diaphragms have no shear transfer path to exterior shear and foundation walls. Extent of load path deficiency is widespread at first and second floor levels. Diaphragm loads are resolved by imposing an out of plane force on exterior walls, as load cannot be transferred to shear walls.</td>
<td></td>
</tr>
<tr>
<td>WALL ANCHORAGE</td>
<td>Wall anchorage at second floor classrooms and roof above first floor classrooms is deficient, particularly at hip framing. No Subdiaphragm exists to transfer wall out-of-plane forces to continuous diaphragm cross-ties. See Appendix A.3.</td>
<td></td>
</tr>
<tr>
<td>None identified</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
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.

| C | NC | U | NA | 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. |
| C | NC | U | NA | 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. |
| C | NC | U | NA | 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. |
| C | NC | U | NA | 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. |
| C | NC | U | NA | 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. |
| C | NC | U | NA | Masonry Units | There shall be no evidence of or reason to suspect structural capacity loss due to deterioration of masonry units. |
| C | NC | U | NA | 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. |
| C | NC | U | NA | Masonry Lay-up | Filled collar joints of multi-wythe masonry infill walls shall have negligible voids. |
**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.

**BUILDING CONFIGURATION**

<table>
<thead>
<tr>
<th>Critical Item</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>NC U NA</td>
<td>LOAD PATH. 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. <strong>Set ID 1, Sheet E7, details titled “TYPICAL BEARING IN WALL” and “TYPICAL SECTION”. See photos. No positive connection between floor diaphragm and shear walls or steel trusses. First floor diaphragm lateral forces will load the perimeter concrete walls out of plane.</strong></td>
</tr>
<tr>
<td>NC U NA</td>
<td>WEAK STORY. 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>NC U NA</td>
<td>SOFT STORY. 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>NC U NA</td>
<td>GEOMETRY. 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>NC U NA</td>
<td>VERTICAL DISCONTINUITIES. All vertical elements of the seismic force-resisting system shall be continuous to the foundation.</td>
</tr>
<tr>
<td>NC U NA</td>
<td>MASS. 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>NC U NA</td>
<td>TORSION. 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>NC U NA</td>
<td>ADJACENT BUILDINGS. 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.</td>
</tr>
<tr>
<td>NC U NA</td>
<td>MEZZANINES. 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

**Critical Item**

<table>
<thead>
<tr>
<th>C</th>
<th>NC</th>
<th>U</th>
<th>NA</th>
</tr>
</thead>
</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>
<tr>
<td>AXIAL STRESS CHECK (Concrete columns).</td>
<td>The axial stress due to gravity loads in columns subjected to seismic overturning forces shall be less than 0.10$f'_c$. 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$f'_c$.</td>
</tr>
<tr>
<td>AXIAL STRESS CHECK (Steel columns).</td>
<td>The axial stress due to gravity loads in steel columns subjected to seismic overturning forces shall be less than 0.10$F_y$. 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 0.30$F_y$.</td>
</tr>
<tr>
<td>FLAT SLAB FRAMES.</td>
<td>The seismic force-resisting system shall not be a frame consisting of columns and a flat slab/plate without beams.</td>
</tr>
<tr>
<td>PRESTRESSED FRAME ELEMENTS.</td>
<td>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.</td>
</tr>
<tr>
<td>CAPTIVE COLUMNS.</td>
<td>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.</td>
</tr>
<tr>
<td>NO SHEAR FAILURES.</td>
<td>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.</td>
</tr>
<tr>
<td>STRONG COLUMN/WEAK BEAM.</td>
<td>The sum of the moment capacity of the columns shall be 20% greater than that of the beams at concrete frame joints.</td>
</tr>
<tr>
<td>STRONG COLUMN/WEAK BEAM.</td>
<td>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.</td>
</tr>
<tr>
<td>BEAM BARS.</td>
<td>At least two longitudinal top and two longitudinal bottom bars shall extend continuously throughout the length of each frame beam.</td>
</tr>
<tr>
<td>COLUMN BAR SPLICES.</td>
<td>All column bar lap splice lengths shall be greater than 35$d_b$, and shall be enclosed by ties spaced at or less than 8$d_b$. 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.</td>
</tr>
</tbody>
</table>
### 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.

### STIRRUP SPACING

All beams shall have stirrups spaced at or less than $d/2$ throughout their length. At potential plastic hinge locations stirrups shall be spaced at or less than the minimum of $8d_b$ or $d/4$.

### JOINT REINFORCING

Beam-column joints shall have ties spaced at or less than $8d_b$.

### COMPLETE FRAMES

Concrete frames that are not part of the seismic force-resisting system shall form a complete gravity load carrying system.

### DEFLECTION COMPATIBILITY

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.

### FLAT SLABS

Flat slabs/plates that are not part of the seismic force-resisting system shall have continuous bottom steel through the column joints.

### RECONFIGURABLE WALLS

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.)

### PRECAST CONNECTION CHECK

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

### PRECAST FRAMES

For buildings with concrete shear walls, precast concrete frame elements shall not be necessary as primary components for resisting seismic forces.

### PRECAST CONNECTIONS

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.

### DRIFT CHECK

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.

### MOMENT-RESISTING CONNECTIONS

All moment connections shall be able to develop the strength of the adjoining members or panel zones.

### PANEL ZONES

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.
### COLUMNS SPLICES
- All column splice details located in moment-resisting frames shall include connection of both flanges and the web.

### COMPACT MEMBERS
- All frame elements shall meet section requirements set forth by Table I-9-1 of Seismic Provisions for Structural Steel Buildings (AISC, 1997).

### SHEAR WALLS

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

#### 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 $2\sqrt{f'_{c}}$.

#### REINFORCING STEEL
- In concrete or precast shear walls, 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. The spacing of reinforcing steel shall be equal to or less than 18 inches.
  
  Set ID 1, sheet E1, GENERAL NOTES, commonly 12” thick walls w/ #3 at 18” o.c. EWEF, ratio = 0.0010 < 0.0015 < 0.0025.

#### 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.

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

#### 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.)

#### 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.)

#### 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 $2\sqrt{f'_{c}}$. 

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**SE Firm Address:** 3900 Cover St., Long Beach, CA 90808  
**SE Firm Phone #:** 562-985-3200
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.

Set ID 1, sheet 3. Wall openings at perimeter constitute > 75% of wall length.

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.

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.

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.

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

Critical Item

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.

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

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

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

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

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

DIAPHRAGMS

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

CROSS TIES. There shall be continuous cross ties between diaphragm chords.
## ROOF CHORD CONTINUITY

All roof chord elements shall be continuous, regardless of changes in roof elevation.

## 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.

## OPENINGS AT BRACED FRAMES

Diaphragm openings immediately adjacent to the braced frames shall extend less than 25% of the frame length.

## OTHER DIAPHRAGMS

The diaphragm shall not consist of a system other than wood, metal deck, concrete or horizontal bracing.

## TOPPING SLAB

Precast concrete diaphragm elements shall be interconnected by a continuous reinforced concrete topping slab.

## STRAIGHT SHEATHING

All straight sheathed diaphragms shall have aspect ratios less than 2 to 1 in the direction being considered.

## SPANS

All wood diaphragms with spans greater than 24 ft shall consist of wood structural panels or diagonal sheathing.

## 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

#### 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.

At corners/hips of roof framing at high and low roofs no Subdiaphragm exists to transfer wall out-of-plane forces into the diaphragm. Subdiaphragm aspect ratios exceed 5.1 > maximum allowed ratio of 3.0.

#### WOOD LEDGERS

The connection between the wall panels and the diaphragm shall not induce cross-grain bending or tension in the wood ledgers.

#### PRECAST PANEL CONNECTIONS

There shall be at least two anchors from each precast wall panel into the diaphragm elements.

#### 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.)

C  NC  U  NA  |  CORBEL BEARING.  If precast concrete frame girders bear on column corbels, the length of bearing shall be greater than 3”.

C  NC  U  NA  |  CORBEL CONNECTIONS.  Precast concrete frame girders shall not be connected to corbels with welded elements.

C  NC  U  NA  |  TRANSFER TO SHEAR WALLS.  Diaphragms shall be connected for transfer of loads to shear walls.

    Set ID 1, Sheet E7, detail titled “TYPICAL BEARING IN WALL” and Set ID 1, Sheet 6 “TYPICAL TWO-STORY WALL SECTION”. No shear transfer bolts provided between diaphragm and wall (bearing only). Lateral forces will load perimeter walls out of plane.

C  NC  U  NA  |  TRANSFER TO STEEL FRAMES.  Diaphragms shall be connected for transfer of loads to the steel frames.

C  NC  U  NA  |  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.

C  NC  U  NA  |  CONCRETE COLUMNS.  All concrete columns shall be doweled into the foundation.

C  NC  U  NA  |  FOUNDATION DOWELS.  Wall reinforcement shall be doweled into the foundation.

C  NC  U  NA  |  PRECAST WALL PANELS.  Precast wall panels shall be connected to the foundation.

C  NC  U  NA  |  UPLIFT AT PILE CAPS.  Pile caps shall have top reinforcement and piles shall be anchored to the pile caps.

C  NC  U  NA  |  STEEL COLUMNS: The columns in lateral-force-resisting frames shall be anchored to the building foundation.

C  NC  U  NA  |  WALL PANELS: Metal, fiberglass or cementitious wall panels shall be positively attached to the foundation.

C  NC  U  NA  |  ROOF PANELS: Metal, plastic, or cementitious roof panels shall be positively attached to the roof framing to resist seismic forces.
<table>
<thead>
<tr>
<th>Component</th>
<th>Status</th>
<th>Notes</th>
<th>Details</th>
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<tr>
<td>WALL PANELS</td>
<td>C</td>
<td>NC</td>
<td>Metal, fiberglass or cementitious wall panels shall be positively</td>
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<td></td>
<td></td>
<td></td>
<td>attached to the framing to resist seismic forces.</td>
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<tr>
<td>FOUNDATION</td>
<td>C</td>
<td>NC</td>
<td>POLE FOUNDATIONS. Pole foundations shall have a minimum embedment depth</td>
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<tr>
<td></td>
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<td>of 4 ft.</td>
</tr>
<tr>
<td>TIES BETWEEN FOUNDATION</td>
<td>C</td>
<td>NC</td>
<td>ELEMENTS. The foundation shall have ties adequate to resist seismic</td>
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<td></td>
<td></td>
<td></td>
<td>forces where footings, piles, and piers are not restrained by beams,</td>
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<td></td>
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<td></td>
<td>slabs, or soils in Site Class A, B, or C.</td>
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### GEOLOGIC SITE HAZARDS

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

SE Firm Name: MHP, Inc.
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Appendices

A.3 Photographs and details

PLAN VIEW OF ROOF FRAMING – CRITICAL WALL ANCHORAGE LOCATIONS WITH HIP ROOF FRAMING
TYPICAL PLAN VIEW OF HIP ROOF FRAMING –
NON-CONFORMING SUBDIAPHRAGM ASPECT RATIO

12" CONCRETE WALLS BELOW

SUBDIAPHRAGM SPAN = 23'-0"

SUBDIAPHRAGM DEPTH = 4'-6"
SECTIONS AT STEEL TRUSS/SHEAR TRANSFER
FROM ORIGINAL CONSTRUCTION DOCUMENTS, SET ID 1
# ELIGIBILITY EVALUATION REPORT

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<tr>
<td>School Address:</td>
<td>8701 Charleville Blvd., Beverly Hills, CA 90211</td>
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<tr>
<td>Building Name/ID:</td>
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<tr>
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</table>

PHOTO 1 OF EXISTING FLOOR LOAD PATH
(Note: no shear transfer between diaphragm and steel truss)
PHOTO 2 OF EXISTING FLOOR LOAD PATH
(Note: screed clips shown)
<table>
<thead>
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<td>Page 28 of 29</td>
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<tr>
<td>Building Name/ID:</td>
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<td>64311-39</td>
<td></td>
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</tbody>
</table>

PHOTO 3 OF EXISTING FLOOR LOAD PATH
(Note: condition at bearing wall)