Beverly Hills Unified School District

MEMORANDUM

Date: April 28, 2015

To: Board of Education

From: Tim Buresh

Regarding: BHUSD CAPITAL IMPROVEMENTS PROGRAM

Subject: AUDITORIUM CLOSURE AND BUILDING RISK

The Temporary Auditorium Closure and Building Risk

The Board has directed the temporary discontinued use of Peters, Hawthorne and El Rodeo Auditoriums Facilities pending receipt letters from architects or engineers regarding the relative safety or danger in their continued use. This has raised the greater question of the safety or danger in the continued use of the buildings surrounding the auditoriums and other structures across the District. Attached letters from the following firms regarding their particular area of work in the program:

- Horace Mann - WLC Architects (incorporating the professional opinion of their structural engineer subconsultant into their opinion as lead architect)
- BHHS - DLR Group (incorporating the professional opinion of their structural engineer subconsultant Saiful Bouquet in addition to their opinion as lead architect)
- BHHS – Saiful Bouquet Structural Engineers
- Hawthorne - DLR Group (incorporating the professional opinion of their structural engineer subconsultant Saiful Bouquet in addition to their opinion as lead architect)
- Hawthorne – Saiful Bouquet Structural Engineers
- El Rodeo – HMC Architects (incorporating the professional opinion of their structural engineer subconsultant Kanda Tso Associates Consulting Structural engineers).
- El Rodeo – Kanda Tso Associates Consulting Structural Engineers
- Overall District - Kenney GeoScience

There is no letter for Beverly Vista as there are no apparent structural safety risks present at that site based on detailed review by MHP Structural Engineers or subsequent observation.

These letters should be read in the context of additional work done by MHP Structural Engineers for the District in 2007-2008. That work included an over seismic risk assessment for all buildings on all campuses. Two formal reports were issued: one for BHHS alone, and one for the four k-8 schools. Redacted and highlighted copies of those reports are attached. It must be noted that the MHP report addressing the k-8 schools formally ranked and categorized all of the District educational buildings according to an industry standard risk model. Numerous buildings were categorized as: "Risk Category 1 – Building appears to have a significant life-safety hazard – Level of Risk Highest" including:

- El Rodeo Building A
- Hawthorne Buildings A, B, C and D
- Horace Mann Buildings A, B and C
- BHHS Building A, B, E, F and H

The report dealing with BHHS did not apply the same hazard ranking system and is limited to noting a series of serious structural deficiencies in BHHS Buildings A, B (including B1, B2, B3 and B4). How serious? Several of the structural components failed to meet even the most minimal loading required by Code.

The MHP work and its formal conclusions stand today. Their work has been generally confirmed by more extensive analysis field research and the destructive testing program. Additional deficiencies have been noted by the District's current designers.

The structural issues must also be noted in the geologic context. The District has a greater understanding of the background seismic risk posed by the area. As noted on the Kenney GeoScience letter, that risk is elevated in areas under the District schools.

It is the recommendation of staff, supported by the opinions expressed in the letters received from current design and geology team, that continued occupancy of those buildings listed above poses a safety risk to the students and staff of the District. It is staff's continued recommendation that the buildings listed above be unoccupied as soon as possible regardless of timing of the successor renovation contracts. It is staff's further conclusion that continued occupancy of the Peters, Hawthorne and El Rodeo auditoriums poses an avoidable and unacceptable risk to the safety of staff and students and that the temporary closure should be made permanent.

ARCHITECTS CLIENT FOCUSED. PASSION DRIVEN.

WLC

April 27, 2015

Mr. Tim Buresh
Prime Source Management
Beverly Hills Unified School District
255 South Lasky Drive
Beverly Hills, CA 90212-3644

Re: The Future of Building A
Horace Mann School Modernization
Project 1110104.10

Dear Mr. Buresh:

Thank you for your inquiry about the future status and safety of Building A at the Horace Mann School. I hope that this letter helps to clarify the choices, challenges, and opportunities that are currently in front of the Beverly Hills Unified School District.

Campus Master Plan:

The HMS campus master plan calls for Building A to ultimately be completely remodeled. WLC received a DSA stamped approval on a comprehensive renovation project just last year (2014). The phasing plan for the campus reconfiguration calls for a new Classroom Building B, which is currently under construction, and scheduled to be complete by summer 2016. Building B will become the permanent home for virtually all of the instructional spaces on the site. Only the Kindergarten grade level will <u>not be</u> housed in Building B.

The master plan also calls for Building A to be closed, and then renovated, sometime after the completion of Building B. The current functions housed within Building A (office, classrooms, labs and food service) are planned to be temporarily relocated into the new Building B as well as to the presumed to be still standing “Rotunda” Building and Middle School Building.

Following the renovation of Building A the Rotunda and Middle School Buildings are scheduled to be completely demolished. The final phase of construction would be the construction of a new playground on the site of the Rotunda and Middle School Building.

Timeline:

Since Building B is now under construction the ‘wheels have been set in motion’ to complete the above described series of master plan steps. Since Building B consumes virtually the entire western half of the property it is very difficult to change course now. This is not to say, however, that the next steps (Building A’s renovation and the Rotunda/Middle School demolition) have to <u>immediately</u> follow the occupancy of Building B. It should be noted however that the sheer size of Building B will severely limit the exterior playground space for the school unless the remaining phasing steps are eventually carried out to their logical conclusion.

Building A:

Integral to the campus master plan and the timeline is the renovation of Building A. This is where the plan gets complicated. WLC and our consulting engineers have prepared and submitted studies to the District and the Division of the State Architect which outline existing structural inadequacies with this approaching 100-year-old building. The building is in need of structural stiffening. While some work was done almost a decade ago, much remains to be completed. So much so, that the building has now been granted eligibility to receive state funding under the State's Seismic Safety Mitigation Program (SSMP). WLC is in the process of qualifying the recently DSA approved renovation plans with DSA in order that BHUSD can access that funding.

Just as critical as the structural issues are the functional and programmatic issues surrounding Building A. The building houses critically unique campus functions that simply cannot be replaced or permanently relocated into the other remaining structures. Those functions include the School Office, Library, and Food Service spaces. In short, these must be housed somewhere.

Rotunda and Middle School Buildings:

As described above, both the Rotunda and Middle Schools are scheduled to be completely demolished. They should not be retained for several reasons. First, they consume a large footprint of precious site acreage which has only been made more precious with the construction of Building B. Second, the Middle School Building sits above a covered parking layer which will be rendered redundant with the construction of the new subterranean parking level under Building B. Finally, the buildings' floor plans and their locations at the back of the school make them far from conducive to repurposing into any of the quasi-public functions that currently populate Building A. In short, now that Building B is well under construction these two buildings must eventually be demolished for the campus to properly function.

Options, Opportunities, and Challenges:

The construction of Building B and the suspect structural condition of Building A offer the District some opportunities and challenges. The District has the choice to move straight into the renovation of Building A following the completion of Building B (assuming funds will allow). This option is imminent since Building B will be complete in just over one year. But the District also has the option of pausing the phasing timeline and holding off on the Building A renovation. The second option offers two major challenges. First, the building has already been identified to have significant structural needs. And second, unless Building A is quickly renovated the HMS campus will be left with a severe <u>shortage</u> of outdoor play area but with a severe <u>surplus</u> of indoor classroom space.

Closing Building A is another option, however it cannot be done until Building B is complete. There is simply no available site area for temporary, interim housing. Closing Building A following the occupancy of Building B results in significant functional challenges for the school's long- and/or mid-term daily operations.

Recommendations:

WLC recommends that the Beverly Hills Unified School District continue down the path of project phasing that it started when Building B broke ground. It is very difficult to change course now. Should funds prove unavailable then the District and WLC should quickly commence work on other viable functional plans for the relocation of the Office, Library, Food Service, and Kindergarten programs elsewhere on the school. It is important to note that the plans for alternative relocations, even 'temporary,' will also require design time, and DSA processing and approvals, prior to construction commencing on site.

In conclusion, WLC <u>does not</u> recommend that Building A remain in its present state of suspect seismic structural repair for any significant period of time. The District needs to make a decision on its state and status as soon as practically possible. Time is clearly of the essence both for programmatic and structural reasons. Building A should be vacated and renovation work should begin immediately following the completion of Building B next summer (2016).

If you need further information or clarification on this issue, please let me know.

Sincerely,

JAMES P. DiCAMILLO
Architect, AIA
LEED™ AP
President, Principal

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April 27, 2015

Beverly Hills Unified School District
255 South Lasky Drive
Beverly Hills, California 90212

Re: Recommendation to Address Seismic Risk at Beverly Hills High School

Beverly Hills Unified School District Governing Board:

Per Board directive of April 14, 2015, DLR Group submits this response regarding the safety for continued occupancy of existing campus buildings at Beverly Hills High School.

DLR Group is the Architect of Record and Saiful Bouquet, Inc. (SBI, Inc.) is the Structural Engineer of Record for the Beverly Hills High School Modernization project. DLR Group's experience with California K-12 school design and Field Act standards includes large seismic retrofit projects of historic buildings at USC and UCLA, as well as the redesign of the Belmont Learning Center campus for LAUSD where we were hired to address seismic concerns unknown at the time of construction, and most recently, assessments and seismic mitigation of the historic buildings at LAUSD's Jordan High School. SBI is considered an industry expert in seismic structural assessment and design.

Our assessments involved the following BHHS campus existing buildings:

Building A	Main Classroom Building
Building B1	Domestic Science
Building B2	Old Main Classroom Wing and Administration
Building B3	Peters Auditorium
Building B4	Arts and Music
Building E	Konheim Athletics Building
Building F	Swim Gym
Building H	Moreno High/M&O

The DLR Group design team, including structural engineer Saiful Bouquet Inc., has been working on the Beverly Hills High School (BHHS) campus since 2012. We have made architectural and structural assessments which include:

- Review of assessments and reports provided by others including Seismic Risk Evaluation report prepared by *MHP Structural Engineers* dated 12/13/2007 contained in the BHUSD Master Plan 2008.
- Review of field test data provided by the District.
- SBI's development of a Preliminary Structural Seismic Study of Building B (August 2012)
- Development of the Seismic Mitigation Program (SMP) Eligibility Evaluation Reports.

Based on the findings in SBI's 2012 report and the MHP 2007 report, we are concerned that, in the event of a major earthquake, the overall occupant safety of BHHS campus buildings will be compromised, particularly with respect to fire resistance, hazardous materials, access compliance and structural stability. These concerns are further substantiated by the findings of the SMP eligibility evaluations and the field test data provided by the District.

It is important to note that the Division of the State Architect (DSA) has been engaged with our firms throughout the submission of eligibility reports for the SMP program to secure State matching funds under Proposition 1D. DSA determined that the buildings (with the exception of Buildings A and F) are eligible for the funding program. As defined in DSA Procedure 08-03, DSA's eligibility determination was premised on a finding that the deficiencies create "a high potential for local or global collapse" during a major earthquake. For the purpose of SMP evaluation, DSA has stated that "only collapse-prone conditions need to be identified."

As noted above, Building A does not meet the SMP criteria for funding eligibility. However, the gypsum slab roof diaphragm system is of concern due to its brittle nature and poor performance during strong ground shaking. DSA, while reviewing eligibility for SMP, agreed with this concern and is assisting the District with getting Building A to qualify for state Hardship Program funding. We also have concern with the presence of asbestos in Building A's steel plaster drywall which may become friable if disturbed by a seismic event.

In the 2007 Risk Evaluation Report for the District's K-8 schools, MHP identified relative risk categories for each building from 1 to 5 with 1 being the highest risk. MHP did not identify risk on the high school buildings as they did for K-8 schools. However, if the MHP rating system is applied to the high school using the same rationale and align the same deficiencies observed, then below is how SBI, Inc. would rate the buildings at Beverly Hills High School:

- Building A = 2 (in steel frame levels)
- Building B1 = 1
- Building B2 = 1
- Building B3 (Auditorium) = 1
- Building B4 (Arts & Music) = 1
- Building E = 1
- Building F = 3
- Building H = 1

We support the District's phasing plan to renovate classroom space as the first phase to begin as soon as possible as these renovations will correct significant deficiencies in buildings with the greatest student occupancy: Buildings A, B1, and B2.

The DLR Group/SBI, Inc. team urges the Governing Board to immediately implement action to mitigate the risk to life and property in all buildings with the highest risk rating of 1. Indefinite occupancy of these buildings as they await renovation in later phases is not prudent given the risk identified by the numerous sources cited above. DLR Group recommends evacuation of the buildings at highest risk, which includes the auditorium, as quickly as is practical.

Sincerely,

DLR Group

Brett A. Hobza, AIA, LEED AP
Principal

Saiful Bouquet, Inc.

Robert Hale Randall, S.E.
Principal

Encl: none

cc: Dr. Gary woods
Adrian O. Cohen

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April 23, 2015

Mr. Tim Buresh
PRIMESOURCE PROJECT MANAGEMENT
One Civic Plaza Drive, Suite #500
Carson, California 90745

Subject: Preliminary Structural Evaluation Summary - Auditorium
Beverly Hills High School
Beverly Hills Unified – Beverly Hills, CA

Dear Tim;

The Auditorium building at Beverly Hills High School is part of Building B that also includes Domestic Science, Classroom, Administration, Stage Craft, Arts & Music, Instrumental and Drama wings. As indicated in the *"Preliminary Structural Seismic Study"* report prepared by SBI for the campus buildings dated August 2012, we concluded that the Auditorium building did <u>not</u> satisfy the *Life-Safety* performance criteria according to ASCE-31 in its current condition. In addition to noting the non-conformance of the structural building components, the existing heavy plaster ceiling of the Auditorium was flagged as being un-braced (both laterally and vertically) and highly damageable if subjected to strong ground shaking. Our recommendation given in the report was to properly brace (retrofit) or replace the ceiling.

As directed by the District, the Auditorium was included in the submission to DSA for Proposition 1D funding eligibility under the State's Structural Seismic Mitigation Program (SSMP). As a result of DSA's review, they recognize that the building possess characteristics that make it vulnerable to preserving *life-safety* and thus have deemed the building eligible for the funding program.

It is our opinion that given the concerns with the ceiling coupled with the non-compliant structural building elements that can exacerbate the ceiling damage the ceiling in the auditorium is highly damageable if subjected to strong ground shaking. Accordingly, we recommend that the ceiling be replaced as appropriate.

We trust that this letter meets your current needs. Please contact us should you have any questions.

Sincerely,

SAIFUL/BOUQUET, INC.

Robert Hale Randall, S.E.
Principal

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April 27, 2015

Beverly Hills Unified School District
255 South Lasky Drive
Beverly Hills, California 90212

Re: Recommendation to Address Seismic Risk at Hawthorne School

Beverly Hills Unified School District Governing Board:

Per Board directive of April 14, 2015, DLR Group submits this response regarding the safety for continued occupancy of existing campus buildings at Hawthorne School.

DLR Group is the Architect of Record and Saiful Bouquet, Inc. (SBI, Inc.) is the Structural Engineer of Record for the Hawthorne School Modernization project. DLR Group's experience with California K-12 school design and Field Act standards includes large seismic retrofit projects of historic buildings at USC and UCLA, as well as the redesign of the Belmont Learning Center campus for LAUSD where we were hired to address seismic concerns unknown at the time of construction, and most recently, assessments and seismic mitigation of the historic buildings at LAUSD's Jordan High School. SBI is considered an industry expert in seismic structural assessment and design.

Our assessments involved the following Hawthorne School campus existing buildings:

Building A	Administration/Classrooms/Auditorium
Building B	Classrooms
Building C	Classrooms
Building D	Library/Classrooms/PE
Building E	Mechanical
Building F	Food Service/Cafeteria
Building G	Classrooms
Building H	Classrooms
Building J	Classrooms
Building K	Classrooms

The DLR Group design team, including structural engineer Saiful Bouquet Inc., has been working on the Hawthorne School campus since 2011. We have made architectural and structural assessments which include:

- Review of assessments and reports provided by others including Seismic Risk Evaluation report prepared by *MHP Structural Engineers* dated 12/13/2007 contained in the BHUSD Master Plan 2008.
- SBI's development of a Preliminary Structural Seismic Study dated July 2011.
- Development of the Seismic Mitigation Program (SMP) Eligibility Evaluation Reports.

Based on the findings in SBI's 2011 study and the MHP 2007 report, we are concerned that, in the event of a major earthquake, the overall occupant safety of Hawthorne campus buildings will be compromised, particularly with respect to fire resistance, hazardous materials, access compliance and structural stability. These concerns are further substantiated by the findings of the SMP eligibility evaluations.

It is important to note that the Division of the State Architect (DSA) has been engaged with our firms throughout the submission of eligibility reports for the SMP program to secure State matching funds under Proposition 1D. DSA determined that Buildings A, B and C are eligible for the funding program. As defined in DSA Procedure 08-03, DSA's eligibility determination was premised on a finding that the deficiencies create "a high potential for local or global collapse" during a major earthquake. For the purpose of SMP evaluation, DSA has stated that "only collapse-prone conditions need to be identified."

Building D does not meet the SMP criteria for funding eligibility. However, the structural deficiencies identified in SBI's 2011 Study and MHP's 2007 Evaluation are the basis for concern that Building D would perform poorly during strong ground shaking.

In the 2007 Risk Evaluation Report for the District's K-8 schools, MHP identified relative risk categories for each building from 1 to 5 with 1 being the highest risk. SBI, Inc. concurs with these rankings:

- Building A = 1
- Building B = 1
- Building C = 1
- Building D = 1
- Building E = 2
- Building F = 2
- Building G = 3
- Building H = 3
- Building J = 2
- Building K = 4

We support the District's phasing plan to renovate or replace classroom space as the first phase to begin as soon as possible as this phase will correct significant deficiencies in the buildings at greatest risk: Buildings A, B, C, and D.

The DLR Group/SBI, Inc. team urges the Governing Board to immediately implement action to mitigate the risk to life and property in all buildings with the highest risk rating of 1. Indefinite occupancy of these buildings as they await renovation or replacement is not prudent given the risk identified by the numerous sources cited above. DLR Group recommends evacuation of the buildings at highest risk, which includes the auditorium, as quickly as is practical.

Sincerely,

DLR Group	**Saiful Bouquet, Inc.**
Brett A. Hobza, AIA, LEED AP Principal	Robert Hale Randall, S.E. Principal

Encl: none

cc: Dr. Gary woods
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April 23, 2015

Mr. Tim Buresh
PRIMESOURCE PROJECT MANAGEMENT
One Civic Plaza Drive, Suite #500
Carson, California 90745

Subject: Preliminary Structural Evaluation Summary - Auditorium
Hawthorne Elementary School
Beverly Hills Unified – Beverly Hills, CA

Dear Tim;

The Auditorium building at Hawthorne Elementary School is part of Building A that also includes the North, South and West classroom wings with the Auditorium as the East wing. As indicated in the *"Preliminary Structural Seismic Study"* report prepared by SBI for the campus buildings dated July 2011, we concluded that Building A did not satisfy the *Life-Safety* performance criteria according to ASCE-31 in its current condition. The seismic performance concerns were centered on the North, South and West wings. With the exception to the clock tower, the overall perceived seismic performance of the Auditorium wing itself did not appear to have structural seismic concerns. However, the existing heavy plaster ceiling of the Auditorium was flagged as being un-braced (both laterally and vertically) and highly damageable if subjected to strong ground shaking. At the time of the report, the District had elected prior to replace the ceiling in lieu of retrofit as determined from outcome of DSA meeting (see below).

In a 2008 report, *Meyers Houghton Partners Structural Engineers (MHP)* prepared a comprehensive evaluation report similar to that of the 2011 SBI report using the same performance criteria. In that report, MHP stated that *"un-braced, suspended plaster ceiling occurs above the seating area of the auditorium (and possibly additional areas). During strong seismic shaking the plaster can break up and/or the suspension system can fail, creating significant falling hazards."* Their recommend was to *"remove and replace the heavy plaster ceiling above the seating area of the auditorium (and any other areas) with a lighter ceiling system. Alternatively, it may be feasible to isolate and brace the plaster ceiling to resist seismic forces."* Refer to attached excerpt from report. As a follow up to their report, MHP prepared a separate memorandum dated November 10, 2010 regarding the ceilings stating; *"These ceilings are a known collapse hazard under strong earthquake ground motion, and are very dangerous"*. In their memo they recommended retrofit or replacement as an option to address the concern. Refer to attached copy of memorandum.

On November 3, 2010, District representatives met with the Division of State Architect (DSA) to discuss the concern over the auditorium ceiling and to outline procedures for addressing the concern. As indicated in the meeting minutes; *"These heavy ceilings are vulnerable to collapse under strong ground motion and have been identified as perhaps the most significant seismic hazard for the District"*. Refer to attached copy of meeting minutes.

Given the aforementioned, it is our opinion as stated in our 2011 report that the ceiling in the auditorium is highly damageable if subjected to strong ground shaking. Accordingly, we recommend that the ceiling be replaced as appropriate.

We trust that this letter meets your current needs. Please contact us should you have any questions.

Sincerely,

SAIFUL/BOUQUET, INC.

Robert Hale Randall, S.E.
Principal

April 22, 2015
Revised: April 27, 2015

Mr. Tim Buresh
Interim Chief Facilities Officer
Beverly Hills Unified School District
255 South Lasky Drive
Beverly Hills, CA 90212

HMC Architects

Subject: Existing Buildings Structural Conditions
El Rodeo School

Dear Tim:

Please allow this correspondence to serve as a response to the Beverly Hills Unified School District's request for supplemental data that outlines the existing building condition deficiencies being evaluated at the El Rodeo School for the purposes of addressing potential deficiencies in the proposed Modernization Project.

For the past four years our firm and our consulting engineers have worked collaboratively with the District's staff and consultants to become familiar with the known and observable existing site and building conditions at this campus. This due diligence process has involved multiple site visitations and field recordations; reviews of Condition Assessment reports prepared by District consultants; field investigations and testing of existing building systems and materials, as well as the initiation of structural analysis of the existing buildings construction. The submittal of this geotechnical and structural analysis to the California Division of the State Architect (DSA) as part of the Seismic Mitigation Program outlined under the State's Procedural Regulation PR 08-03 (enclosed for reference) has also taken place, and the District has received notification from DSA that structural deficiencies exist at several buildings on the El Rodeo School campus. Additionally, our work efforts have sought to identify deficiencies in the campus' fire life safety and access compliance features that will be required to be brought into current California Building Code (CBC) compliance as part of the proposed Modernization project.

All of these assessment and design activities to date have been focused on initiating meaningful improvements to the campus that will upgrade the physical systems of the structures, improve the safety and operation of the buildings, and enhance the learning/teaching opportunities for the building's occupants. Prior presentations to the District's Facilities staff, District Leadership and Governing Board members have illustrated how the proposed Modernization work scopes at El Rodeo School would address these identified deficiencies.

One particular analysis that has occurred and is ongoing relates to the Seismic Mitigation Program previously referenced. At the El Rodeo campus, each of the five existing buildings (Buildings A, B, C, D, and E) were initially evaluated for eligibility into this supplemental funding program, based upon demonstrating to the Division of the State Architect that the buildings meet prescribed eligibility criteria.

These DSA criteria include providing evidence that the buildings pose an unacceptable risk of injury to its occupants due to building collapse as a result of ground shaking, faulting, liquefaction, or landslides. Buildings A, B, C, and D were identified by DSA as having a high risk of “Collapse Potential Due to Ground Shaking” (refer to attached Eligibility Evaluation Report – Checklist Items 1.3.1 and 1.3.3). The structural deficiencies identified in these Eligibility Evaluation Reports included weaknesses in wall to floor anchorage connections, shear wall stresses, vertical discontinuities, and unknown cast stone anchorage connections. A further detailed description of these risks and hazards is outlined in the April 27, 2015 Seismic Assessment letter prepared by Kanda and Tso consulting Structural Engineers that was submitted separately to the District.

The California Division of the State Architect has declared that these structural deficiencies in Buildings A, B, C, and D meet the criteria of the Seismic Mitigation Program and pose an unacceptable risk of injury to its occupants due to potential building collapse due to ground shaking. (Refer to attached concurrence letters from DSA.)

In our continued collaborations with the District’s Facilities leadership, Program Manager, and separate specialty consultants, we continue to identify the challenges of conducting phased construction activities in portions of the campus while maintaining the operations, safety and learning environments intact at the remainder of the school.

Based upon the structural analysis performed to date and the concurrences by DSA and the consulting Structural Engineer, we are in support of the current District plan to remove the occupants from all of the existing buildings and place the students, faculty, and staff into interim housing as soon as possible. By consolidating the construction activities into a single phase of construction, this will allow for a compressed overall duration of construction activities and provide a safer and more cost-efficient solution than could be achieved with a multi-phased work plan requiring multiple logistical and operational challenges to be overcome.

Please refer to the attached Seismic Mitigation reports, DSA letters of concurrence, and Procedural Regulations outlining the particular structural deficiencies of Buildings A, B, C, and D. Should the District have other questions pertaining to the SMP status of activities, proposed Modernization plan approval and construction timetables, or Interim Housing placement strategies, please do not hesitate to contact our office to discuss.

Sincerely,

HMC Architects

Dan Benner, AIA
Principal

Enclosures: DSA Procedural Regulation PR 08-03
SMP Eligibility Evaluation Reports for Buildings A, B, C, and D
DSA SMP Eligibility "Concurrence" Letters dated April 24, 2014

DB/lc

DSA PR 08-03

PROCEDURE: SCHOOL FACILITY PROGRAM/ SEISMIC MITIGATION PROGRAM

Purpose: This document sets forth the procedures to be followed by applicants seeking funding for seismic mitigation of eligible buildings under the Seismic Mitigation Program (SMP) for California K–12 public schools.

Background: The SMP is authorized by the Kindergarten-University Public Education Facilities Bond Act of 2006 (Proposition 1D) and School Facility Program (SFP) regulations (Title 2, California Code of Regulations, Section 1859.82(a)), and administered by the Office of Public School Construction (OPSC) on behalf of the State Allocation Board (SAB). Proposition 1D provided $199.5 million of state matching funds for seismic mitigation projects, and related ancillary costs, begun on or after May 20, 2006, that meet the eligibility requirements. SMP regulations can be found on the OPSC website.

NOTE: This procedure corresponds to the amended program regulations adopted by the SAB on June 22, 2011, and approved by the Office of Administrative Law on September 8, 2011.

Overview: The following is a brief summary of the steps and the required submittals for DSA review and approval:

Phase (Section)	Required Submittals	DSA Fee	References
1. Verify Eligibility	Eligibility Evaluation Report submitted to DSA headquarters.	$500	Eligibility Evaluation Report Template, ASCE/SEI 31-03
2. Replacement Option Analysis: (not required for Rehabilitation projects)	Structural Engineer's Report, Geotechnical Engineer's Report (if applicable), and Cost Estimate. DSA headquarters reviews the scope of work in the request.	None	Section 2
3. Seismic Rehabilitation Pre-Application (not required for replacement projects)	Evaluation and Design Criteria Report. Submit to the DSA regional office.	Initial fee of $2000 per building. Additional fees based on DSA review hours	ASCE/SEI 41-06, CBC Chapter 34, Title 24 Part 1 Section 4-306
4. Project Application	Construction Plans, Specifications, Calculations, Geohazard Report, and Cost Estimate submitted to the DSA regional office.	Standard plan review fee based on estimated construction cost (see Title 24, Part 1, Section 4-321)	Rehabilitation: ASCE/SEI 41-06, CBC Chapter 34 Replacement: CBC Chapter 16A
5. Seismic Mitigation Funding	See OPSC website for applicable requirements including OPSC Facility Hardship Checklist & form SAB 50-04.	Not Applicable	Section 5

1. PHASE 1 – VERIFY ELIGIBILITY: Only buildings meeting the following eligibility criteria may be funded under this program. The school district must submit a completed Eligibility Evaluation Report (Appendix D) to demonstrate the proposed building meets these eligibility criteria. If your district has an eligible building that was repaired or replaced prior to the issuance of this procedure, contact DSA headquarters for direction.

1.1 Building Occupancy: Indicate whether the building was designed for occupancy by students and staff by providing the DSA application number for the original construction, or the applicable DSA number for projects involving pre-Field Act buildings per Education Code, Section 17367.

1.2 Structural System: Describe the structural system, using the definitions in the *Seismic Evaluation of Existing Buildings (ASCE/SEI 31-03), American Society of Civil Engineers, 2003,* for guidance in determining the structural system. Provide structural framing plan layout drawings/sketches or copies of the structural framing plans used for the original construction. The type of structural system must be one of the following:

- C1 – Concrete Moment Frames
- C1B* – Reinforced Concrete Cantilever Columns
- C2A – Concrete Shear Walls, Flexible Diaphragm
- C3A – Concrete Frame with Infill Masonry Shear Walls, Flexible Diaphragm
- PC1 – Precast/Tilt-up Concrete Shear Walls, Flexible Diaphragm
- PC1A – Precast/Tilt-up Concrete Shear Walls, Rigid Diaphragm
- PC2 – Precast Concrete Frames with Shear Walls, Rigid Diaphragm
- PC2A – Precast Concrete Frames without Shear Walls, Rigid Diaphragm
- RM1 – Reinforced Masonry Bearing Walls, Flexible Diaphragm
- S1B* – Steel Cantilever Columns
- S3 – Steel Light Frames
- URM – Unreinforced Masonry Bearing Walls, Flexible Diaphragm
- URMA – Unreinforced Masonry Bearing Walls, Rigid Diaphragm
- M* – Mixed Systems - building containing at least one of the above lateral-force-resisting systems in at least one direction of seismic loading.

* 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).

1.3 Building Collapse Potential Due to Ground Shaking: Provide evidence that demonstrates that the building poses an unacceptable risk of injury to its occupants due to ground motions, as determined in ASCE/SEI 31-03. Additionally, describe in detail the specific deficiencies and reasoning for these conclusions for at least one potential collapse scenario. The ASCE/SEI 31-03, as amended per the Eligibility Evaluation Report Template, shall be used for the evaluation of the building performance level.

NOTE: If eligibility can be determined based on ground shaking, then it is not necessary to provide a geohazard report in this phase, as referenced in Section 1.4 of this procedure.

1.4 Building Collapse Potential Due to Faulting, Liquefaction, Landslides: If eligibility is based on the presence of faulting, liquefaction or landslide, a geologic analysis must be prepared and submitted to the California Geological Survey (CGS).

Refer to Appendix B for reporting of ground faulting, liquefaction and landslides, and consult with CGS (Jennifer Thornburg, 916-445-5488) prior to submittal of such reports to ensure a complete submittal. For additional information, please visit the CGS website at http://www.conservation.ca.gov/cgs/rghm/reviews/Pages/faq.aspx.

Submit the reports to the address below and include a reference to the Seismic Mitigation Program.

Attn: Margaret Hyland
California Geological Survey
School Review Unit
801 K Street, MS 12-32
Sacramento, CA 95814

CGS will provide a letter to the school superintendent and provide a copy to DSA and OPSC indicating whether or not CGS concurs with the characterization of the geologic hazard and expected magnitude of displacements.

The Eligibility Evaluation Report shall contain a structural analysis demonstrating a high potential for local or global collapse in the evaluation earthquake as a result of the displacements imposed on the structure due to the faulting, liquefaction, or landslide, as indicated in the CGS approved geohazard report. The structural analysis shall comply with California Building Code (CBC), Section 1604A.4. To ensure the analysis approach is acceptable, consult with DSA (contact below) prior to completing the evaluation report.

1.5 Submittal Requirements: The school district must submit a complete application form DSA-4, application fee ($500), and an Eligibility Evaluation Report to DSA headquarters (HQ).

Division of the State Architect
Attn: SMP Program
1102 Q Street, Suite 5100
Sacramento, CA 95811

The report must have the stamp or seal and signature of a California registered structural engineer.

The report and all related documents must be submitted in hard copy, accompanied by a CD containing all submitted documents.

Provide a separate application form DSA-4, application fee, and report for each building even if the buildings are similar or identical in design and construction.

1.6 DSA Review: Submittals will be reviewed within 10 working days of receipt of a complete submittal. If eligibility is based on the presence of faulting, liquefaction or landslide, the report will require additional review time by DSA and CGS. CGS concurrence must be obtained in order for DSA to issue a letter confirming building eligibility for SMP.

DSA HQ will send a letter to the applicant, with copies to the school district superintendent, district facilities director or appropriate contact, structural engineer, and OPSC, indicating whether or not DSA concurs that the building is eligible for funding.

1.7 Evaluation of Mitigation Options: Once the applicant receives confirmation from DSA that the building meets the eligibility criteria for SMP, the applicant may proceed with rehabilitation of the building, replacement of the facility on the same school site, or replacement on a new site. Prior to proceeding with project design, school districts and their design professionals are advised to review OPSC requirements for funding of SMP projects. These requirements include, among other items, a justification by the district of the unmet pupil housing need and a cost benefit analysis that determines the building's qualifications for rehabilitation or replacement funding. The project's eligibility for School Facility Program (SFP) replacement or rehabilitation funding may not match a school district's desired construction outcome. For more information, see OPSC website at www.dgs.ca.gov/opsc or contact OPSC (see Section 5 for OPSC contact information).

2. PHASE 2 – REPLACEMENT OPTION ANALYSIS: To ensure compliance with SFP regulations, Section 1859.82(a)(1), C.C.R., a school district seeking funding to replace an eligible building must demonstrate that the estimated cost of rehabilitation is equal to or greater than 50 percent of replacement value (replacement value is determined by OPSC in accordance with SFP regulations). SFP regulations also require DSA concurrence with the scope of the minimum work required to rehabilitate an eligible building. To obtain DSA concurrence, a school district must submit a structural engineer's report to DSA headquarters, to the address given in Section 1.5. The report must contain the following as applicable to the building deemed eligible in Phase 1:

- Detailed description of seismic deficiencies.
- Description of minimum work required to mitigate seismic deficiencies.
- Description of accessibility and fire and life safety upgrades. To determine applicable required work for fire and life safety and accessibility upgrades, refer to CBC Chapter 34 and Appendix C of this procedure.
- Schematic plans for the above work.
- Cost estimate for the above required work (summary cost estimate, i.e., square footage basis). Other work (including, but not limited to, repair, upgrades, or modifications, etc.) not required as a result of the seismic mitigation or applicable accessibility and fire and life safety work shall not be included in the cost estimate.

DSA will review the cost estimate to ensure that the required work is included, and will not review the estimate for actual cost of construction.

To ensure timely processing, the report must be accompanied by a cover letter requesting Phase 2 concurrence review. Include the Project Tracking Number (PTN) shown on the DSA-4 form submitted in Phase 1.

Upon review and concurrence, DSA will issue a letter to the applicant and provide a copy to the school district superintendent, facilities director, structural engineer, and OPSC. The school district may proceed to Section 4 below.

NOTE: For projects involving liquefaction or landslides as the geologic hazard contributing to the collapse potential of the eligible building, a geohazard report will be required to document the potential for building displacement and recommended site improvements to mitigate the hazard. Such report shall be submitted to CGS for review if the geohazard report submitted in the eligibility phase (refer to Section 1.4 above) did not address the selected mitigation measures.

For projects involving faulting as a hazard contributing to the collapse of a building, a geohazard report is not required as this hazard cannot be mitigated and the building must be replaced.

3. PHASE 3 – SEISMIC REHABILITATION OPTION: The approval of a rehabilitation plan is a two-step process that includes the filing of the pre-application and the project application. The pre-application will establish the criteria for evaluation and design, material testing and condition assessment requirements, and is described in this section. The project application, described in Section 4, will include the design development of construction plans, specifications, and calculations, using the criteria established in the pre-application.

Projects with an estimated cost of rehabilitation equal to or greater than 50 percent of replacement value will only qualify for replacement funding under the provisions of the SMP. It is advised that school districts contact OPSC for more information prior to submitting a Phase 3 approval request to DSA.

3.1 Pre-Application: The district must submit to DSA a pre-application, form DSA-1.REH, required fees in accordance with Title 24 Part 1, Section 4-326, and an Evaluation and Design Criteria Report per Title 24 Part 1, Section 4-306 and CBC Sections 3417.4, 3419, and 3423.1. The Evaluation and Design Criteria Report shall also include the proposed fire and life safety and accessibility criteria (see Appendix C).

3.2 Scope of Work: Rehabilitation projects funded by SMP shall be designed to meet the current CBC requirements for seismic rehabilitation. For the 2013 CBC, seismic rehabilitation shall be designed in accordance with Sections 3417 to 3423, utilizing the performance requirements in CBC Table 3417.5 for "public schools."

A seismic rehabilitation includes strengthening of all structural elements that do not comply with ACSE 41, Section 2.3.2 - *Systematic Rehabilitation Method* and is not limited to those deficient items found in the ASCE/SEI 31-03 analysis described in Section 1, above.

In addition, the seismic rehabilitation requires a full inventory, analysis, and strengthening, where required, of the non-structural components of the building in accordance with Section 11 in ASCE/SEI 41-06 utilizing the criteria in CBC Table 3417.5, and as outlined in CBC Section 3419.9. See Appendix C for applicable requirements for fire and life safety and accessibility.

Seismic rehabilitation projects under the SMP will be subject to a structural rehabilitation (wind and seismic force requirements) per Section 4-306 if alterations to the existing structural components, or additions of new structural components, exceed the limitation of Title 24, Part 1, Section 4-309(c) 2. The cost trigger for structural rehabilitation in Title 24, Part 1, Section 4-309(c)1 need not apply to seismic rehabilitation projects under the SMP since the cost of the seismic rehabilitation need not be included in this cost analysis. Conversely, the seismic rehabilitation costs shall be included in the Replacement Option Analysis in Section 2.

NOTE: A project consisting of repairs designed pursuant to only Section 3419.12, Part 2, Title 24 CCR (voluntary modifications to the lateral-force-resisting system) is not eligible for funding under the SMP. Only seismic rehabilitation in accordance with Section 3.2 of this procedure will be eligible for SMP funding.

3.3 **DSA Review:** Upon review and approval of the Evaluation and Design Criteria Report, DSA will date, sign, and stamp the report with the applicable REH application number. An REH application number is assigned to a project prior to the DSA application number to facilitate tracking of rehabilitation projects. The Evaluation and Design Criteria Report shall be used to prepare the project submittal, per Section 4.

4. **PHASE 4 – PROJECT APPLICATION:** To facilitate a complete application submittal to DSA, applicant school districts should contact the DSA regional office to schedule a design phase consultation. For rehabilitation projects, the meeting should include verification of certification of prior construction projects involving the eligible building(s), and the scope of fire and life safety and accessibility upgrades to be included in the project.

4.1 **DSA Submittal Requirements:** The submittal must include *Application for Approval of Plans and Specifications* (form DSA 1), *Project Submittal Checklist* (form DSA 3) along with all applicable documents, required fees in accordance with Title 24, Part 1, Sections 4-321 and 4-324, geohazard report in accordance with DSA IR A-4, construction plans, specifications, and the design phase meeting minutes (if applicable).

Replacement projects do not require an Evaluation and Design Criteria Report, per Section 3.1 above, as a prerequisite for submittal of plans.

The application package shall be submitted to the appropriate DSA regional office. DSA will assign a project application number.

4.2. **Rehabilitation Project Scope:** When an applicant school district wishes to expand the scope of the project beyond seismic rehabilitation, the project application must be submitted to DSA in increments per DSA IR A-11. One of the increments must contain only the work which is expected to receive state funds for seismic rehabilitation and associated required fire and life safety and accessibility upgrades. The other increment(s) must include work unrelated to seismic rehabilitation and associated required fire and life safety and accessibility. For the purposes of the SMP only, each increment need not be independently complete and code compliant per DSA IR A-11, Section 1.1, provided the combined increments are complete, code compliant, and are submitted and approved concurrently, and construction certification of each increment is tied to each other. Application submittal must include form DSA 1-INC, *Definition of Scope of Increments*, and a letter from the design professional in general reasonable charge stating that the scope of work in the seismic rehabilitation increment contains only the minimum work needed to mitigate the seismic deficiencies and associated required fire and life safety, and accessibility upgrades.

4.3 **DSA Review:** The DSA regional office will review the construction documents and, upon determining compliance with CBC requirements for school buildings, issue a Plan Approval letter. Consult the DSA regional office regarding expected timeline of review and approval of projects.

4.4 **OPSC Requirements:** SFP Regulation, Section 1859.82(a)(1), requires that project funding be limited to the work required to obtain DSA approval for the work required for the seismic rehabilitation and related fire and life safety, and accessibility upgrades. To fulfill this requirement, OPSC requires a school district to provide a letter of concurrence from DSA at the time of submittal of a funding application to OPSC.

A school district or its design professional in general responsible charge shall request the letter of concurrence from the DSA regional office after approval of the project plans and specifications. For project applications submitted in increments, the DSA concurrence letter will address the increment of the project containing seismic rehabilitation work and associated required fire and life safety, and accessibility upgrades.

5. **PHASE 5 – SEISMIC MITGATION FUNDING:** Upon receipt of DSA Plan Approval letter, the school district must forward a copy of the letter to OPSC as a part of its application for funding (form SAB 50-04), along with any other applicable documents.

Any questions related to funding available for the SMP, including eligibility for various grants and allowances, should be directed to the following OPSC staff members:

- Hannah Konnoff, Facility Hardship Program Analyst at hannah.konnoff@dgs.ca.gov or (916) 375-4037
- Dennis Schrader, Facility Hardship Program Analyst at dennis.schrader@dgs.ca.gov or (916) 375-4988
- Tasha Brennan, Facility Hardship Supervisor at tasha.brennan@dgs.ca.gov or (916) 375-4138

Appendix A: **Process Flow Charts**

Appendix B: **Documenting Geologic Hazards for SMP Projects**

Appendix C: **Guidelines for Determining Fire Life Safety and Accessibility Requirements**

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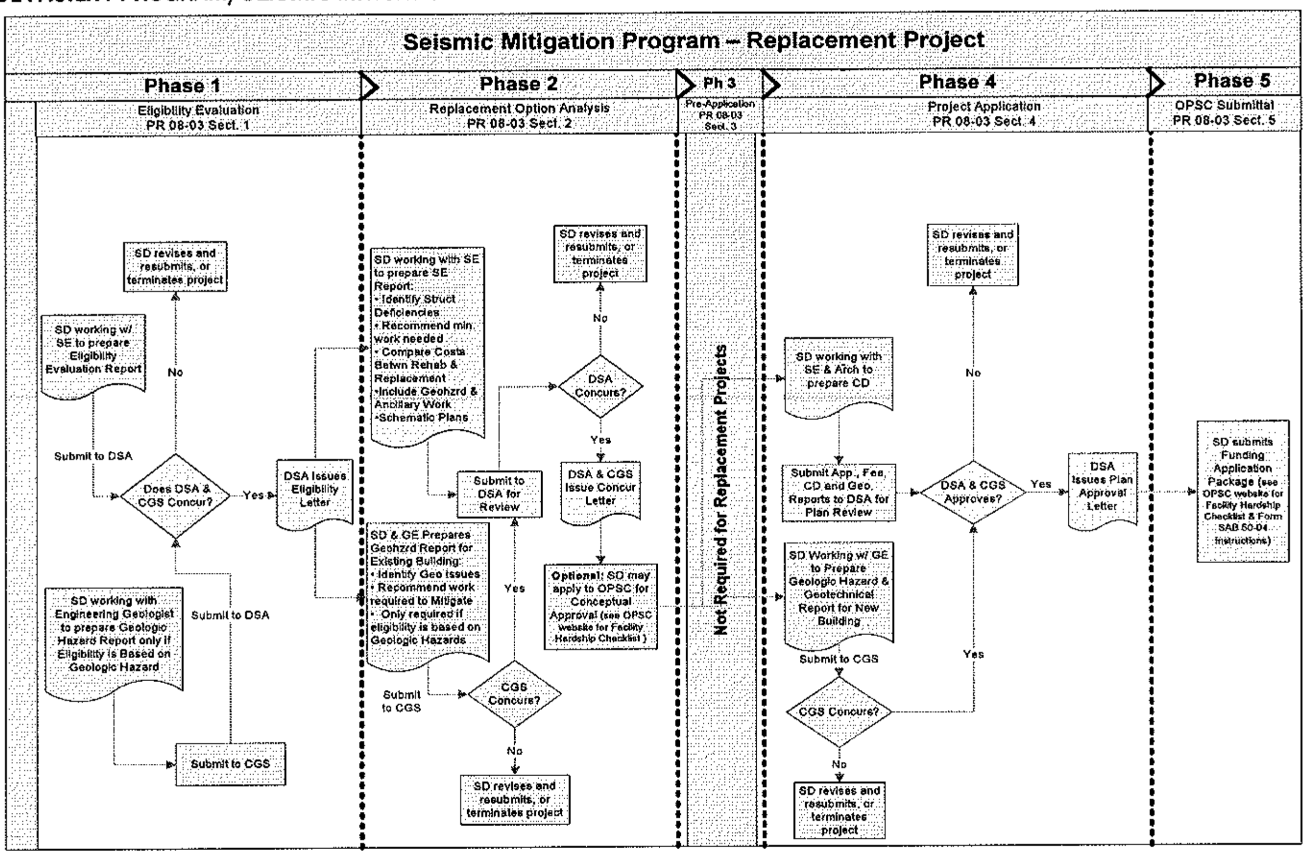
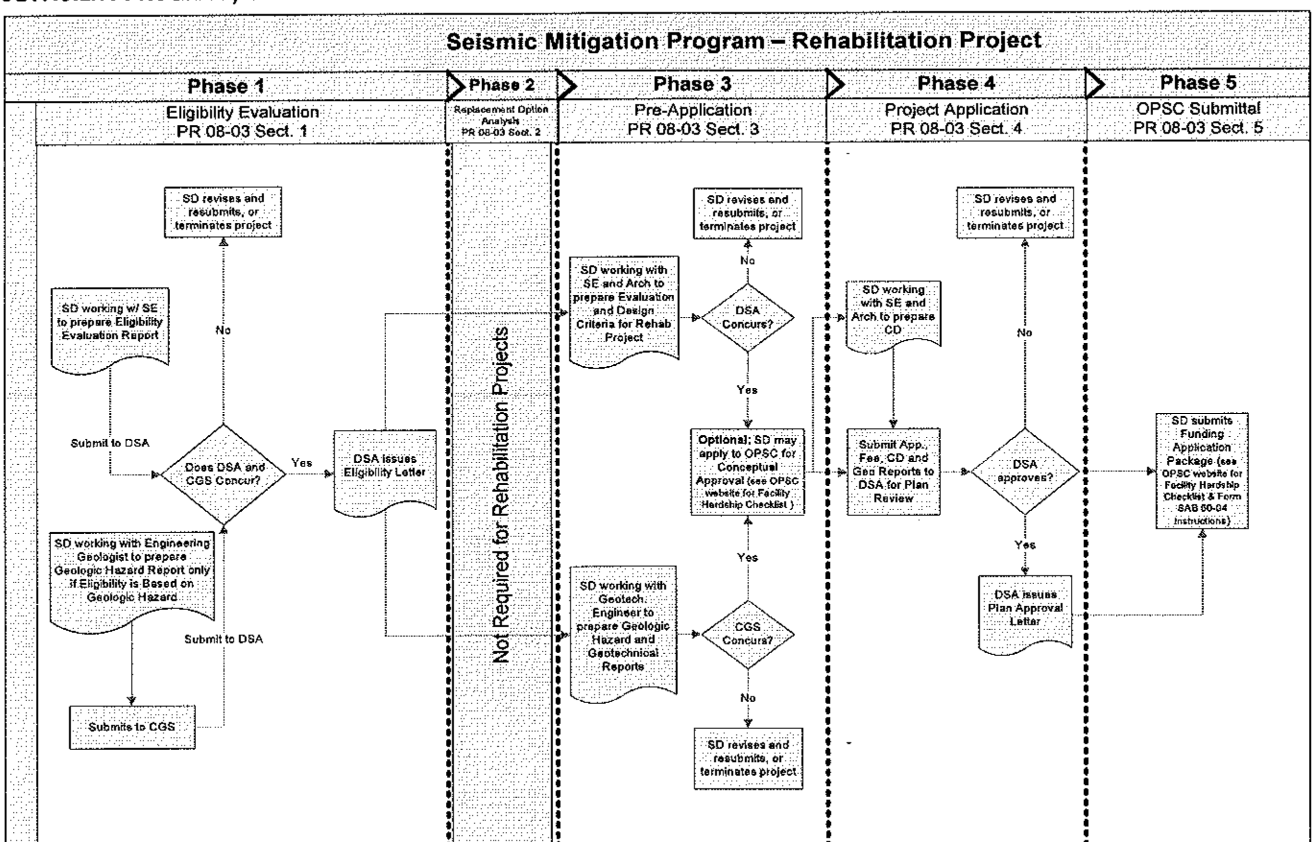


Key to Abreviations:
CD=Construction Document, CGS=California Geologic Survey, DSA=Division of the State Architect, GE=Geotechnical Engineer, OPSC=Office of Public School Construction, SAB=State Allocation Board, SD= School District SE= Structural Engineer

Disclaimer: This flowchart is intended as a general guide and tool for district use.



Key to Abreviations:
CD=Construction Document, CGS=California Geologic Survey, DSA=Division of the State Architect, GE=Geotechnical Engineer, OPSC=Office of Public School Construction, SAB=State Allocation Board, SD= School District, SE=Structural Engineer

Disclaimer: This flowchart is intended as a general guide and tool for district use.

Appendix B – Documenting Geologic Hazards for SMP Projects

Introduction: If eligibility for Proposition 1D funding is based on the presence of faulting, liquefaction or landslide, a geologic analysis must be prepared and submitted to CGS.

Procedure: For each building evaluated for SMP eligibility, provide evidence that the geologic hazard is present on the site, and provide the anticipated magnitude of surface displacement in accordance with the guidelines below. Displacement results must be sufficiently detailed for structural engineers to use in their analysis of structural performance. These analyses are not typical geotechnical engineering or engineering geology practice, and each project will be reviewed for scientific credibility on its own merit. Supporting site data must be presented and must be shown to be directly relevant to the structure being evaluated. Adequate scientific justification for all interpretations must be presented. Overly "conservative" approaches may result in unreasonably large estimates of displacement which, for this program, will be questioned by CGS.

Resources: See these documents for guidance (all are available online):

- California Geological Survey Note 48, 2013, Checklist for the Review of Engineering Geology and Seismology Reports for California Public Schools, Hospitals, and Essential Services Buildings (PDF – 95 KB).
- California Geological Survey, 2008, Guidelines for Evaluating and Mitigating Seismic Hazards in California, CGS Special Publication 117A (PDF – 1.24 MB).
- Martin, G.R. and Lew, M., 1999, Recommended Procedures for Implementation of DMG Special Publication 117: Guidelines for Analyzing and Mitigating Liquefaction in California; Southern California Earthquake Center (PDF – 2.13.MB).
- Blake, T.F. Hollingsworth, R.A., and Stewart, J.P., 2002, Recommended Procedures for Implementation of DMG Special Publication 117: Guidelines for Analyzing and Mitigating Landslide Hazards in California, Southern California Earthquake Center (PDF – 3.24 MB).
- California Geological Survey Note 49, 2002, Guidelines for Evaluating the Hazard of Surface Fault Rupture (PDF – 350 KB).

1. LIQUEFACTION: Engineering geologists or geotechnical engineers working for the school district as consultants should estimate displacement of ground surface assuming the site is subject to peak ground acceleration (PGA) calculated with the adjusted MCE_G peak ground acceleration (PGA_M) in accordance with ASCE 7-10, Section 11.8.3, and historical high ground water level. The consultants should show how PGA and ground-water parameters are derived. Adequate site-specific density data should be provided through boring logs, cone penetration test correlated with borings, or down-hole shear-wave velocity data. Vertical and lateral extent of liquefiable layers should be shown in geologic cross sections.

Show calculations to document one or more of the following failure mechanisms:

1.1 Loss of bearing capacity:

- **Report undrained residual bearing capacity** and analyze the potential for punching shear failure.

1.2 Lateral spread:

- Provide geologic cross section showing extent of lateral spread with respect to the building. Indicate if the building is on the margins of expected lateral spread, or if it lies within a recognizable coherent block.
- **Report vertical and lateral displacement at the location of the structure.**

1.3 Differential settlement:

- Using a factor of safety for liquefaction of 1.3, **report maximum differential settlement across the building footprint.**
- Actual differential settlement must be supported by two or more borings. Assumption of some fraction of total liquefaction settlement will not be accepted.
- Dry seismic settlement above the historical high ground-water level will not be considered for this program.

2. SEISMICALLY INDUCED LANDSLIDES: Evaluate the potential for ground failure assuming the site is subject to PGA calculated with the design earthquake ground motion equal to $PGA_M/1.5$.

2.1 Site Geologic Map: Present a site geologic map and one or more geologic cross sections showing the relationship between topography, geologic units, existing or modeled slide planes, and all structures such as retaining walls and buildings. At least one cross section should be drawn along the critical profile for stability analyses. Document surface and subsurface observations, including evidence of slope movement, building distress, slope monitoring data, and depth and extent of slip surfaces or planes of weakness. Indicate if the building is on the landslide margin or recognizable graben feature, or if it lies within a recognizable coherent block. Justify assumptions regarding ground water, and provide evidence for unit weight and shear strength values used in slope stability calculations.

2.2 Slope stability profiles should be based on the geologic cross sections. If the slope fails a pseudostatic screening procedure, **estimate vertical and horizontal earthquake-induced displacement at the location of the structure**, and demonstrate whether the building straddles a critical slip surface or will be subject to severe deformation due to the modeled slope movement.

3. SURFACE FAULT RUPTURE: A probabilistic fault displacement analysis is not a practical approach at this time for most sites. Therefore, any Holocene-active fault will be considered to have sufficient *probability* of rupture, and an estimate of expected surface displacement should be presented. Unusually large displacement estimates will be carefully considered by CGS. CGS should be provided an opportunity to review in the field any new exploratory fault trenches excavated at the site. The project geologist is strongly encouraged to discuss the site with CGS prior to embarking on the fault investigation.

The consultants should provide evidence of the existence of Holocene surface rupture within the footprint of the building. Given the maximum characteristic magnitude on the main trace of this fault and the characteristics of the splay underlying the building, **estimate both vertical and horizontal components of fault displacement.** The consultants' analysis should be fully explained, and will be critically reviewed by CGS.

If the building is eligible for funding under the SMP due to surface fault rupture, the rehabilitation option is not allowed since rehabilitated buildings must meet current building code requirements, which is not possible for a building within 50 feet of a Holocene-active fault. Therefore, the building must be abandoned and replaced, rather than rehabilitated.

Appendix C– Guidelines for Determining Fire & Life Safety and Accessibility Requirements

C.1 Fire & Life Safety Requirements:

C.1.1 Fire & Life Safety provisions shall apply strictly to area(s) of rehabilitation work within the scope of proposed improvements (2013 California Building Code (CBC), Chapter 34, Sections 3401.4.1 and 3412.2).

C.1.2 Whatever portions of the building are demolished, new construction will be reviewed under current provisions of the CBC.

C.1.3 In compliance with 2013 CBC, Section 3423.1 (1) applicant shall include in the "Evaluation and Design Criteria Report" the following information pursuant to the code edition applicable at the time of original plan approval.

a) A complete building code analysis that includes construction type, building height and area, allowable building size increases, and occupancy group(s).

b) Identify means of egress configuration and characteristics in the building. Information shall include dead-ends where two or more exits are required, and travel distances. Rehabilitation work that affects the means of egress may generate additional requirements.

c) Identify location and type of fire rated construction; including corridor walls and vertical openings. Through membrane penetrations of rated systems will require a fire-rated fire stop system with the same or greater hourly rating as the violated rated construction.

d) Existing building fire rated components that require asbestos abatement within scope of work, shall be reconstructed with rated equivalent materials as needed to maintain fire-rating.

e) Identify existing individual room occupancy group as noted on the original approved plans. Identify if the occupancy group(s) have changed from the approved plans. Change of use in any room would require current code provisions to be met.

f) Identify the HVAC systems ability to resist the movement of smoke and fire beyond the point of origin. HVAC systems that are impacted by the rehabilitation, and incorporate smoke detector shut down, shall be tested prior to approval of the project to verify correct operation of the system. In the event that the system does not function as originally designed, repairs or replacements will be required for the automatic shutdown feature.

g) Provide an evaluation of the fire alarm and fire suppression system features of the building. Where a system, or portion of a system, is temporarily removed to allow seismic upgrades, a complete test will be required of the system to verify correct operation of the system after it has been re-installed. Test(s) shall be in accordance with National Fire Protection Association Standards. In the event that the system or components of the system are found not operable, repairs or replacements will be required.

C.1.4 Compliance alternatives may be considered as found in the 2013 CBC, Chapter 34, Section 3412. Evaluations may trigger additional scope of work.

C.2 **Access Compliance Requirements:** The seismic repair of an existing facility is governed by 11B-202 of the 2013 CBC. In addition, In Legal Opinion No. 94-1109, dated May 10, 1995, the Attorney General for the State of California concluded that seismic strengthening work in an existing building constitutes a "building alteration, structural repair or addition" for purposes of providing access to the building for persons with disabilities.

In existing buildings or facilities, if seismic strengthening or upgrade work does not alter the primary use or function of the building or facility and/or does not alter the design of specific rooms or spaces, then the requirement for an accessible path of travel to the area of specific alteration does not apply. However, the requirement to provide an accessible primary entrance, sanitary facilities, drinking fountains, signs, and public telephones, as well as an accessible path of travel connecting these elements comply with the currently effective regulations.

In existing buildings or facilities, when the primary use or function of the building or facility and/or design of specific rooms or spaces are altered, the seismic strengthening or upgrade work must comply with all applicable accessibility regulations for new construction. In addition, the obligation to provide an accessible primary entrance to the building or facility, and primary path of travel to the specific area of alteration, including sanitary facilities, drinking fountains, signs, and public telephones serving the area must be met.

ELIGIBILITY EVALUATION REPORT

School District:	BEVERLY HILLS UNIFIED SCHOOL DISTRICT	Original Report Date:	DEC. 20, 2013
School Campus:	EL RODEO SCHOOL (K-8)		
School Address:	605 WHITTIER DRIVE BEVERLY HILLS, CA 90210	Last Revision Date:	
Building Name/ID:	ABC		
Project Tracking No.:			

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

Appendix A.1 Structural calculations
Appendix A.2 Evaluation statement notes
Appendix A.3 Photographs and details

KANDA & TSO, ASSOCIATES – INC. KANDA | TSO

SE Firm Name (Logo optional)

SE Address: **511 MISSION STREET**
SOUTH PASADENA, CA 91030

Phone: **(626) 441-1211 / www.kandatso.com / LesTso@KandaTso.com**
(website or email address optional)

LESLIE TSO
Name of SE whose stamp is above

1. Eligibility Check Summary

	YES	NO
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 = 1.847

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

☐ LIQUEFACTION ☐ SLOPE STABILITY FAILURE ☐ SURFACE FAULT RUPTURE

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

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[1], 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.

[1] *Seismic Evaluation of Existing Buildings* (ASCE/SEI 31-03), American Society of Civil Engineers, 2003.

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.[2]
- 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.

[2] *2007 California Building Standards Administrative Code (California Code of Regulations, Title 24 Part 1)*, Chapter 6, "Seismic Evaluation Procedures for Hospital Buildings," Section 1.4.5.1.2, October 23, 2008 Emergency Supplement.

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 (and elsewhere) of this report.

SET ID	DATE	DESCRIPTION
D1	Dated Feb. 3, 1927	**'BEVERLY HILLS GRAMMAR SCHOOL'** John C. Austin, FAIA; Frederic M. Ashley, AIA; W. Asa Hudson, AIA – Architects Original Construction Drawings Sheets 1, 2, 3, 4, 5, 6, 7, 10, 11, 101, 102, 103, 104, 105, 106, 107, 108 (17 total sheets)
D2	DSA Approved June 7, 1934 (App No 290)	**'EL RODEO SCHOOL'** Holmes and Narver, Inc. – Structural Engineer Retrofit Construction Drawings Sheets 1, 2, 2', 3, 4, 5, 6 (7 total sheets)
D3	DSA Approved July 29, 1966 (App No 27533)	**'ADDITIONS AND ALTERATIONS - EL RODEO ELEMENTARY SCHOOL'**** Maurice H. Fleishman, AIA – Architect Goldsmith, Chi & Associates, Inc. – Structural Engineer Additions/Original Construction Drawings (selected sheets) Sheets S1; S-1; S2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13, 14, 15, 16, 17, 18, 19, 20 (21 total sheets)
D4	DSA Approved May 12, 1999 (App No 101856)	**'MODERNIZATION OF BEVERLY HILLS UNIFIED SCHOOL DISTRICT - EL RODEO SCHOOL'** Landgon Wilson International, Inc. – Architect Hillman, Biddison & Loevenguth, Inc. – Structural Engineer Modernization/Retrofit Construction Drawings (selected sheets) Sheets ST-1, -2, -3; S-1.1, -1.2, -1.3, -1.4, -2.1.1, -2.1.2, -2.1.3, -2.1.4, -2.2.1, -2.2.2, -2.2.3, -2.2.4, 2.3.1, -2.3.3, -2.3.4, -4.1, -4.2, -4.3, -4.4, -4.5, -5.1, -5.2, -5.3, -5.4 (27 total sheets)
D5	Dated Nov. 11, 2008	**'SEISMIC RISK EVALUATION BEVERLY HILLS K-8 SCHOOLS'** MHP, Inc – Structural Engineer Report (selected sheets) (42 total pages)

****Set ID D3 shows construction of new 1966 building, Building E, as well as additions/alterations to Building ABC (1927) of this report, and Building D (1966). Building D is being submitted with its own EER. Set ID D3 is included for reference to said additions/alterations -- Building E itself is not being submitted with its own EER being that it does not have critical deficiencies associated with high potential for local/global collapse in Section 4 and 5.**

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:	**November 8th, 13th, 2013**
Visiting engineer(s) and staff:	**Les Tso, S.E.; Casey Piedra, P.E. - KTA**
School district contact person:	**Charlotte Clement, Chief Facilities Officer - BHUSD**
School campus representative (if different than above):	**(same)**

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
☐ OTHER:

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

SET ID	CONDITION SHOWN ON PLANS	CONDITION OBSERVED AT SITE VISIT
D1	**Diagonal sheathing over wood truss framing connected via 14 gauge clip to 2x sill-plate at top of concrete wall (Sheet 104, Set ID D1)**	**Retrofitted roof-to-wall connections in some locations. Retrofit includes angles mounted to existing wood truss framing and anchored to existing concrete walls. Same areas include plywood sheathing at perimeter (above angles) in lieu of diagonal sheathing per plan. Blocking is provided at the intersection point of the two sheathings. Simpson A34/35 clips installed onto existing blocking between wood truss framing at top of existing concrete wall. No drawings on record provide details for said retrofit observed. See App. A.3, Pg. 40.**

3. Site and Building Description

3.1 Building description

General

Year originally built: **1927**

DSA Application number: **(none)**

☒ Original Construction ☐ Work done pursuant to the Garrison Act (Ed Code 17367)

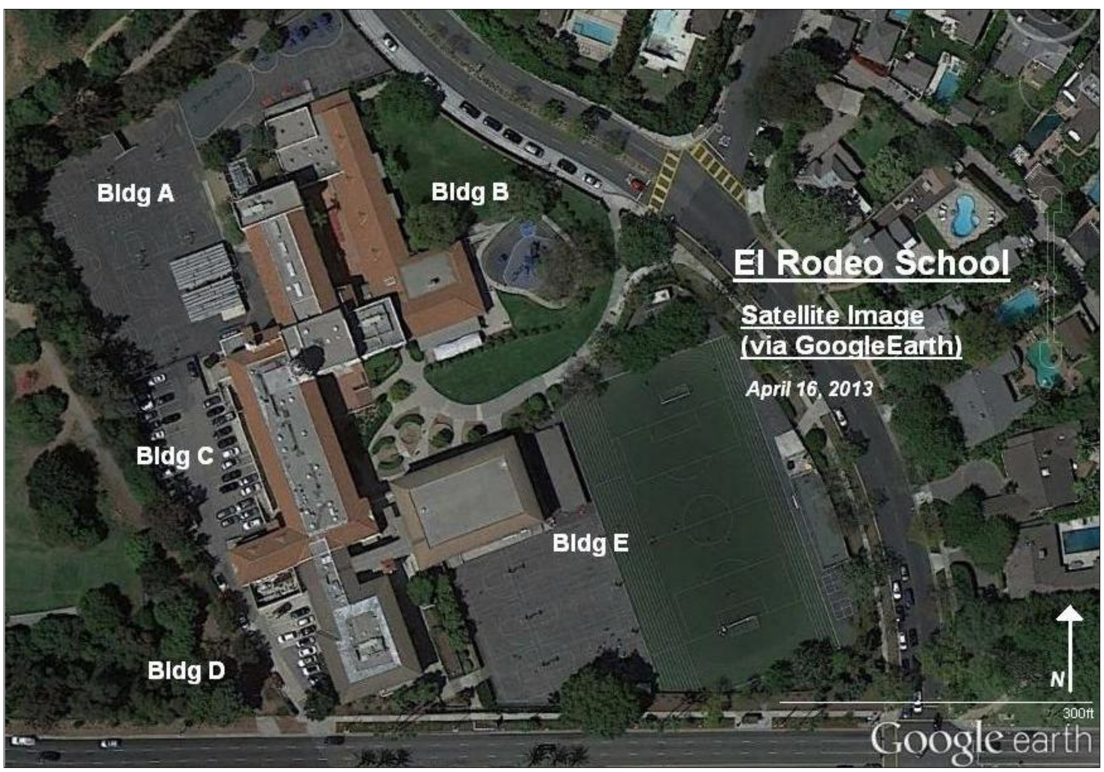
Number of stories above/below grade: ***BLDG A; 2-3 Stories/1 Basement**
***BLDG B; 1 Story (partially subterranean)**
***BLDG C; 2 Stories/1 Basement (partially subterranean)**

Total floor area (sq ft, approx): **49,000 sq ft (footprint)**

Other essentially identical buildings on this campus? ☐ Yes ☒ No

***Note: Building ABC (of this EER) is composed of three 'wings' (A, B, C); however, no structural separations are between the wings, and they are contiguous in all structural aspects. 'Wings' are henceforth referenced as 'Buildings.' Building D and Building E of the campus are structurally separated from each other as well as Building ABC, with Building D being submitted with its own EER. **See note on page 5 regarding Building E.**

Photographs

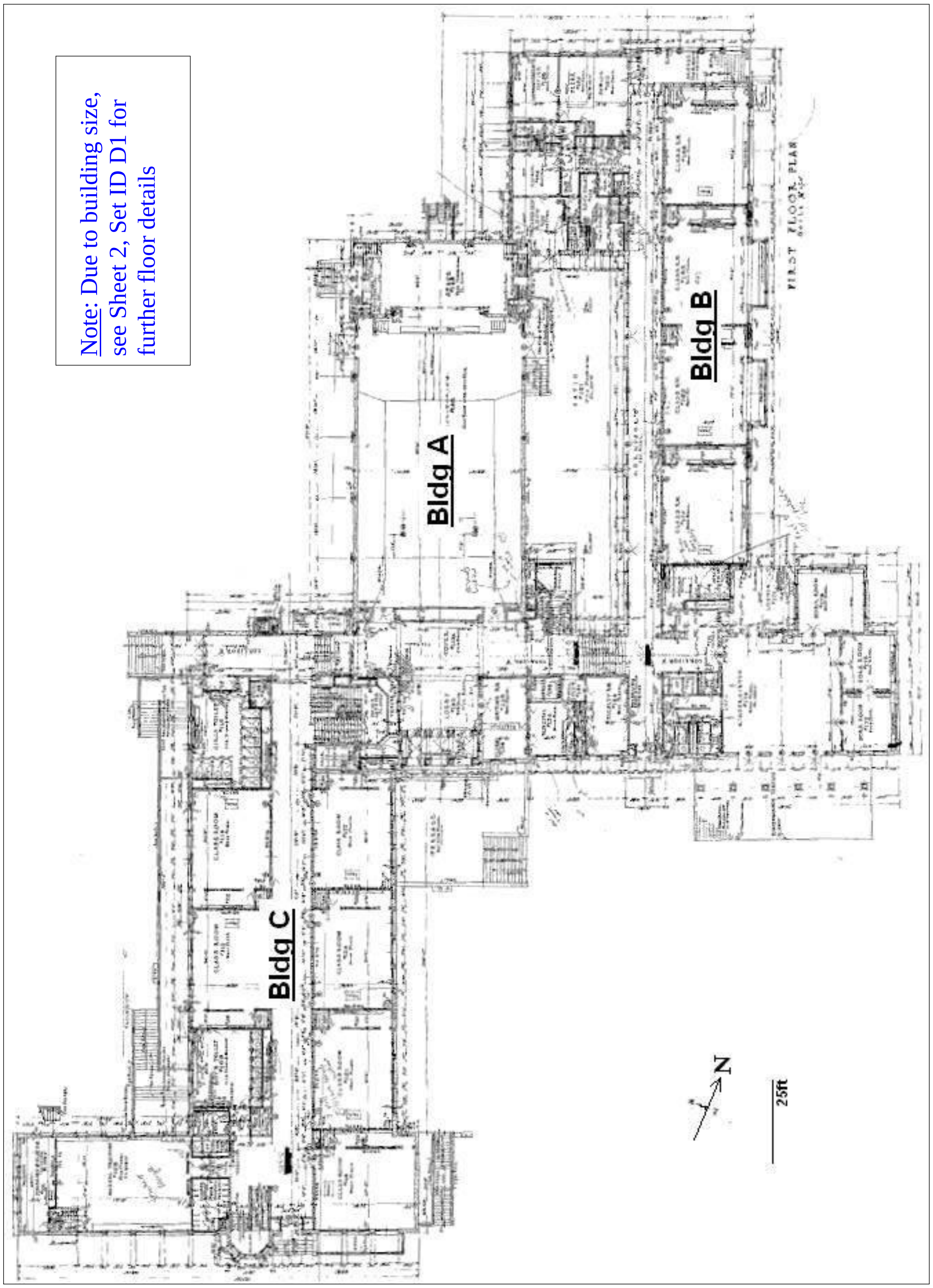


***Photograph 1** - Campus Satellite Image*
Take April 16, 2013 (Via GoogleEarth v7.1.2.2041)

***Photograph 2** – Partial Eastern Elevation (looking South-West)*
Bldg B (low, foreground) and Bldg A (high, background)
Taken April 2013 (via GoogleMaps.com)

***Photograph 3** – Partial Eastern Elevation (looking North-West)*
Bldg C (left) and Bldg A-B (mid-to-right)
Taken 2002 (via Wikipedia.com)

Ground Floor Plan



***Ground Floor Plan** – Bldg ABC Sheet 2, Set ID D1*

3.2 Building Occupancy

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

	Original Use	Current Use	Planned Future Use
Office / Administration	☒	☒	☒
Classrooms / Instruction Areas	☒	☒	☒
Kitchen	☒	☒	☒
Assembly: Dining	☒	☒	☒
Assembly: Auditorium	☒	☒	☒
Assembly: Gymnasium	☐	☐	☐
Locker rooms	☒	☒	☒
Patio cover / ~~bus shelter~~ / walkway cover	☒	☒	☒
Bleachers / stadium structure	☐	☐	☐
Other occupied	☐	☐	☐
Mechanical / utility rooms or enclosures	☒	☒	☒
Bulk storage	☐	☐	☐
Vacant / unused	☐	☐	☐
Other unoccupied	☐	☐	☐

3.3 Seismicity

Latitude: **34.067222°N**

Longitude: **-118.4163°W**

Site Class per ASCE 31, Section 3.5.2.3: **Site Class D**

Basis for Site Class determination: **By Default**

****See App A.1 for analysis**

Period [sec]	Mapped MCE values from ASCE 7-05 [g]	Site Coefficients from ASCE 31 Tables 3-5, 3-6	Design values per ASCE 31 section 3.5.2.3.1 [g]	S_a per ASCE 31 section 3.5.2.3.1, [g]
0.2	S_S = **1.847**	F_a = **1.000**	S_{DS} = (2/3) S_S F_a = **1.232**	$S_{a,0.2}$ = S_{DS} = **1.232**
1.0	S_1 = **0.622**	F_v = **1.500**	S_{D1} = (2/3) S_1 F_v = **0.622**	$S_{a,1.0}$ = min (S_{DS} , S_{D1}/T) = **1.232**

3.4 Gravity System

Roof diaphragm and framing: **BLDG A** **- Reinforced concrete slab over concrete beams/steel trusses to exterior concrete bearing (shear) walls.**
BLDG B, C **- Diagonal wood sheathing over wood rafters/trusses to exterior/interior reinforced concrete bearing (shear) walls.**
- Reinforced concrete slab (over middle corridor), to interior reinforced concrete bearing (shear) walls.

Typical floor diaphragm and framing: **BLDG A** **- Reinforced concrete slab over concrete beams to exterior concrete bearing (shear) walls.**
BLDG B, C **- Diagonal wood sheathing over wood joists to exterior/interior reinforced concrete bearing (shear) walls.**
- Reinforced concrete slab (over middle corridor), to interior reinforced concrete bearing (shear) walls.
Note: Bldg B's 1st floor is partially subterranean

Ground floor framing: **ALL BLDGS** **-Reinforced concrete slab on grade.**

Vertical load-bearing elements: **ALL BLDGS** **-Exterior/interior reinforced concrete bearing/shear walls. Isolated concrete/wood/steel posts in some areas.**

Basement walls: **ALL BLDGS** **-Reinforced concrete bearing/shear walls.**

Foundation: **ALL BLDGS** **-Reinforced concrete spread/pad footings.**

Snow load for use in load combinations involving earthquake: **-not applicable-**

ELIGIBILITY EVALUATION REPORT

School District:	BEVERLY HILLS UNIFIED SCHOOL DISTRICT	Original Report Date:	DEC. 20, 2013
School Campus:	EL RODEO SCHOOL (K-8)		
School Address:	605 WHITTIER DRIVE BEVERLY HILLS, CA 90210	Last Revision Date:	
Building Name/ID:	ABC		
Project Tracking No.:			

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

	North-South	East-West
C1 Concrete Moment Frames	☐	☐
C1B* Reinforced Concrete Cantilever Columns	☐	☐
C2A Concrete Shear Walls, Flexible Diaphragm	☐	☐
C3A Concrete Frame with Infill Masonry Shear Walls, Flexible Diaphragm	☐	☐
PC1 Precast/Tilt-up Concrete Shear Walls, Flexible Diaphragm	☐	☐
PC1A Precast/Tilt-up Concrete Shear Walls, Rigid Diaphragm	☐	☐
PC2 Precast Concrete Frames with Shear Walls, Rigid Diaphragm	☐	☐
PC2A Precast Concrete Frames without Shear Walls, Rigid Diaphragm	☐	☐
RM1 Reinforced Masonry Bearing Walls, Flexible Diaphragm	☐	☐
S1B* Steel Cantilever Columns	☐	☐
S3 Steel Light Frames	☐	☐
URM Unreinforced Masonry Bearing Walls, Flexible Diaphragm	☐	☐
URMA Unreinforced Masonry Bearing Walls, Rigid Diaphragm	☐	☐
M* Mixed Systems - construction containing at least one of the above lateral-force-resisting systems in at least one direction of seismic loading. **Bldg A – C2A/Conc. SW, Rigid Diaph.** (mixed in both directions/at all stories) **Bldg B – C2A** (lng. dir.); **C2A/Reinf. CMU non-brg SW** (trnvs. dir) **Bldg C – C2A/Conc. SW, Rigid Diaph.** (mixed in both directions/at all stories) **Steel BF, Flex./Rigid Diaphs** (trnvs. dir)	☒	☒
None of the above	☐	☐

* 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	***(see below as applicable to each bldg/system)***
Vertical system combinations	***(see below as applicable to each bldg/system)***
SFRS foundation	***All Bldgs*** *-reinf. conc. pad/spread footings, typ.* ***Bldg B*** *-deep pad/spread footings/caissons in some areas (Set ID D3, S14)*
Gravity loading	***Bldg A*** *-8"w to 12"w reinf. conc. shear walls support gravity loads in transverse direction.* ***Bldgs B, C*** *-8"w to 12"w reinf. conc. shear walls support gravity loads typically at exterior perimeter and at interior corridor, in both longitudinal and transverse directions.* *(note: isolated concrete/steel/wood posts in some areas. see plans)*

3.5 Structural System – *cont.*

System details	***Bldg A*** *-6'd x 55'l steel angle trusses @ 18'o.c. bolted to 24"x36" reinf. conc. columns w/ a) (2)-3/8" dia. anchor bolts (Set ID D1, Sheet 107), b) angle connection (Set ID D2, Sheet 1).* ***Bldgs B, C*** *-4"d to 7"d reinf. conc. slab (middle corridor) doweled to 8"d to 12"d reinf. conc. shear/bearing wall (Set ID D1, Sheets 102, 103)* *-3'd x 23'l 2x wood trusses @ 30"o.c. 14ga clipped/nailed to reinf. conc. shear/bearing wall. some instances have undocumented angle anchorages (no other positive connection, (Set ID D1, Sheet 104; see page 7 for angle retrofit description)*
Structural materials	***Original Construction (1927)*** *-uncertain-* ***Retrofit Construction (1934)*** *-ASTM Spec. A7-29 Steel, typical (Set ID D2, Sheet 1)* ***Addition Construction (1966)*** *(Set ID D3, Sheet 18, General Notes)* ***Concrete/Reinforcing*** *-3000psi 28-day strength Concrete, typical* *-ASTM A15-62T Rebars* ***Steel*** *-ASTM A36 Steel, typical* *-ASTM A53 58T Pipe* ***Wood*** *-DFL, Construction Grade* ***Modernization/Retrofit Construction (1999)*** ***(Set ID D4, Sheet ST-1, General Notes)*** ***Concrete/Reinforcing*** *-4000psi 28-day strength Concrete (walls); 3000psi otherwise* *-ASTM A615 Gr 40 Rebars (#4s and smaller); Gr 60 otherwise* ***Masonry*** *-ASTM C90, Medium Weight Block* *-2000psi 28-day strength Grout* ***Steel*** *-ASTM A36 Steel, typical* *-ASTM A500 Gr B HSS Shapes* ***Wood*** *-DLF No-1, SS, typical*
Original design code	***-uncertain- (pre-dates first UBC of October 1927)***

3.5 Structural System –cont.

History of seismic retrofit or significant alteration	***1934 (Set ID D2)*** *(code uncertain)* ***Building A*** *-Angle wall tie conns to (e) stl trusses added (1)* *-Reinf. conc. cols & angle/rod bracing to bell-tower (2)* ***Building C*** *-Horiz. angle bracing added (1)* ***1966 (Set ID D3)*** *(code uncertain)* ***Building B*** *-1-story wood framed addition (S14)* ***1999 (Set ID D4)*** *(1995 CBC)* ***Building A*** *-Full height reinf. conc. col. at bell-tower added (S-5.3)* *-Horiz. channel bracing at bell-tower added (S-2.3.3)* *-Horiz. HSS wall ties (2/S-5.1), and anchs to (e) wood ledger added (all flrs; A/S-4.5)* ***Building B*** *-Full height CMU shear walls added (2 lines, trnsv. dir.; F,H,G/S-4.5)* *-Reinf. conc. infills added to 3 (e) open'gs (1 line, lng. dir.; 8/ST-3)* ***Building C*** *-Full height HSS chevron BFs added (2 lines, trnsv. dir.; A/S-5.2).* *-Full height reinf. shotcrete added to (e) reinf. conc. shear wall (1 line, trnsv. dir.; L,M/S-4.5)* *-Fnd-to-1ST flr. reinf. conc. shear walls added (2 lines, lng. dir.; S-1.3)* *-Fiberwrap to (e) basement reinf. conc. cols added (12 cols; S-1.3)* *-Horiz. angle bracing added (all flrs;S-2.1.3)* *-Horiz. HSS wall ties added (all flrs; S-2.1.3)*
Benchmark year check	***-not applicable-***

ELIGIBILITY EVALUATION REPORT

4. Deficiency list

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

Non-compliant condition	Discussion	Additional evaluation recommended
SHEAR STRESS CHECK**	**Previous limited analysis indicates isolated shear stresses were non-compliant when compared to LS requirements of ASCE 31. See ID D5, Pg. 30.**	*In-depth analysis of entire lateral system/Tier 2 Evaluation*
WALL OPENINGS	By inspection, longitudinal perimeter walls at Building B are non-compliant, as shown in Set ID D1, Sheet 103.	*None*
REINFORCING STEEL (CMU)	Added 1999 retrofit (Set ID D4) CMU shear walls' horizontal and vertical reinforcing ratios to Bldg B were non-compliant, as show in App A.2, Pg. 35. Note, two lines of resistances were added in transverse direction.	*None*
CONNECTION STRENGTH**	Assumed by inspection; diagonals are HSS8x8x5/8, ASTM A500 Gr B (Set ID D4, ST-1; A/S-5.2). See note below.	*None*
WALL ANCHORAGE**	**Comprehensive out-of-plane (OOP) anchorage of concrete shear walls to diaphragms was not addressed in original construction drawings or subsequent retrofits for Building B (Set ID D1, D2, D3, D4; D5, Pg. 29). Attempts at OOP anchorage in the 1999 Retrofit (Set ID D4) for Building C were incomprehensive, with isolated retrofit connections proving inadequate for Life Safety performance levels (App A.2, Pg. 37). Apparent retrofit in 1934 (Set ID D2) for Building A entails further analysis and investigation.**	*Additional non-destructive testing/analysis (Bldg A only)*

Unknown condition	Discussion	Additional evaluation recommended
REINFORCING STEEL (CONC.)	Set ID D1 of original construction lack concrete shear wall reinforcing information/detailing. See App A.2 for analysis of reinforcing of added shear walls from 1999 retrofit (Set ID D4), which were compliant.	*Additional non-destructive investigation*
AXIAL STRESS CHECK**	See note below.	*In-depth analysis of entire lateral system/Tier 2 Evaluation*
ROOF CHORD CONTINUITY	Previous limited analysis indicates that the chord of the reinforced concrete slab roof diaphragm of Building A does not meet Life Safety performance level. See Set ID D5, Pg 29.	*In-depth analysis of entire lateral system/Tier 2 Evaluation*

****Note**: '**WALL ANCHORAGE**' considered key critical non-compliant deficiency. Other unknown or non-compliant conditions were researched/analyzed but may warrant further investigation/analysis for complete evaluation. Also, **bold print** indicates critical condition.

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.

Note: Description/Instructions of selected items marked C or NA are removed for brevity

CONDITION OF MATERIALS	
[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.

C NC U NA	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	
C NC U NA **Critical Item**	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.
C NC U NA **Critical Item**	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.
C NC U NA **Critical Item**	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.
C NC U NA	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.
C NC U NA **Critical Item**	VERTICAL DISCONTINUITIES. All vertical elements of the seismic force-resisting system shall be continuous to the foundation.
C NC U NA **Critical Item**	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.
C NC U NA **Critical Item**	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.
C NC U NA **Critical Item**	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. **Per ASCE 31, C4.3.1.2, buildings that are the same height and have matching floor levels need not comply with 4.3.1.2. Bldg C (of Bldg ABC) and Bldg D are such buildings.**
C NC U NA **Critical Item**	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.

MOMENT FRAMES

**** No Moment Frames in SFRS. Descriptions within check-list removed for brevity ****

C NC U NA Critical Item	SHEAR STRESS CHECK (Columns)
C NC U NA Critical Item	AXIAL STRESS CHECK (Concrete columns)
C NC U NA	AXIAL STRESS CHECK (Steel columns)
C NC U NA Critical Item	FLAT SLAB FRAMES
C NC U NA	PRESTRESSED FRAME ELEMENTS
C NC U NA Critical Item	CAPTIVE COLUMNS
C NC U NA	NO SHEAR FAILURES
C NC U NA	STRONG COLUMN/WEAK BEAM
C NC U NA	STRONG COLUMN/WEAK BEAM
C NC U NA Critical Item	BEAM BARS
C NC U NA	COLUMN BAR SPLICES
C NC U NA	BEAM BAR SPLICES
C NC U NA	COLUMN TIE SPACING
C NC U NA	STIRRUP SPACING
C NC U NA	JOINT REINFORCING
C NC U NA	COMPLETE FRAMES
C NC U NA Critical Item	DEFLECTION COMPATIBILITY
C NC U NA Critical Item	FLAT SLABS
C NC U NA Critical Item	REDUNDANCY (Moment frame)
C NC U NA	INTERFERING WALLS

C NC U NA	PRECAST CONNECTION CHECK
C NC U NA	PRECAST FRAMES
C NC U NA	PRECAST CONNECTIONS
C NC U NA	DRIFT CHECK
C NC U NA	MOMENT-RESISTING CONNECTIONS
C NC U NA	PANEL ZONES
C NC U NA	COLUMN SPLICES
C NC U NA	COMPACT MEMBERS
SHEAR WALLS	
C NC U NA 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.
C NC U NA 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, isshall be less than the greater of 100 psi or $2\sqrt{f'_c}$.** **See Set ID D5, Pg. 30. Also, see **Note, Page 16, and description in Section 4.**
C NC U NA	**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 D1 of original construction lack concrete shear wall reinforcing information/detailing. Also, see App A.2, Pg. 35.**
C NC U NA	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 135º or more.
C NC U NA Critical Item	REDUNDANCY (Shear wall). The number of lines of shear walls in each principal direction shall be greater than or equal to 2.
C NC U NA	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.)

C NC U $\boxed{\text{NA}}$	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.)
C NC U $\boxed{\text{NA}}$	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.)
C NC U $\boxed{\text{NA}}$ **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 $2\sqrt{f'_c}$.
C $\boxed{\text{NC}}$ U NA	**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.** **By inspection; see Set ID D1, Sheet 103.**
C NC U $\boxed{\text{NA}}$	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.
C NC U $\boxed{\text{NA}}$ **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.
C NC U $\boxed{\text{NA}}$ **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.
C NC U $\boxed{\text{NA}}$	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
C $\boxed{\text{NC}}$ U NA	**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.** **See App. A.2, Pg. 35.**
BRACED FRAMES	
C NC U $\boxed{\text{NA}}$ **Critical Item**	REDUNDANCY: The number of lines of braced frames in each principal direction shall be greater than or equal to 2.
C NC $\boxed{\text{U}}$ NA **Critical Item****	**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 $0.50F_y$.** **See **Note, Page 16, and description in Section 4.**

$\boxed{\text{C}}$ NC U NA	SLENDERNESS OF DIAGONALS: All diagonal elements required to carry compression shall have Kl/r ratios less than 120. **See App A.2, Pg. 36**
C $\boxed{\text{NC}}$ U NA	CONNECTION STRENGTH: All the brace connections shall develop the yield capacity of the diagonals. **Assumed by inspection (diagonals are HSS8x8x5/8, ASTM A500 Gr B (Set ID D4, ST-1; A/S-5.2). Also, see **Note, Page 16, and description in Section 4.**
C NC U $\boxed{\text{NA}}$	K-BRACING: The bracing system shall not include K-braced bays.
DIAPHRAGMS	
C NC U $\boxed{\text{NA}}$	DIAPHRAGM CONTINUITY. The diaphragm shall not be composed of split-level floors and shall not have expansion joints.
$\boxed{\text{C}}$ NC U NA	CROSS TIES. There shall be continuous cross ties between diaphragm chords.
C NC $\boxed{\text{U}}$ NA	**ROOF CHORD CONTINUITY. All roof chord elements shall be continuous, regardless of changes in roof elevation.** **See Set ID D5, Pg. 29. Also, see **Note, Page 16, and description in Section 4.**
C NC U $\boxed{\text{NA}}$ Critical Item	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.
$\boxed{\text{C}}$ NC U NA	OPENINGS AT BRACED FRAMES. Diaphragm openings immediately adjacent to the braced frames shall extend less than 25% of the frame length.
$\boxed{\text{C}}$ NC U NA	OTHER DIAPHRAGMS. The diaphragm shall not consist of a system other than wood, metal deck, concrete or horizontal bracing.
C NC U $\boxed{\text{NA}}$ Critical Item	TOPPING SLAB. Precast concrete diaphragm elements shall be interconnected by a continuous reinforced concrete topping slab.
C NC U $\boxed{\text{NA}}$	STRAIGHT SHEATHING. All straight sheathed diaphragms shall have aspect ratios less than 2 to 1 in the direction being considered.
$\boxed{\text{C}}$ 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 $\boxed{\text{NA}}$	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 **Critical Item**	**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.** **See App A.2, Pg. 37, and description in Section 4.**
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. **Out of plane anchorage to wood elements are not known to exist. Also, see 'WALL ANCHORAGE' check, above.**
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.
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.

$\boxed{C}$ NC U NA	CONCRETE COLUMNS. All concrete columns shall be doweled into the foundation.
$\boxed{C}$ NC U NA	FOUNDATION DOWELS. Wall reinforcement shall be doweled into the foundation.
C NC U $\boxed{NA}$	PRECAST WALL PANELS. Precast wall panels shall be connected to the foundation.
C NC U $\boxed{NA}$	UPLIFT AT PILE CAPS. Pile caps shall have top reinforcement and piles shall be anchored to the pile caps.
$\boxed{C}$ NC U NA	STEEL COLUMNS: The columns in lateral-force-resisting frames shall be anchored to the building foundation.
C NC U $\boxed{NA}$	WALL PANELS: Metal, fiberglass or cementitious wall panels shall be positively attached to the foundation.
C NC U $\boxed{NA}$	ROOF PANELS: Metal, plastic, or cementitious roof panels shall be positively attached to the roof framing to resist seismic forces.
C NC U $\boxed{NA}$	WALL PANELS: Metal, fiberglass or cementitious wall panels shall be positively attached to the framing to resist seismic forces.
FOUNDATION	
C NC U $\boxed{NA}$	POLE FOUNDATIONS. Pole foundations shall have a minimum embedment depth of 4 ft.
C NC U $\boxed{NA}$	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	
Geologic Site Hazards not used for basis of eligibility. Descriptions within check-list removed for brevity	
C NC U $\boxed{NA}$	LIQUEFACTION
C NC U $\boxed{NA}$	SLOPE FAILURE
C NC U $\boxed{NA}$	SURFACE FAULT RUPTURE

Appendices

A.1 Structural calculations

MCE SPECTRAL RESPONSE ACCELERATIO PARAMETER ANALYSIS

SEE PROCEEDING PAGES FOR LAT/LONG & AREA/REFERENCE USED FOR ACCELERATIONS

CAMPUS AREA 3	S_1	S_S	S_{D1}	S_{DS}
'CENTER' (PER USGS LAT./LNG.) 2	0.619	1.841	0.619	1.227
'CENTER' (PER WEB. ITOUCHMAP) 1	0.619	1.841	0.619	1.227
'CENTER' (PER MHP REPORT)	0.619	1.841	0.62	1.227
NORTHERN END 1,2	0.617	1.836	0.617	1.224
SOUTHERN END 1,2	0.622	1.847	0.622	1.232
EASTERN END	0.619	1.841	0.619	1.227
WESTERN END 1,2	0.620	1.843	0.620	1.229

∴ GOVERNING ACCELERATIONS

$S_1 = 0.622$

$S_S = 1.847$

$S_{D1} = 0.622$

$S_{DS} = 1.232$

NOTES:
1. USES 'ITOUCHMAP.COM' FOR LAT./LNG. COORD
2. USES USGS EQ GROUND MOTION PARAMETERS V.5.1.0 FOR ACCELERATIONS
3. ALL AREAS ASSUMED AS SITE CLASS D

A.1 Structural calculations – cont.

USGS
Geographic Names Information System (GNIS)

Stop! Do not bookmark or copy/paste this URL before reading FAQs.

Query Result FAQs

Feature Detail Report for: El Rodeo Elementary School

ID:	241922
Name:	El Rodeo Elementary School
Class:	School (Definitions)
Citation:	California Department of Education. Calfornia Public School Directory. Sacramento, California, 1994. 1994
Address:	605 Whittier Drive
City:	Beverly Hills
State:	CA
ZIP:	90210-3112
Entry Date:	14-Jun-2000
*Elevation:	302/92

*Elevations in feet/meters from the National Elevation Dataset

Mapping Services

- GNIS in ESRI Map
- USGS The National Map
- HomeTownLocator
- ACME Mapper 2.0
- Microsoft Virtual Earth
- TerraFly.com
- Find the Watershed
- MapQuest

USGS
GEOGRAPHIC NAMES INFORMATION SYSTEMS (GNIS)

LAT./LNG.
FOR
EL RODEO SCHOOL
(& SITE COOR.)

Variant Names

Variant Name
El Rodeo School Citation

Counties

Sequence	County	Code	State	Code	Country
1	Los Angeles	037	California	06	US

Coordinates (One point per USGS topographic map containing the feature, NAD83)

Sequence	Latitude(DEC)	Longitude(DEC)	Latitude(DMS)	Longitude(DMS)	Map Name
1	34.0677700	-118.4158700	340404N	1182457W	Beverly Hills

U.S. Department of the Interior || U.S. Geological Survey
12201 Sunrise Valley Drive, Reston, VA 20192, USA
gnis_manager@usgs.gov
Form updated: November 19, 2013
USGS Privacy Policy and Disclaimers

A.1 Structural calculations – cont.

iTouchMap.com
Mobile and Desktop Maps

Maps | Country - State | Places | Google Earth | Cities | Lat - Long

Home » Latitude and Longitude of a Point

To find the latitude and longitude of a point **Click** on the map, **Drag** the marker, or enter the...

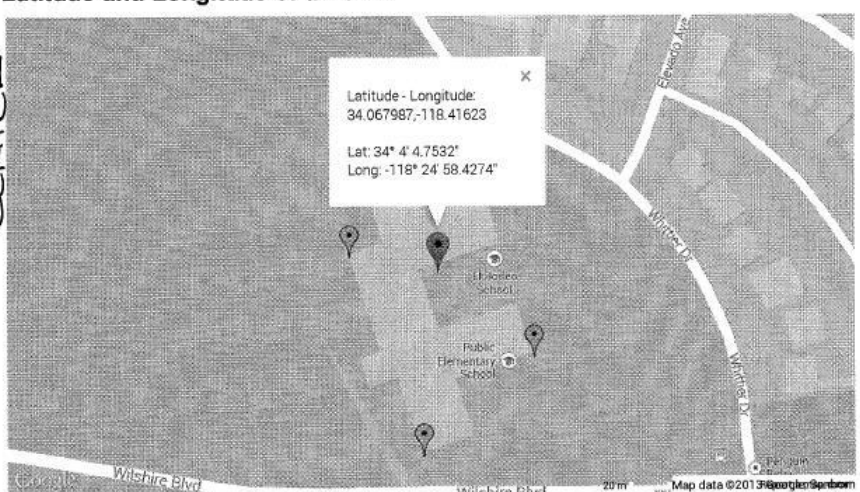
Address: 605 Whittier Blvd Beverly Hills, CA [GO]

Map Center: Get Address - Land Plat Size - Street View - Area Photographs

Try out 3D Google Earth. Google Earth gives you a 3D look of the area around the center of the map, which is usually your last click point, and includes latitude, longitude and elevation information.

Latitude and Longitude of a Point

CENTER



Clear / Reset | Remove Last Blue Marker | Center Red Marker

Get the Latitude and Longitude of a Point

When you click on the map, move the marker or enter an address the latitude and longitude coordinates of the point are inserted in the boxes below.

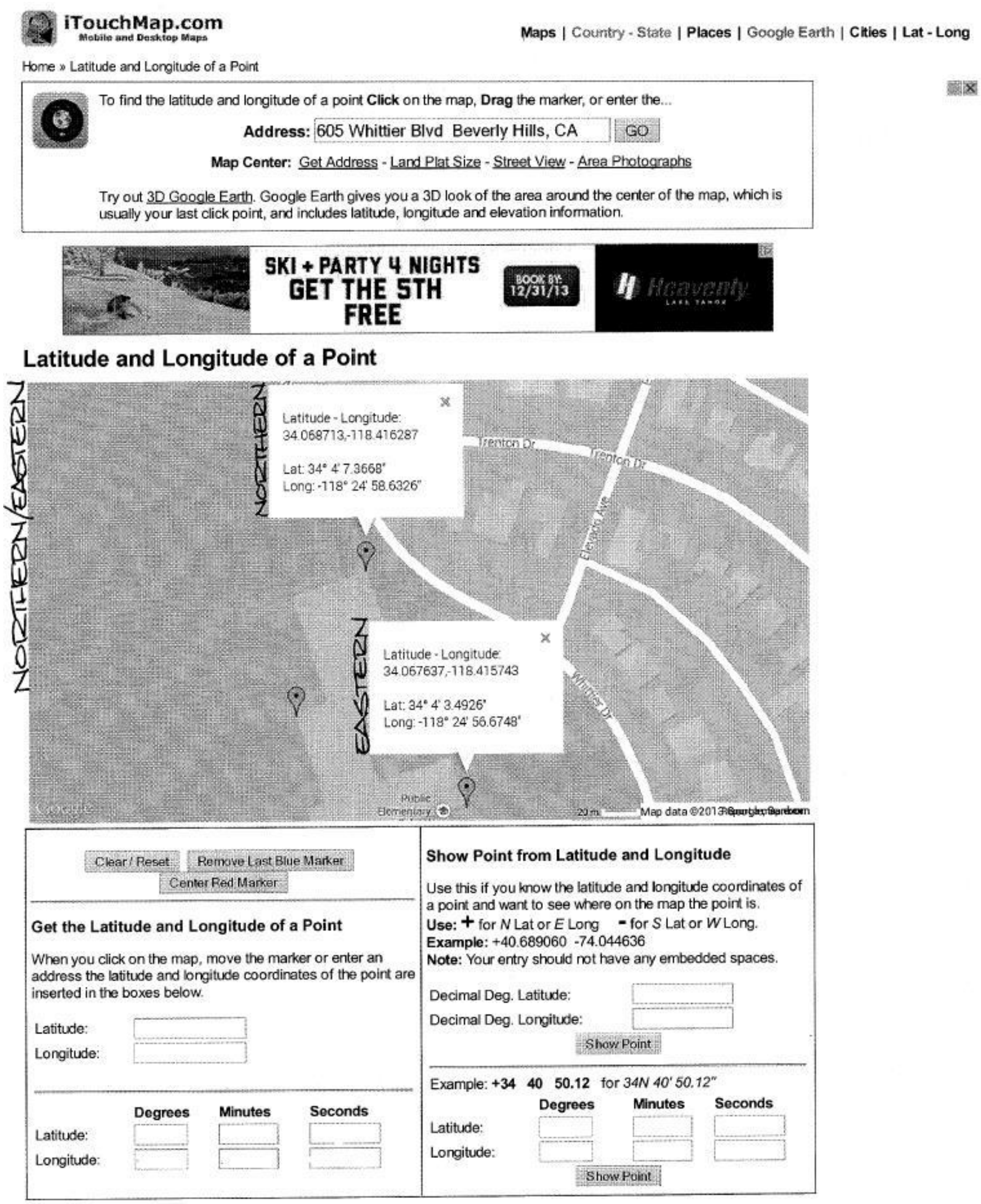
Latitude: 34.067987
Longitude: -118.416230

	Degrees	Minutes	Seconds
Latitude:	34	4	4.7532
Longitude:	-118	24	58.4274

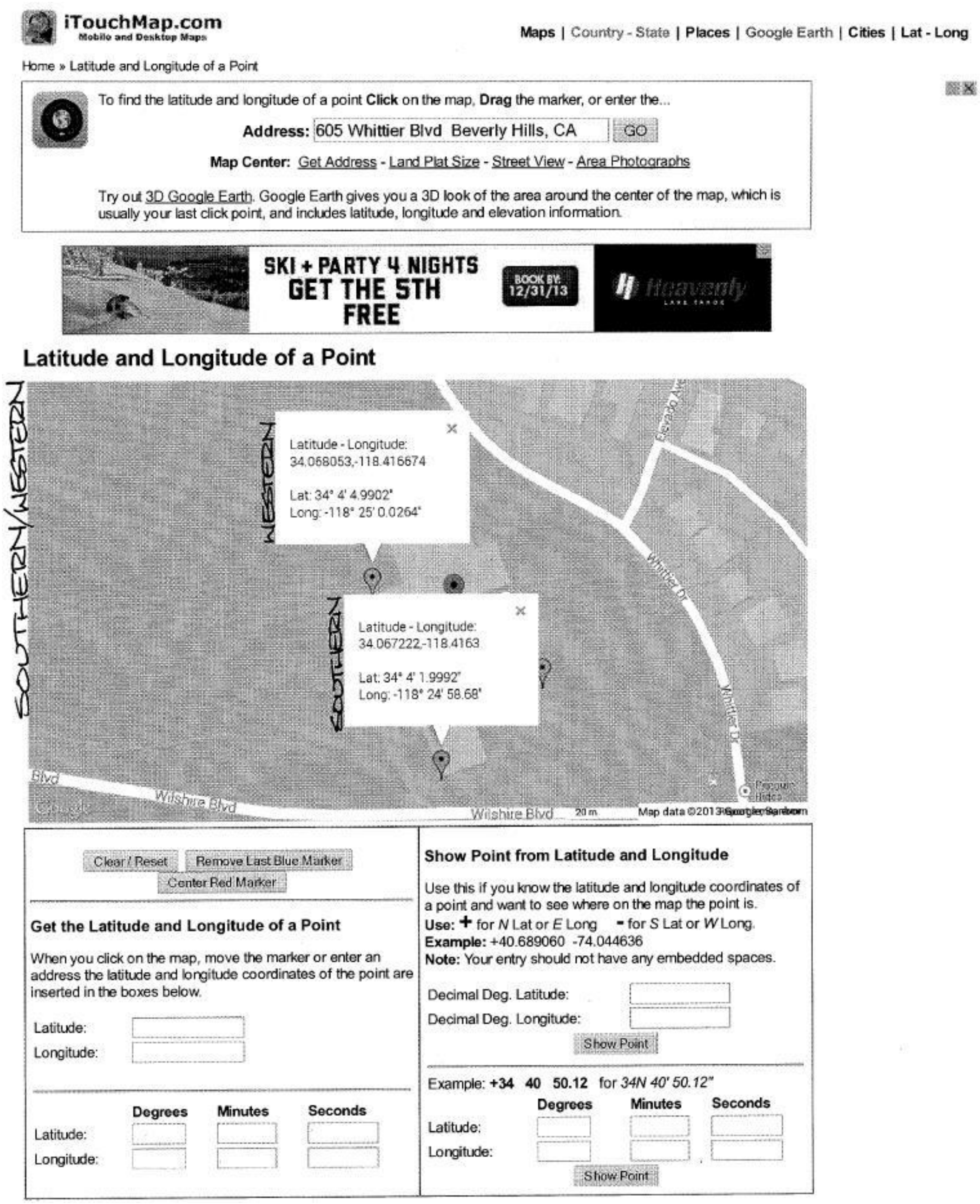
iTOUCHMAPS

LAT. / LNG.
FOR
EL RODEO SCHOOL
(@ EXTREME BLDG/SITE ENDS)

A.1 Structural calculations – cont.



A.1 Structural calculations – cont.



A.1 Structural calculations – cont.

Project Name = EL RODEO - SMP
Date = Fri Dec 06 11:54:32 PST 2013

CENTER COOR'D (PER USGS)

Conterminous 48 States
2005 ASCE 7 Standard
Latitude = 34.06777
Longitude = -118.41587
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
Period Sa
(sec) (g)
0.2 1.841 (Ss, Site Class B)
1.0 0.619 (S1, Site Class B)

Conterminous 48 States
2005 ASCE 7 Standard
Latitude = 34.06777
Longitude = -118.41587
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
Period Sa
(sec) (g)
0.2 1.227 (SDs, Site Class D)
1.0 0.619 (SD1, Site Class D)

Conterminous 48 States
2005 ASCE 7 Standard
Latitude = 34.06777
Longitude = -118.41587
Spectral Response Accelerations SMs and SM1
SMs = Fa x Ss and SM1 = Fv x S1
Site Class D - Fa = 1.0 ,Fv = 1.5
Period Sa
(sec) (g)
0.2 1.841 (SMs, Site Class D)
1.0 0.929 (SM1, Site Class D)

USGS
EARTHQUAKE GROUND MOTION PARAMETERS v5.1.0

SPECTRAL RESPONSE ACCELERATIONS
S_1, S_S
S_{M_1}, S_{MS}
S_{D_1}, S_{DS}

Project Name = EL RODEO - SMP
Date = Fri Dec 06 11:55:54 PST 2013

CENTER COOR'D (PER iTOUCHMAPS)

FOR EL RODEO SCHOOL

Conterminous 48 States
2005 ASCE 7 Standard
Latitude = 34.067987
Longitude = -118.41622999999998
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
Period Sa
(sec) (g)
0.2 1.841 (Ss, Site Class B)
1.0 0.619 (S1, Site Class B)

Conterminous 48 States
2005 ASCE 7 Standard
Latitude = 34.067987
Longitude = -118.41622999999998
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
Period Sa
(sec) (g)
0.2 1.227 (SDs, Site Class D)
1.0 0.619 (SD1, Site Class D)

Conterminous 48 States
2005 ASCE 7 Standard
Latitude = 34.067987
Longitude = -118.41622999999998
Spectral Response Accelerations SMs and SM1
SMs = Fa x Ss and SM1 = Fv x S1
Site Class D - Fa = 1.0 ,Fv = 1.5
Period Sa
(sec) (g)
0.2 1.841 (SMs, Site Class D)
1.0 0.929 (SM1, Site Class D)

A.1 Structural calculations – cont.

Conterminous 48 States
2005 ASCE 7 Standard
Latitude = 34.068053
Longitude = -118.41667399999999
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

Period (sec)	Sa (g)
0.2	1.843 (Ss, Site Class B)
1.0	0.620 (S1, Site Class B)

Conterminous 48 States
2005 ASCE 7 Standard
Latitude = 34.068053
Longitude = -118.41667399999999
Spectral Response Accelerations SMs and SM1
SMs = Fa x Ss and SM1 = Fv x S1
Site Class D - Fa = 1.0 ,Fv = 1.5

Period (sec)	Sa (g)
0.2	1.843 (SMs, Site Class D)
1.0	0.929 (SM1, Site Class D)

Conterminous 48 States
2005 ASCE 7 Standard
Latitude = 34.068053
Longitude = -118.41667399999999
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

Period (sec)	Sa (g)
0.2	1.229 (SDs, Site Class D)
1.0	0.620 (SD1, Site Class D)

WESTERN COORD

Conterminous 48 States
2005 ASCE 7 Standard
Latitude = 34.067222
Longitude = -118.4163
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

Period (sec)	Sa (g)
0.2	1.847 (Ss, Site Class B)
1.0	0.622 (S1, Site Class B)

Conterminous 48 States
2005 ASCE 7 Standard
Latitude = 34.067222
Longitude = -118.4163
Spectral Response Accelerations SMs and SM1
SMs = Fa x Ss and SM1 = Fv x S1
Site Class D - Fa = 1.0 ,Fv = 1.5

Period (sec)	Sa (g)
0.2	1.847 (SMs, Site Class D)
1.0	0.933 (SM1, Site Class D)

Conterminous 48 States
2005 ASCE 7 Standard
Latitude = 34.067222
Longitude = -118.4163
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

Period (sec)	Sa (g)
0.2	1.232 (SDs, Site Class D)
1.0	0.622 (SD1, Site Class D)

SOUTHERN COORD

A.1 Structural calculations – cont

EL RODEO - SMP
Fri Dec 06 11:58:50 PST 2013

Conterminous 48 States
2005 ASCE 7 Standard
Latitude = 34.068713
Longitude = -118.41628699999998
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

Period (sec)	Sa (g)	
0.2	1.836	(Ss, Site Class B)
1.0	0.617	(S1, Site Class B)

Conterminous 48 States
2005 ASCE 7 Standard
Latitude = 34.068713
Longitude = -118.41628699999998
Spectral Response Accelerations SMs and SM1
SMs = Fa x Ss and SM1 = Fv x S1
Site Class D - Fa = 1.0 ,Fv = 1.5

Period (sec)	Sa (g)	
0.2	1.836	(SMs, Site Class D)
1.0	0.925	(SM1, Site Class D)

Conterminous 48 States
2005 ASCE 7 Standard
Latitude = 34.068713
Longitude = -118.41628699999998
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

Period (sec)	Sa (g)	
0.2	1.224	(SDs, Site Class D)
1.0	0.617	(SD1, Site Class D)

NORTHERN COOR'D

Conterminous 48 States
2005 ASCE 7 Standard
Latitude = 34.067637
Longitude = -118.415743
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

Period (sec)	Sa (g)	
0.2	1.841	(Ss, Site Class B)
1.0	0.619	(S1, Site Class B)

Conterminous 48 States
2005 ASCE 7 Standard
Latitude = 34.067637
Longitude = -118.415743
Spectral Response Accelerations SMs and SM1
SMs = Fa x Ss and SM1 = Fv x S1
Site Class D - Fa = 1.0 ,Fv = 1.5

Period (sec)	Sa (g)	
0.2	1.841	(SMs, Site Class D)
1.0	0.929	(SM1, Site Class D)

Conterminous 48 States
2005 ASCE 7 Standard
Latitude = 34.067637
Longitude = -118.415743
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

Period (sec)	Sa (g)	
0.2	1.227	(SDs, Site Class D)
1.0	0.619	(SD1, Site Class D)

EASTERN COOR'D

A.1 Structural calculations – cont

5.4 Site-Specified Ground Motion

Ground Motion at the project sites was evaluated in accordance with the requirements of ASCE/SEI 31-03 (formerly FEMA 310). Earthquake ground motion is based upon an earthquake hazard level that is equal to 2/3 of the Maximum Considered Earthquake (MCE). The MCE is defined as an earthquake hazard based on a probability of exceedance of 2 percent in a 50-year exposure period (approximately 2500 year return period). At 2/3 of the MCE level, as used in these building evaluations, the design earthquake is typically roughly similar to the earthquake hazard based on a probability of exceedance of 10 percent in a 50-year period (approximately 475 year return period). The 475 year earthquake is the defined earthquake ground motion that has been used for decades (until recently) for the design of buildings. The MCE Response Spectrum is defined by two values obtained from 2002 USGS study for rock; S_S and S_1, the Short-Period Spectral Response Acceleration and Spectral Response Acceleration at one second, respectively. The Ss parameter is defined at 0.2 seconds; both the S_S and S_1 values are listed in the table below. Actual spectral *design* values are modified for Site Class D (stiff soil), which is appropriate for each of the school sites. The Peak Ground Acceleration for a 475 year return period, Short-Period *Design* Spectral Response Acceleration Parameter (S_{DS}), and *Design* Spectral Response Acceleration Parameter at one second (S_{D1}) are also provided in the following table:

SITE SPECTRAL RESPONSE ACCELERATION PARAMETERS

School	PGA	S_S	S_1	S_{DS}	S_{D1}
El Rodeo	0.42g	1.840g	0.619g	1.227g	0.62g
Hawthorne	0.43g	1.760g	0.601g	1.173g	0.60g
Horace Mann	0.43g	1.698g	0.652g	1.132g	0.65g
Beverly Vista	0.43g	1.727g	0.652g	1.151g	0.65g

PG. 28
SEISMIC RISK EVALUATION REPORT DATED 11/11/2008
MHP INC.
JN: 08.0897.002

A.1 Structural calculations – cont

SPECTRAL ACCELERATION - ASCE 31, §3.5.2.3.1

$Sa_{0.2,1.0} \Rightarrow T = C_t \times h_n^{\beta}$ (EQ. 3-8)

$C_t = 0.020$ ALL BLDGS ("ALL OTHER FRMG")

$h_n = \sim 35'$ BLDG A, C, D, E } TO ROOF FR. APPROX. GRADE

$= \sim 20'$ BLDG. B

$\beta = 0.75$ ALL BLDGS ("ALL OTHER FRMG")

$= 0.288s$, TYP

$= 0.189s$, BLDG. B

$Sa_{0.2} \Rightarrow$ @ SHORT PERIODS

$= S_{DS}$, (EER §3.3)

$\therefore Sa_{0.2} = 1.232$

$Sa_{1.0} \Rightarrow$ @ 1.0 SEC. PERIODS

$= \min[S_{DS}, S_{D1}/T]$, (EER §3.3)

$\checkmark S_{D1}/T = 2.160$

$= 3.291$

$\leq S_{DS}$

$= S_{DS}$,

$\therefore Sa_{1.0} = 1.232$

** LET $S_a = S_{DS}$

$= 1.232$

PSEUDO LATERAL FORCE - ASCE 31, §3.5.2.1

$V = C \cdot S_a \cdot W$ (EQ. 3-1)

$C \Rightarrow$ MAJORITY OF ALL BLDGS' SFRS IS CONC SHEAR WALLS. ∴ USE AS BASIS FOR 'C'

$= 1.2$, (TBL. 3-4, APPROX. 2 STORIES)

$S_a = S_{DS}$

$= 1.232$, (SEE PREV.)

$\therefore V = 1.48 \times W$

** LET $C' = 1.48$

A.2 Evaluation statement notes

✓EER, §5. ASCE 31 EVAL STATEMENTS
"REINFORCING STEEL" (CONC), SWs

SEE SET ID D4, 'GEN. NOTES'/ST-1
✓12" SWs (S-1.3, @ BLDG C)

12" THICK	VERT'L REINF A_s	GROSS A_g	RATIO A_s/A_g	HORIZ'L REINF A_s	GROSS A_g	RATIO A_s/A_g
t = 12 IN	0.40 IN² (2-#4)	18" o.c. × t = 216 IN²	.0019	0.40 IN² (2-#4)	13" o.c. × t = 156 IN²	.0026
			✓, ≥ .0015			✓, ≥ .0025

NOTE: SPAC'G ≤ 18" o.c., TYP.

∴ CHECK IS COMPLIANT

✓EER, §5. ASCE 31 EVAL. STATEMENTS
"REINFORCING STEEL" (CMU), SWs

SEE SET ID D4, 'GEN. NOTES'/ST-1

THICKNESS t	VERT'L REINF. A_s	GROSS A_g		RATIO A_s/A_g	HORIZ'L REINF. A_s	GROSS A_g	RATIO A_s/A_g
5.625 IN (6" CMU)	0.2 IN² (#4)	16" o.c. × t	= 90 IN²	.0022	0.2 IN² (#4)	24" o.c. × t = 135 IN²	.0015
7.625 IN (8" ")	" (")	" × "	= 122 "	.0016	" (")	16" " × " = 122 "	.0016
11.625 IN (12")	0.62 " (2-#5)	32" " × "	= 372 "	.0017	0.65 " (2-#5)	32" " × " = 372 "	.0017

** = RATIO ≤ 0.002

NOTE: ① ALL RATIOS ≥ 0.0007
② " SPAC'G ≤ 48" o.c.

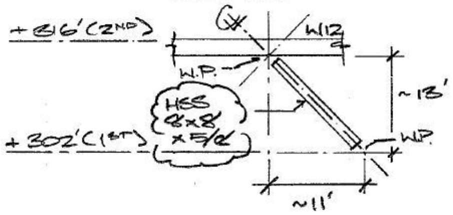
∴ CHECK IS NON-COMPLIANT

A.2 Evaluation statement notes – cont.

✓ EER, §5. ASCE 31 EVAL STATEMENTS
"SLENDERNESS OF DIAGONALS", BRACE FRAMES

SEE SET ID D4, A/S-5.2

✓ SECOND FLR BAY



$\ell = [13^2 + 11^2]^{1/2} = 17.03$ FT

$K \Rightarrow$ PIN-PIN $= 1.0$

$r \Rightarrow$ HSS8x8x5/16 $= 2.99$ IN

$K\ell/r = 68.85 < 120$ ✓

BY INSPECTION, ALL OTHER BAYS OK

∴ CHECK IS COMPLIANT

A.2 Evaluation statement notes – cont.

√ EER §5 ASCE 31 EVAL. STATEMENTS
"WALL ANCHORAGE", CONN'S

PER. §3.5.37

HORIZ. SEISMIC FORCE (TYP)

$T_c \Rightarrow ? = 0.9$ LIFE SAFETY

$S_{DS} = 1.232$ SEE PREV.

$w_p' = W_p \times A_p$

= TRIBUTARY WT TO O.O.P ANCH.

$= ? \cdot S_{DS} \cdot w_p'$

$= \underline{1.11 \times w_p'}$

√ SET ID D4, DTL. 2/S-5.1

@ 2ND FLOOR, BLDG. C (D4, S-21.3)

$w_p' \Rightarrow$ 1ST – 2ND = |+302' – +316'| = 14 FT
2ND – ROOF = |+316' – +328.7'| = 12.7 FT } $\underline{T_H}$ = 14'/2 + 12.7'/2 = ~13.35 FT

⇒ SEE PART'L PLAN (S-2.1.3) } CRITICAL $\underline{T_W}$ = ~16.00 FT

⇒ ~14" W. CONC.* = ~1'2"/12" × 150 PCF ≈ = 0.175 KSF

= $\underline{0.175\ KSF}$ × (13.35' × 16' = ~$\underline{214\ SF}$)   * 1'2" = SET ID D1, 103

= 37.236 KIPS/ANCH

$\therefore T_c = 1.11 \times w_p'$
$= \underline{41.3\ KIPS/ANCH.}$;

√ 2/S-5.1 USES (4) 1"Ø × 6" EMBED HILTI KB1 EXP. ANCH (NO LONGER PRODUCE)

TRY 1"Ø × 6" EMBED $\underline{KB\ III}$/3

N_{PN}, UNCRKED = 15.6K,
× ASSUME Ø = MIN. 0.6
= 9.4 KIPS, × 4
= $\underline{37.6\ KIPS}$, < T_c

EXTREMELY IDEAL SIT. → { FOR KB3 BOLTS, UNCRACKED CONC. ASSUMING NO EDGE OR GROUP EFFECTS

$\boxed{\text{∴ BY INSPECTION CHECK IS NON-COMPLIANT}}$

A.3 Photographs and details

Picture 1 *– Top of Wall to Roof Condition in Classroom Ceiling Plenum – Building B*
(showing Simpson A35 retrofit, ply'd shtg, lack of OOP anch's)

Picture 2 *– Top of Wall to Roof Condition in Classroom Ceiling Plenum – Building B*
(showing Simpson A35 retrofit, ply'd shtg, lack of OOP anch's)

A.3 Photographs and details – (cont.)

***Picture 3** – Wall to Floor Condition in Classroom Ceiling Plenum – Building C*
(showing lack of OOP anch's)

***Picture 4** – Building C (conc. wall) to Building D (wood framing) Condition at in Hallway Ceiling Plenum – 1st Floor*
(showing seismic separation)

A.3 Photographs and details – (cont.)

***Picture 5** – Roof to Top of Wall Condition – Building B*
(showing angle and ply'd retrofit)

***Picture 6** – Roof Plywood Sheathing to Diagonal Sheathing Condition – Building B*
(showing angle, ply'd, blocking retrofit)

ELIGIBILITY EVALUATION REPORT

School District:	BEVERLY HILLS UNIFIED SCHOOL DISTRICT	Original Report Date:	DEC. 20, 2013
School Campus:	EL RODEO SCHOOL (K-8)		
School Address:	605 WHITTIER DRIVE BEVERLY HILLS, CA 90210	Last Revision Date:	
Building Name/ID:	D		
Project Tracking No.:			Page 1 of 33

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

Appendix A.1. Structural calculations

KANDA & TSO, ASSOCIATES – INC. KANDA | TSO

SE Firm Name (Logo optional)

SE Address: **511 MISSION STREET**
SOUTH PASADENA, CA 91030

Phone: **(626) 441-1211 / www.kandatso.com / LesTso@KandaTso.com**
(website or email address optional)

LESLIE TSO
Name of SE whose stamp is above

1. Eligibility Check Summary

	YES	NO
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 = 1.847

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

☐ LIQUEFACTION ☐ SLOPE STABILITY FAILURE ☐ SURFACE FAULT RUPTURE

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

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[1], 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.

[1] *Seismic Evaluation of Existing Buildings* (ASCE/SEI 31-03), American Society of Civil Engineers, 2003.

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.[2]
- 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.

[2] *2007 California Building Standards Administrative Code (California Code of Regulations, Title 24 Part 1)*, Chapter 6, "Seismic Evaluation Procedures for Hospital Buildings," Section 1.4.5.1.2, October 23, 2008 Emergency Supplement.

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 (and elsewhere) of this report.

SET ID	DATE	DESCRIPTION
D1	DSA Approved Sept. 25, 1962 (App No 22744)	**'ADDITIONS TO BE CONSTRUCTED AT EL RODEO ELEMENTARY SCHOOL'** William Shinderman, AIA – Architect Richard L. Brown, SE – Structural Engineer Original Construction Drawings (selected sheets) Sheets S-1, -2, -3, -4, -5, -6, -7, -8, -9 (9 total sheets)
D2	DSA Approved July 29, 1966 (App No 27533)	**'ADDITIONS AND ALTERATIONS - EL RODEO ELEMENTARY SCHOOL'**** Maurice H. Fleishman, AIA – Architect Goldsmith, Chi & Associates, Inc. – Structural Engineer Additions/Original Construction Drawings (selected sheets) Sheets S1; S-1; S2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13, 14, 15, 16, 17, 18, 19, 20 (21 total sheets)
D3	Dated Nov. 11, 2008	**'SEISMIC RISK EVALUATION BEVERLY HILLS K-8 SCHOOLS'** MHP, Inc – Structural Engineer Report (42 total pages)

****Set ID D2 shows construction of new 1966 building, Building E, as well as additions/alterations to Building D (1962) of this report, and Building ABC (1927). Building ABC is being submitted with its own EER. Set ID D2 is included for reference to said additions/alterations -- Building E itself is not being submitted with its own EER being that it does not have critical deficiencies associated with high potential for local/global collapse in Section 4 and 5.**

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:	**November 8th, 13th, 2013**
Visiting engineer(s) and staff:	**Les Tso, S.E.; Casey Piedra, P.E. - KTA**
School district contact person:	**Charlotte Clement, Chief Facilities Officer - BHUSD**
School campus representative (if different than above):	**(same)**

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
☐ OTHER:

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

SET ID	CONDITION SHOWN ON PLANS	CONDITION OBSERVED AT SITE VISIT
-none-	**-none-**	**-none-**

3. Site and Building Description

3.1 Building description

General

Year originally built: **1962**

DSA Application number: **22744** ☒ Original Construction ☐ Work done pursuant to the Garrison Act (Ed Code 17367)

Number of stories above grade: **Two (2)**

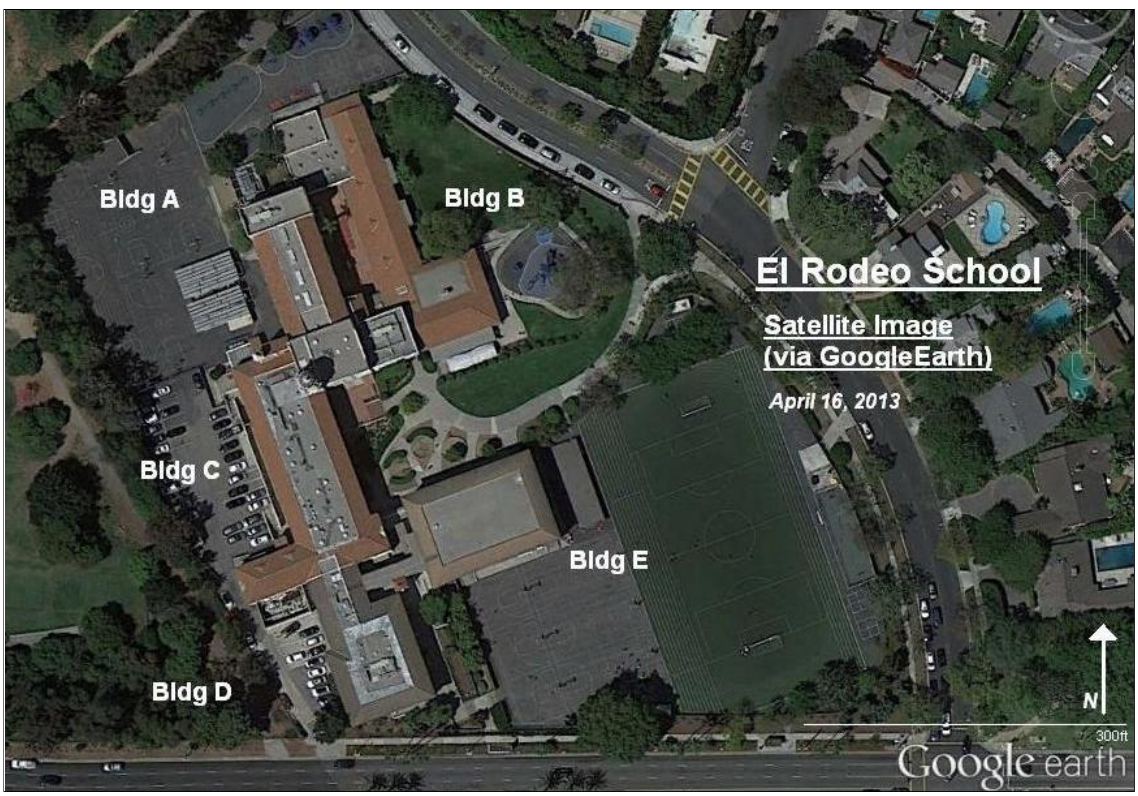
Number of stories below grade: **One (1)/partially subterranean**

Total floor area (sq ft, approx): **6,930 sq ft (footprint)**

Other essentially identical buildings on this campus? ☐ Yes ☒ No

Note: **Building D of this report is seismically separated from Building E and Building ABC (note; Building ABC consists of three contiguous 'wings' -- A, B, and C; no seismic separation is present between the wings). Building ABC is being submitted with its own EER. **See note on page 5 regarding note on Building E.**

Photographs



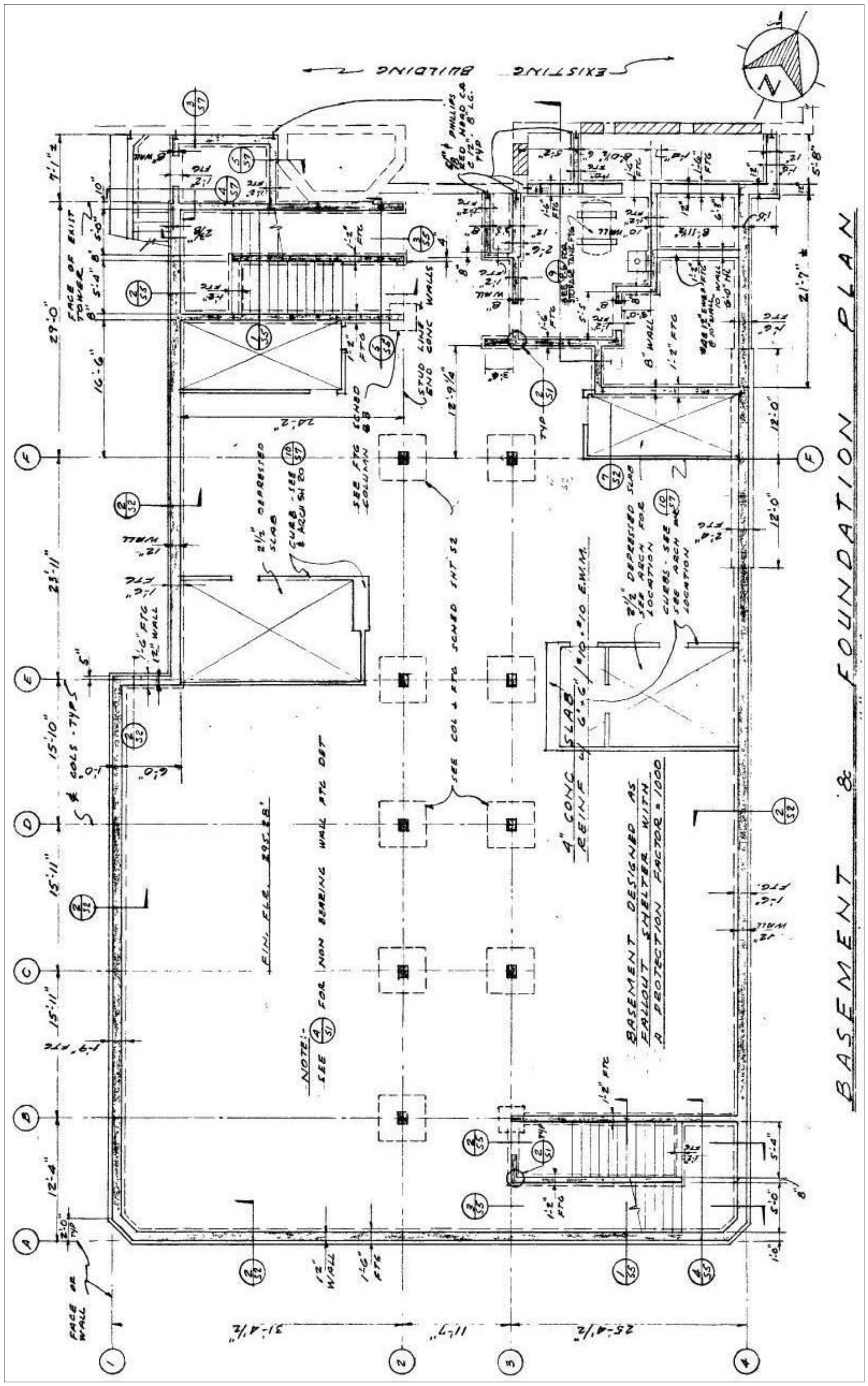
***Photograph 1** - Campus Satellite Image*

Take April 16, 2013 (Via GoogleEarth v7.1.2.2041)

***Photograph 2** – Southern and Western Elevation (looking North-East)*
Bldg D (foreground) and Bldg C (left, background)
Taken April 2013 (via GoogleMaps.com)

***Photograph 3** – Southern and Eastern Elevation (looking North-West)*
Building D; taken April 2013 (via GoogleMaps.com)

Ground Floor Plan



Ground/Basement Floor Plan – Bldg D
Sheet S-2, Set D1

3.2 Building Occupancy

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

	Original Use	Current Use	Planned Future Use
Office / Administration	☐	☐	☐
Classrooms / Instruction Areas	☒	☒	☒
Kitchen	☐	☐	☐
Assembly: Dining	☐	☐	☐
Assembly: Auditorium	☐	☐	☐
Assembly: Gymnasium	☐	☐	☐
Locker rooms	☒	☒	☒
Patio cover / bus shelter / walkway cover	☐	☐	☐
Bleachers / stadium structure	☐	☐	☐
Other occupied	☐	☐	☐
Mechanical / utility rooms or enclosures	☒	☒	☒
Bulk storage	☐	☐	☐
Vacant / unused	☐	☐	☐
Other unoccupied	☐	☐	☐

3.3 Seismicity

Latitude: **34.067222°N**

Longitude: **-118.4163°W**

Site Class per ASCE 31, Section 3.5.2.3: **Site Class D**

Basis for Site Class determination: **By Default**

****See App A.1 for analysis**

Period [sec]	Mapped MCE values from ASCE 7-05 [g]	Site Coefficients from ASCE 31 Tables 3-5, 3-6	Design values per ASCE 31 section 3.5.2.3.1 [g]	S_a per ASCE 31 section 3.5.2.3.1, [g]
0.2	S_S = **1.847**	F_a = **1.000**	$S_{DS} = (2/3)\, S_S\, F_a$ = **1.232**	$S_{a,0.2} = S_{DS}$ = **1.232**
1.0	S_1 = **0.622**	F_v = **1.500**	$S_{D1} = (2/3)\, S_1\, F_v$ = **0.622**	$S_{a,1.0} = \min(S_{DS}, S_{D1}/T)$ = **1.232**

3.4 Gravity System

Roof diaphragm and framing: **Plywood sheathing over wood rafters/joists to wood stud bearing (shear) walls.**

Typical floor diaphragm and framing: **Plywood sheathing over wood rafters/joists to wood stud bearing (shear) and reinforced concrete bearing (shear) walls.**

First floor framing: Reinforced (all) concrete slab over concrete beams to concrete bearing (shear) walls.

Basement framing: Reinforced concrete slab on grade.

Vertical load-bearing elements: **Reinforced concrete bearing/shear walls, wood stud bearing/shear walls, wood posts and reinforced concrete columns.**

Basement walls: **Reinforced concrete bearing/shear walls.**

Foundation: **Reinforced concrete spread/pad footings.**

Snow load for use in load combinations involving earthquake: **-not applicable-**

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

	North-South	East-West
C1 Concrete Moment Frames	☐	☐
C1B* Reinforced Concrete Cantilever Columns	☐	☐
C2A Concrete Shear Walls, Flexible Diaphragm	☐	☐
C3A Concrete Frame with Infill Masonry Shear Walls, Flexible Diaphragm	☐	☐
PC1 Precast/Tilt-up Concrete Shear Walls, Flexible Diaphragm	☐	☐
PC1A Precast/Tilt-up Concrete Shear Walls, Rigid Diaphragm	☐	☐
PC2 Precast Concrete Frames with Shear Walls, Rigid Diaphragm	☐	☐
PC2A Precast Concrete Frames without Shear Walls, Rigid Diaphragm	☐	☐
RM1 Reinforced Masonry Bearing Walls, Flexible Diaphragm	☐	☐
S1B* Steel Cantilever Columns	☐	☐
S3 Steel Light Frames	☐	☐
URM Unreinforced Masonry Bearing Walls, Flexible Diaphragm	☐	☐
URMA Unreinforced Masonry Bearing Walls, Rigid Diaphragm	☐	☐
M* Mixed Systems - construction containing at least one of the above lateral-force-resisting systems in at least one direction of seismic loading. **C2A/Wood Stud, Plywood SW** (mixed in both directions/at all stories)	☒	☒
None of the above	☐	☐

* 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).

3.5 Structural System – *cont.*

Horizontal system combinations	***(see below)***
Vertical system combinations	***(see below)***
SFRS foundation	*reinf. conc. pad/spread footings*
Gravity loading	*8"w reinf. conc. shear walls and 3/8" to 1/2" plywood over 2x6 stud shear walls support gravity loads in both directions (intermittent wood/conc. posts also support some gravity loads).*
System details	***Floor (Longitudinal)** - 8"w reinf. conc. shear walls connected to 5/8" ply'd diaphragm via 4x8 ledger w/ (2) 5/8"dia. anch. bolts at 4'o.c. (induces cross-grain bending; Set ID D1, Sheet S-6).* ***Floor (Transverse)** - 8"w reinf. conc. shear walls connected to 2x16 floor joists w/ hangers to 4x16 ledger w/ (2) 5/8"dia. anch. bolts at 4'o.c. (induces cross-grain bending, no other positive connection; Set ID D1, Sheet S-6).* *--similar detailing for **Roof (Lng. & Trnsv.)** – Set ID D1, Sheet S-5--*
Structural materials	*(Set ID D1, Sheet S-1, General Notes)* ***Concrete/Reinforcing*** *-3000psi 28-day strength Concrete (walls); 2-2500psi SOG/ftgs.* *-ASTM A-15, A-305 Rebars* ***Steel*** *-ASTM Spec. A-7, typical* ***Wood*** *-DLF, Construction Grade*
Original design code	*(Set ID D1, Sheet S-1, General Notes)* *'State of California Administrative Code, Title 21, Public Works'*
History of seismic retrofit or significant alteration	***-not applicable-***
Benchmark year check	***-not applicable-***

4. Deficiency list

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

Non-compliant condition	Discussion	Additional evaluation recommended
VERTICAL DISCONTI -NUITIES**	**By inspection, Tier 1 evaluation is non-compliant. Further analysis via Tier 2 may be warranted. Also, see note below**	*In-depth analysis of entire lateral system/Tier 2 Evaluation*
WALL OPENINGS	By inspection, longitudinal perimeter walls are non-compliant, as shown in Set ID D1, Sheet S-3	*None*
WALL ANCHORAGE**	**Comprehensive out-of-plane (OOP) anchorage of concrete shear walls above 1st floor to wood floor and roof diaphragms were not addressed in original construction drawings or subsequent. What anchorages are present from original construction are non-compliant with 'WOOD LEDGER' requirements, below.**	*None*
WOOD LEDGERS	Connections from original construction (Set ID D1, Details 11, 12/S-6), by inspection induce cross-grain bending in ledgers.	*None*

Unknown condition	Discussion	Additional evaluation recommended
-none	-none-	-none-

****Note**: <u>'**WALL ANCHORAGE**' considered key critical non-compliant deficiency</u>. Other unknown or non-compliant conditions were researched/analyzed but may warrant further investigation/analysis for complete evaluation. Also, **bold print** indicates critical condition.

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.

****Note: Description/Instructions of selected items marked C or NA are removed for brevity**

CONDITION OF MATERIALS	
$\boxed{\text{C}}$ 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.
$\boxed{\text{C}}$ 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.
$\boxed{\text{C}}$ 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 $\boxed{\text{NA}}$	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 $\boxed{\text{NA}}$	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.
$\boxed{\text{C}}$ NC U NA	MASONRY UNITS. There shall be no evidence of or reason to suspect structural capacity loss due to deterioration of masonry units.
$\boxed{\text{C}}$ 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.
$\boxed{\text{C}}$ NC U NA	MASONRY LAY-UP. Filled collar joints of multi-wythe masonry infill walls shall have negligible voids.

$\boxed{C}$ NC U NA	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	
$\boxed{C}$ NC U NA **Critical Item**	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.
$\boxed{C}$ NC U NA **Critical Item**	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.
$\boxed{C}$ NC U NA **Critical Item**	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.
$\boxed{C}$ NC U NA	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.
C NC $\boxed{U}$ NA **Critical Item**	**VERTICAL DISCONTINUITIES. All vertical elements of the seismic force-resisting system shall be continuous to the foundation.** **See **Note, Page 15, and description within Section 5**
$\boxed{C}$ NC U NA **Critical Item**	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.
$\boxed{C}$ NC U NA **Critical Item**	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.
$\boxed{C}$ NC U NA **Critical Item**	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. **Per ASCE 31, C4.3.1.2, buildings that are the same height and have matching floor levels need not comply with 4.3.1.2. Bldg C (of Bldg ABC) and Bldg D are such buildings.**
C NC U $\boxed{NA}$ **Critical Item**	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.

MOMENT FRAMES

**** No Moment Frames within SFRS. Descriptions within check-list removed for brevity ****

C NC U NA Critical Item	SHEAR STRESS CHECK (Columns)
C NC U NA Critical Item	AXIAL STRESS CHECK (Concrete columns)
C NC U NA	AXIAL STRESS CHECK (Steel columns)
C NC U NA Critical Item	FLAT SLAB FRAMES
C NC U NA	PRESTRESSED FRAME ELEMENTS
C NC U NA Critical Item	CAPTIVE COLUMNS
C NC U NA	NO SHEAR FAILURES
C NC U NA	STRONG COLUMN/WEAK BEAM
C NC U NA	STRONG COLUMN/WEAK BEAM
C NC U NA Critical Item	BEAM BARS
C NC U NA	COLUMN BAR SPLICES
C NC U NA	BEAM BAR SPLICES
C NC U NA	COLUMN TIE SPACING
C NC U NA	STIRRUP SPACING
C NC U NA	JOINT REINFORCING
C NC U NA	COMPLETE FRAMES
C NC U NA Critical Item	DEFLECTION COMPATIBILITY
C NC U NA Critical Item	FLAT SLABS
C NC U NA Critical Item	REDUNDANCY (Moment frame)
C NC U NA	INTERFERING WALLS

C NC U NA	PRECAST CONNECTION CHECK
C NC U NA	PRECAST FRAMES
C NC U NA	PRECAST CONNECTIONS
C NC U NA	DRIFT CHECK
C NC U NA	MOMENT-RESISTING CONNECTIONS
C NC U NA	PANEL ZONES
C NC U NA	COLUM SPLICES
C NC U NA	COMPACT MEMBERS
SHEAR WALLS	
C NC U NA **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.
C NC U NA **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 $2\sqrt{f'_c}$. **By inspection, assume shear stresses compliant. Note; majority of SFRS above 1st floor contains wood framed shear walls, with reinf. conc. shear walls at isolated ends of building (near stairway locations).**
C NC U NA	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. **By inspection, assume reinforcing steel compliant, and not of consequence to failure mechanism of SFRS. Also, see 'SHEAR STRESS CHECK' above.**
C NC U NA	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 135º or more.
C NC U NA **Critical Item**	REDUNDANCY (Shear wall). The number of lines of shear walls in each principal direction shall be greater than or equal to 2.
C NC U NA	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.)

C NC U $\boxed{\text{NA}}$	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.)
C NC U $\boxed{\text{NA}}$	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.)
C NC U $\boxed{\text{NA}}$ 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 $2\sqrt{f'_c}$.
C $\boxed{\text{NC}}$ U NA	**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.** **By inspection, see Set ID D1, Sheet S-3.**
C NC U $\boxed{\text{NA}}$	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.
C NC U $\boxed{\text{NA}}$ 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.
C NC U $\boxed{\text{NA}}$ 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.
C NC U $\boxed{\text{NA}}$	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
C NC U $\boxed{\text{NA}}$	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 **** No Braced Frames in SFRS. Descriptions within check-list removed for brevity ****	
C NC U $\boxed{\text{NA}}$ Critical Item	REDUNDANCY
C NC U $\boxed{\text{NA}}$ Critical Item	AXIAL STRESS CHECK
C NC U $\boxed{\text{NA}}$	SLENDERNESS OF DIAGONALS

C NC U NA	CONNECTION STRENGTH
C NC U NA	K-BRACING
DIAPHRAGMS	
C NC U NA	DIAPHRAGM CONTINUITY. The diaphragm shall not be composed of split-level floors and shall not have expansion joints.
C NC U NA	CROSS TIES. There shall be continuous cross ties between diaphragm chords.
C NC U NA	ROOF CHORD CONTINUITY. All roof chord elements shall be continuous, regardless of changes in roof elevation.
C NC U NA **Critical Item**	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.
C NC U NA	OPENINGS AT BRACED FRAMES. Diaphragm openings immediately adjacent to the braced frames shall extend less than 25% of the frame length.
C NC U NA	OTHER DIAPHRAGMS. The diaphragm shall not consist of a system other than wood, metal deck, concrete or horizontal bracing.
C NC U NA **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 **Critical Item**	**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.** **No anchorage to resist out-of-plane (OOP) forces exists. See description in Section 4.**

C $\boxed{\text{NC}}$ 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.** **See Set ID D1, 11 & 12/S-6.**
C NC U $\boxed{\text{NA}}$	PRECAST PANEL CONNECTIONS. There shall be at least two anchors from each precast wall panel into the diaphragm elements.
C NC U $\boxed{\text{NA}}$	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. **See 'WOOD LEDGERS' & 'WALL ANCHORAGE' checks above, and description within Section 4.**
C NC U $\boxed{\text{NA}}$	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 $\boxed{\text{NA}}$	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 $\boxed{\text{NA}}$	CORBEL BEARING. If precast concrete frame girders bear on column corbels, the length of bearing shall be greater than 3".
C NC U $\boxed{\text{NA}}$	CORBEL CONNECTIONS. Precast concrete frame girders shall not be connected to corbels with welded elements.
$\boxed{\text{C}}$ NC U NA	TRANSFER TO SHEAR WALLS. Diaphragms shall be connected for transfer of loads to shear walls.
C NC U $\boxed{\text{NA}}$	TRANSFER TO STEEL FRAMES. Diaphragms shall be connected for transfer of loads to the steel frames.
C NC U $\boxed{\text{NA}}$	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.
$\boxed{\text{C}}$ NC U NA	CONCRETE COLUMNS. All concrete columns shall be doweled into the foundation.
$\boxed{\text{C}}$ NC U NA	FOUNDATION DOWELS. Wall reinforcement shall be doweled into the foundation.
C NC U $\boxed{\text{NA}}$	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.
C NC U **NA**	WALL PANELS: Metal, fiberglass or cementitious wall panels shall be positively attached to the framing to resist seismic forces.
FOUNDATION	
C NC U **NA**	POLE FOUNDATIONS. Pole foundations shall have a minimum embedment depth of 4 ft.
C NC U **NA**	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 *Geologic Site Hazards not used for basis of eligibility. Descriptions within check-list removed for brevity*	
C NC U NA	LIQUEFACTION
C NC U NA	SLOPE FAILURE
C NC U NA	SURFACE FAULT RUPTURE

Appendices

A.1 Structural calculations

MCE SPECTRAL RESPONSE ACCELERATIO PARAMETER ANALYSIS

SEE PROCEEDING PAGES FOR LAT/LONG & AREA REFERENCE USED FOR ACCELERATIONS

CAMPUS AREA[3]	S_1	S_S	S_{D1}	S_{DS}
'CENTER' (PER USGS LAT./LNG.)[2]	0.619	1.841	0.619	1.227
'CENTER' (PER WEB. iTOUCHMAP)[1]	0.619	1.841	0.619	1.227
'CENTER' (PER MHP REPORT)	0.619	1.841	0.62	1.227
NORTHERN END[1,2]	0.617	1.836	0.617	1.224
SOUTHERN END[1,2]	0.622	1.847	0.622	1.232
EASTERN END	0.619	1.841	0.619	1.227
WESTERN END[1,2]	0.620	1.843	0.620	1.229

GOVERNING

∴ ACCELERATIONS

$S_1 = 0.622$

$S_S = 1.847$

$S_{D1} = 0.622$

$S_{DS} = 1.232$

NOTES: 1. USES 'iTOUCHMAP.COM' FOR LAT./LNG. COORD

2. USES USGS EQ GROUND MOTION PARAMETERS V.5.1.0 FOR ACCELERATIONS

3. ALL AREAS ASSUMED AS SITE CLASS D

A.1 Structural calculations – cont.

USGS
Geographic Names Information System (GNIS)

Stop! Do not bookmark or copy/paste this URL before reading FAQs.

Query Result FAQs

Feature Detail Report for: El Rodeo Elementary School

ID:	241922
Name:	El Rodeo Elementary School
Class:	School (Definitions)
Citation:	California Department of Education. Calfornia Public School Directory. Sacramento, California, 1994. 1994
Address:	605 Whittier Drive
City:	Beverly Hills
State:	CA
ZIP:	90210-3112
Entry Date:	14-Jun-2000
*Elevation:	302/92

*Elevations in feet/meters from the National Elevation Dataset

Mapping Services

- GNIS in ESRI Map
- USGS The National Map
- HomeTownLocator
- ACME Mapper 2.0
- Microsoft Virtual Earth
- TerraFly.com
- Find the Watershed
- MapQuest

USGS
GEOGRAPHIC NAMES INFORMATION SYSTEMS (GNIS)

LAT./LNG.
FOR
EL RODEO SCHOOL
(@SITE CODE)

Variant Names

Variant Name
El Rodeo School Citation

Counties

Sequence	County	Code	State	Code	Country
1	Los Angeles	037	California	06	US

Coordinates (One point per USGS topographic map containing the feature, NAD83)

Sequence	Latitude(DEC)	Longitude(DEC)	Latitude(DMS)	Longitude(DMS)	Map Name
1	34.0677700	-118.4158700	340404N	1182457W	Beverly Hills

U.S. Department of the Interior || U.S. Geological Survey
12201 Sunrise Valley Drive, Reston, VA 20192, USA
gnis_manager@usgs.gov
Form updated: November 19, 2013
USGS Privacy Policy and Disclaimers

A.1 Structural calculations – cont.

iTouchMap.com Mobile and Desktop Maps

Maps | Country - State | **Places** | Google Earth | **Cities** | **Lat - Long**

Home » Latitude and Longitude of a Point

To find the latitude and longitude of a point **Click** on the map, **Drag** the marker, or enter the...

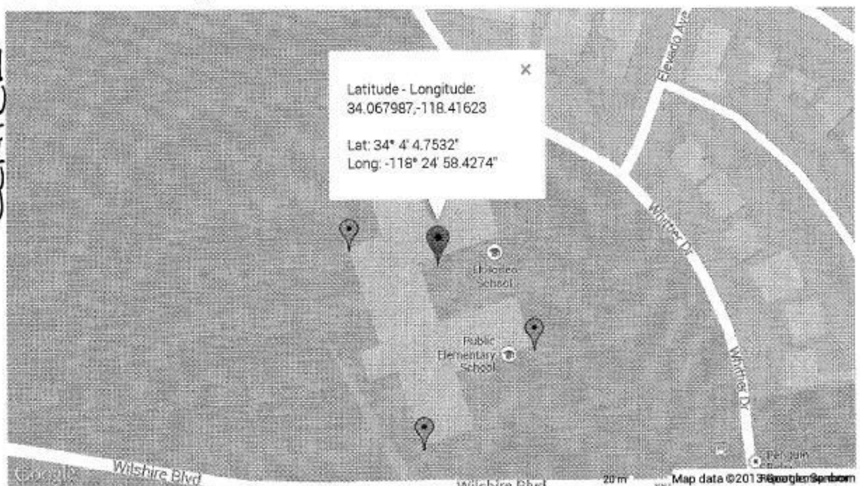
Address: 605 Whittier Blvd Beverly Hills, CA GO

Map Center: Get Address - Land Plat Size - Street View - Area Photographs

Try out 3D Google Earth. Google Earth gives you a 3D look of the area around the center of the map, which is usually your last click point, and includes latitude, longitude and elevation information.

Latitude and Longitude of a Point

CENTER



Clear / Reset | Remove Last Blue Marker | Center Red Marker

Get the Latitude and Longitude of a Point

When you click on the map, move the marker or enter an address the latitude and longitude coordinates of the point are inserted in the boxes below.

Latitude:	34.067987
Longitude:	-118.416230

	Degrees	Minutes	Seconds
Latitude:	34	4	4.7532
Longitude:	-118	24	58.4274

iTOUCHMAPS

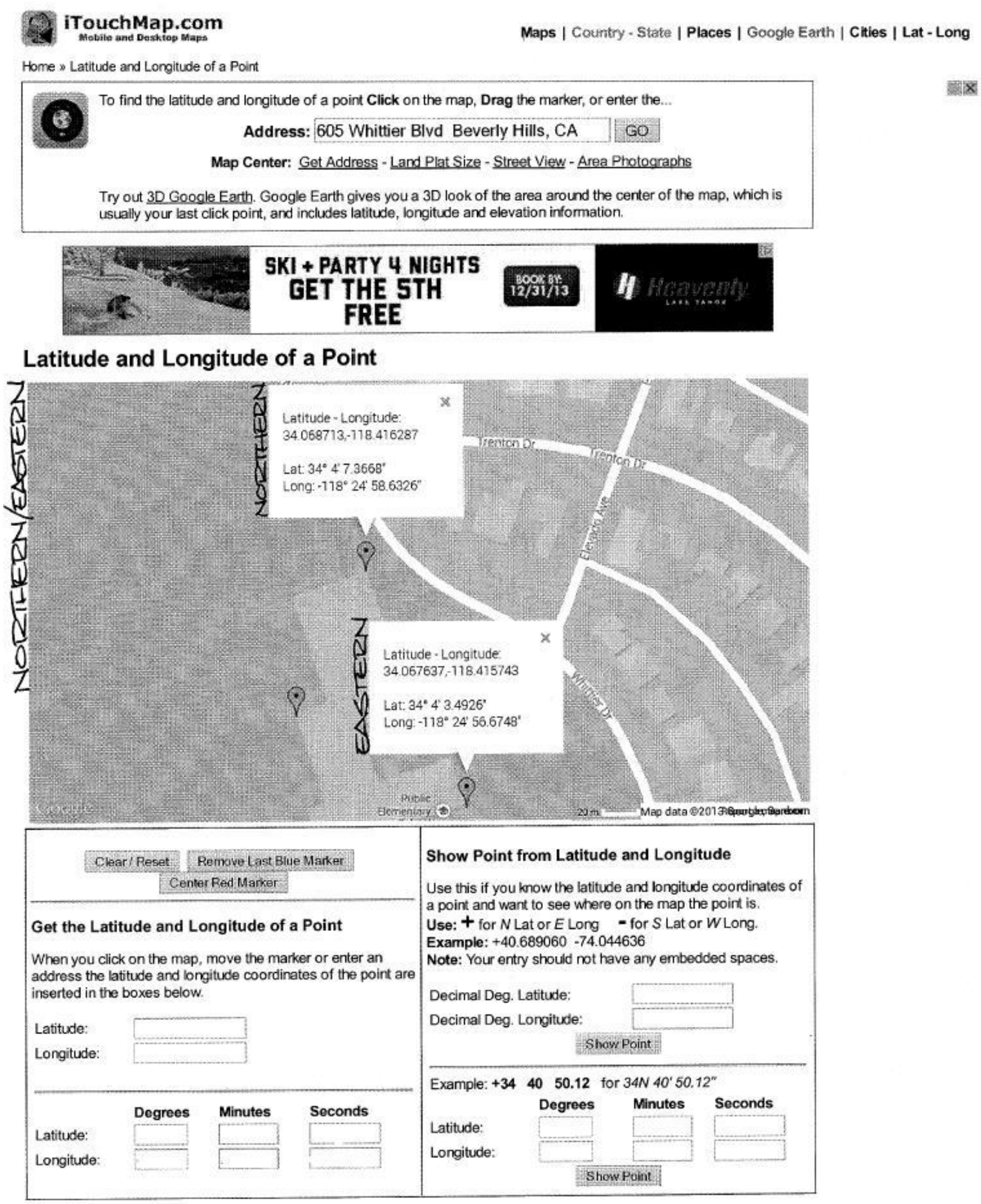
LAT. / LNG.

FOR

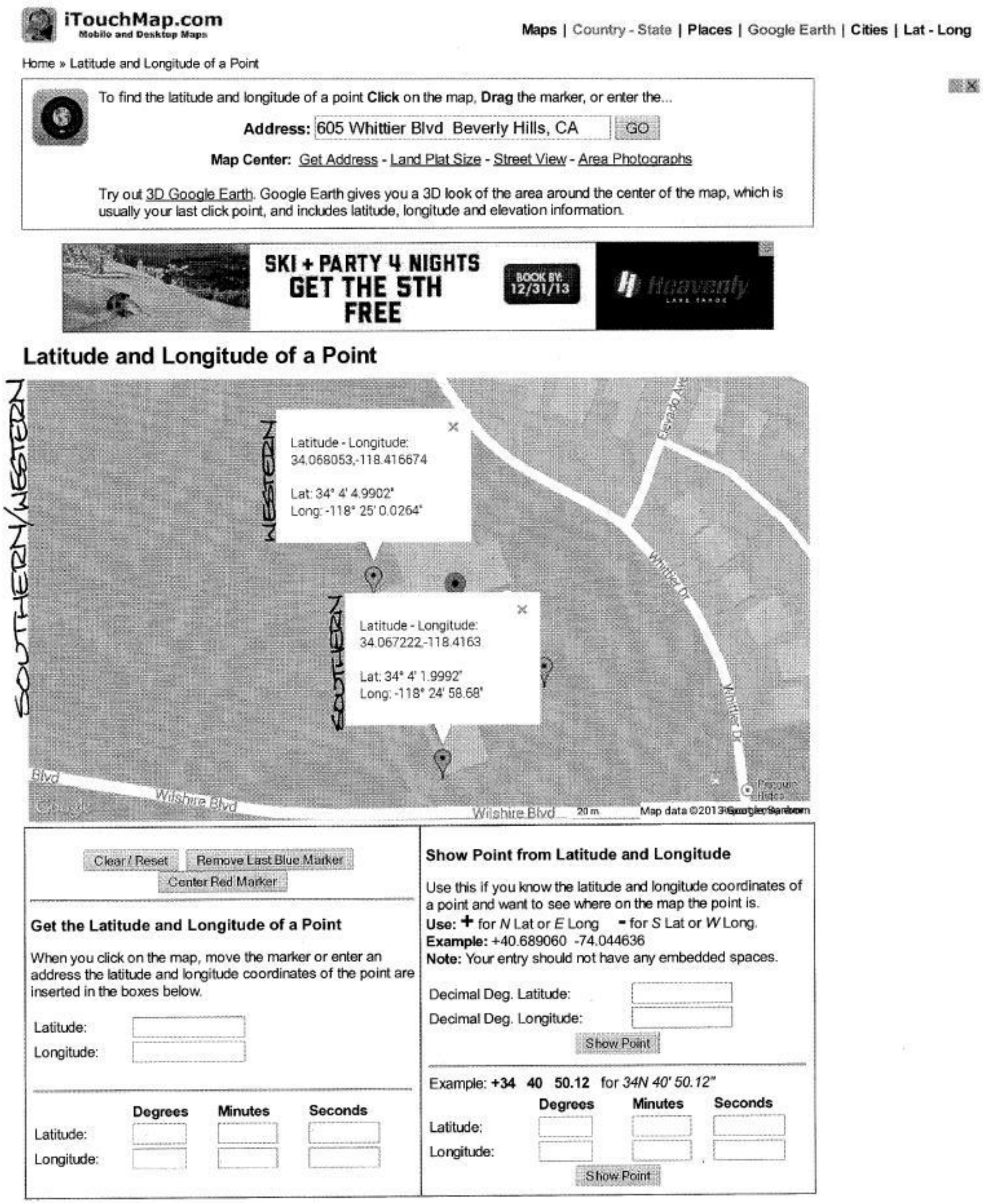
EL RODEO SCHOOL

(@ EXTREME BLDG/SITE ENDS)

A.1 Structural calculations – cont.



A.1 Structural calculations – cont.



A.1 Structural calculations – cont.

Project Name = EL RODEO - SMP
Date = Fri Dec 06 11:54:32 PST 2013

CENTER COOR'D (PER USGS)

Conterminous 48 States
2005 ASCE 7 Standard
Latitude = 34.06777
Longitude = -118.41587
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

Period (sec)	Sa (g)	
0.2	1.841	(Ss, Site Class B)
1.0	0.619	(S1, Site Class B)

Conterminous 48 States
2005 ASCE 7 Standard
Latitude = 34.06777
Longitude = -118.41587
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

Period (sec)	Sa (g)	
0.2	1.227	(SDs, Site Class D)
1.0	0.619	(SD1, Site Class D)

Conterminous 48 States
2005 ASCE 7 Standard
Latitude = 34.06777
Longitude = -118.41587
Spectral Response Accelerations SMs and SM1
SMs = Fa x Ss and SM1 = Fv x S1
Site Class D - Fa = 1.0 ,Fv = 1.5

Period (sec)	Sa (g)	
0.2	1.841	(SMs, Site Class D)
1.0	0.929	(SM1, Site Class D)

USGS

EARTHQUAKE GROUND MOTION PARAMETERS v5.1.0

SPECTRAL RESPONSE ACCELERATIONS: S_1, S_S; S_{M_1}, S_{MS}; S_{D_1}, S_{DS}

Project Name = EL RODEO - SMP
Date = Fri Dec 06 11:55:54 PST 2013

CENTER COOR'D (PER iTOUCHMAPS)

FOR EL RODEO SCHOOL

Conterminous 48 States
2005 ASCE 7 Standard
Latitude = 34.067987
Longitude = -118.41622999999998
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

Period (sec)	Sa (g)	
0.2	1.841	(Ss, Site Class B)
1.0	0.619	(S1, Site Class B)

Conterminous 48 States
2005 ASCE 7 Standard
Latitude = 34.067987
Longitude = -118.41622999999998
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

Period (sec)	Sa (g)	
0.2	1.227	(SDs, Site Class D)
1.0	0.619	(SD1, Site Class D)

Conterminous 48 States
2005 ASCE 7 Standard
Latitude = 34.067987
Longitude = -118.41622999999998
Spectral Response Accelerations SMs and SM1
SMs = Fa x Ss and SM1 = Fv x S1
Site Class D - Fa = 1.0 ,Fv = 1.5

Period (sec)	Sa (g)	
0.2	1.841	(SMs, Site Class D)
1.0	0.929	(SM1, Site Class D)

A.1 Structural calculations – cont.

```
Conterminous 48 States
2005 ASCE 7 Standard
Latitude = 34.068053
Longitude = -118.41667399999999
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
  Period    Sa
  (sec)     (g)
   0.2     1.843 (Ss, Site Class B)
   1.0     0.620 (S1, Site Class B)

Conterminous 48 States
2005 ASCE 7 Standard
Latitude = 34.068053
Longitude = -118.41667399999999
Spectral Response Accelerations SMs and SM1
SMs = Fa x Ss and SM1 = Fv x S1
Site Class D -  Fa = 1.0 ,Fv = 1.5

  Period    Sa
  (sec)     (g)
   0.2     1.843 (SMs, Site Class D)
   1.0     0.929 (SM1, Site Class D)

Conterminous 48 States
2005 ASCE 7 Standard
Latitude = 34.068053
Longitude = -118.41667399999999
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

  Period    Sa
  (sec)     (g)
   0.2     1.229 (SDs, Site Class D)
   1.0     0.620 (SD1, Site Class D)
```

WESTERN COOR'D

```
Conterminous 48 States
2005 ASCE 7 Standard
Latitude = 34.067222
Longitude = -118.4163
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
  Period    Sa
  (sec)     (g)
   0.2     1.847 (Ss, Site Class B)
   1.0     0.622 (S1, Site Class B)

Conterminous 48 States
2005 ASCE 7 Standard
Latitude = 34.067222
Longitude = -118.4163
Spectral Response Accelerations SMs and SM1
SMs = Fa x Ss and SM1 = Fv x S1
Site Class D -  Fa = 1.0 ,Fv = 1.5

  Period    Sa
  (sec)     (g)
   0.2     1.847 (SMs, Site Class D)
   1.0     0.933 (SM1, Site Class D)

Conterminous 48 States
2005 ASCE 7 Standard
Latitude = 34.067222
Longitude = -118.4163
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

  Period    Sa
  (sec)     (g)
   0.2     1.232 (SDs, Site Class D)
   1.0     0.622 (SD1, Site Class D)
```

SOUTHERN COOR'D

A.1 Structural calculations – cont

EL RODEO - SMP
Fri Dec 06 11:58:50 PST 2013

Conterminous 48 States
2005 ASCE 7 Standard
Latitude = 34.068713
Longitude = -118.41628699999998
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

Period (sec)	Sa (g)	
0.2	1.836	(Ss, Site Class B)
1.0	0.617	(S1, Site Class B)

Conterminous 48 States
2005 ASCE 7 Standard
Latitude = 34.068713
Longitude = -118.41628699999998
Spectral Response Accelerations SMs and SM1
SMs = Fa x Ss and SM1 = Fv x S1
Site Class D - Fa = 1.0 ,Fv = 1.5

Period (sec)	Sa (g)	
0.2	1.836	(SMs, Site Class D)
1.0	0.925	(SM1, Site Class D)

Conterminous 48 States
2005 ASCE 7 Standard
Latitude = 34.068713
Longitude = -118.41628699999998
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

Period (sec)	Sa (g)	
0.2	1.224	(SDs, Site Class D)
1.0	0.617	(SD1, Site Class D)

NORTHERN COOR'D

Conterminous 48 States
2005 ASCE 7 Standard
Latitude = 34.067637
Longitude = -118.415743
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

Period (sec)	Sa (g)	
0.2	1.841	(Ss, Site Class B)
1.0	0.619	(S1, Site Class B)

Conterminous 48 States
2005 ASCE 7 Standard
Latitude = 34.067637
Longitude = -118.415743
Spectral Response Accelerations SMs and SM1
SMs = Fa x Ss and SM1 = Fv x S1
Site Class D - Fa = 1.0 ,Fv = 1.5

Period (sec)	Sa (g)	
0.2	1.841	(SMs, Site Class D)
1.0	0.929	(SM1, Site Class D)

Conterminous 48 States
2005 ASCE 7 Standard
Latitude = 34.067637
Longitude = -118.415743
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

Period (sec)	Sa (g)	
0.2	1.227	(SDs, Site Class D)
1.0	0.619	(SD1, Site Class D)

EASTERN COOR'D

ELIGIBILITY EVALUATION REPORT

School District:	BEVERLY HILLS UNIFIED SCHOOL DISTRICT	Original Report Date:	DEC. 20, 2013
School Campus:	EL RODEO SCHOOL (K-8)		
School Address:	605 WHITTIER DRIVE BEVERLY HILLS, CA 90210	Last Revision Date:	
Building Name/ID:	D		
Project Tracking No.:			

A.1 Structural calculations – cont

5.4 Site-Specified Ground Motion

Ground Motion at the project sites was evaluated in accordance with the requirements of ASCE/SEI 31-03 (formerly FEMA 310). Earthquake ground motion is based upon an earthquake hazard level that is equal to 2/3 of the Maximum Considered Earthquake (MCE). The MCE is defined as an earthquake hazard based on a probability of exceedance of 2 percent in a 50-year exposure period (approximately 2500 year return period). At 2/3 of the MCE level, as used in these building evaluations, the design earthquake is typically roughly similar to the earthquake hazard based on a probability of exceedance of 10 percent in a 50-year period (approximately 475 year return period). The 475 year earthquake is the defined earthquake ground motion that has been used for decades (until recently) for the design of buildings. The MCE Response Spectrum is defined by two values obtained from 2002 USGS study for rock; S_S and S_1, the Short-Period Spectral Response Acceleration and Spectral Response Acceleration at one second, respectively. The Ss parameter is defined at 0.2 seconds; both the S_S and S_1 values are listed in the table below. Actual spectral *design* values are modified for Site Class D (stiff soil), which is appropriate for each of the school sites. The Peak Ground Acceleration for a 475 year return period, Short-Period *Design* Spectral Response Acceleration Parameter (S_{DS}), and *Design* Spectral Response Acceleration Parameter at one second (S_{D1}) are also provided in the following table:

SITE SPECTRAL RESPONSE ACCELERATION PARAMETERS

School	PGA	S_S	S_1	S_{DS}	S_{D1}
El Rodeo	0.42g	1.840g	0.619g	1.227g	0.62g
Hawthorne	0.43g	1.760g	0.601g	1.173g	0.60g
Horace Mann	0.43g	1.698g	0.652g	1.132g	0.65g
Beverly Vista	0.43g	1.727g	0.652g	1.151g	0.65g

PG. 28
SEISMIC RISK
EVALUATION
REPORT
DATED 11/11/2008
MHP INC.
JN: 08.0897.002

A.1 Structural calculations – cont

$\boxed{\text{SPECTRAL ACCELERATION}}$ - ASCE 31, §3.5.2.3.1

$S_{a_{0.2,1.0}} \Rightarrow \quad T = C_t \times h_n^{\beta}$ (EQ. 3-8)

$C_t = 0.020$ ALL BLDGS ("ALL OTHER FRMG")

$h_n = \sim 35'$ BLDG A, C, D, E } TO ROOF FR. APPROX. GRADE

$= \sim 20'$ BLDG. B

$\beta = 0.75$ ALL BLDGS ("ALL OTHER FRMG")

$= 0.288 s$, TYP

$= 0.189 s$, BLDG. B

$S_{a_{0.2}} \Rightarrow$ @ SHORT PERIODS

$= S_{DS}$, (EER §3.3)

$\therefore S_{a_{0.2}} = 1.232$

$S_{a_{1.0}} \Rightarrow$ @ 1.0 SEC. PERIODS

$= \min\left[S_{DS}, S_{D1}/T\right]$, (EER §3.3)

$\checkmark S_{D1}/T = 2.160$

$= 3.291$

$\leq S_{DS}$

$= S_{DS}$,

$\therefore S_{a_{1.0}} = 1.232$

** LET $S_a = S_{DS}$

$= 1.232$

$\boxed{\text{PSEUDO LATERAL FORCE}}$ - ASCE 31, §3.5.2.1

$V = C \cdot S_a \cdot W$ (EQ. 3-1)

$C \Rightarrow$ MAJORITY OF ALL BLDGS' SFRS IS CONC SHEAR WALLS. ∴ USE AS BASIS FOR 'C'

$= 1.2$, (TBL. 3-4, APPROX. 2 STORIES)

$\therefore V = 1.48 \times W$

$S_a = S_{DS}$

$= 1.232$, (SEE PREV.)

** LET $C' = 1.48$

DSA
DIVISION OF THE STATE ARCHITECT
DEPARTMENT OF GENERAL SERVICES

STATE OF CALIFORNIA
GOVERNOR EDMUND G. BROWN JR.

April 24, 2014

Dr. Gary Woods
District Superintendent
Beverly Hills Unified School District
255 South Lasky Drive
Beverly Hills, CA 90212

EVALUATION OF ELIGIBILITY FOR SEISMIC MITIGATION PROGRAM: DSA CONCURRENCE
Project Tracking Number: 64311-49 (Please use this number for all future correspondence).

Dear Dr. Woods,

The Division of the State Architect (DSA) has received the Eligibility Evaluation Report prepared by Kanda & Tso Structural Engineers, dated December 20, 2013, for the building identified as:

Building: Classroom Bldg D
Campus: El Rodeo School
Address: 605 Whittier Drive
Beverly Hills, CA 90210

Pursuant to Section 1859.82(a), Title 2, California Code of Regulations, the report has been reviewed by DSA. Based on the report findings, the building identified above meets the eligibility requirements for Seismic Mitigation Program (SMP) funding.

For projects consisting of replacement of eligible buildings, a Replacement Option Analysis shall be prepared and submitted to DSA Headquarters in accordance with Section 2 of DSA Procedure 08-03, which is available on the DSA website (Publications Section).

For projects consisting of seismic rehabilitation of eligible buildings, an Evaluation and Design Criteria Report shall be prepared and submitted to the DSA Regional Office in accordance with Section 3 of DSA Procedure 08-03.

Please contact me at (916) 324-4390 if you have any questions regarding this notice. For questions regarding the Seismic Mitigation Program funding and Office of Public School Construction (OPSC) procedures, please contact Ms. Hannah Konnoff at (916) 375-4037 or Ms. Tasha Brennan at (916) 375-4138.

Sincerely,

William Van Woert

William Van Woert
DSA Senior Structural Engineer

cc: D. Murakawa-Leopard, Beverly Hills Unified School District
Les Tso, Kanda & Tso, Structural Engineers
Hannah Konnoff, Project Manager, Office of Public School Construction
Doug Humphrey, Regional Manager, DSA Los Angeles Regional Office

DSA
DIVISION OF THE STATE ARCHITECT
DEPARTMENT OF GENERAL SERVICES

STATE OF CALIFORNIA
GOVERNOR EDMUND G. BROWN JR.

April 24, 2014

Dr. Gary Woods
District Superintendent
Beverly Hills Unified School District
255 South Lasky Drive
Beverly Hills, CA 90212

EVALUATION OF ELIGIBILITY FOR SEISMIC MITIGATION PROGRAM: DSA CONCURRENCE
Project Tracking Number: 64311-50 (Please use this number for all future correspondence).

Dear Dr. Woods,

The Division of the State Architect (DSA) has received the Eligibility Evaluation Report prepared by Kanda & Tso Structural Engineers, dated December 20, 2013, for the building identified as:

Building:	Classroom, Admin., Auditorium, Kitchen/Dining Bldg A-B-C
Campus:	El Rodeo School
Address:	605 Whittier Drive Beverly Hills, CA 90210

Pursuant to Section 1859.82(a), Title 2, California Code of Regulations, the report has been reviewed by DSA. Based on the report findings, the building identified above meets the eligibility requirements for Seismic Mitigation Program (SMP) funding.

For projects consisting of replacement of eligible buildings, a Replacement Option Analysis shall be prepared and submitted to DSA Headquarters in accordance with Section 2 of DSA Procedure 08-03, which is available on the DSA website (Publications Section).

For projects consisting of seismic rehabilitation of eligible buildings, an Evaluation and Design Criteria Report shall be prepared and submitted to the DSA Regional Office in accordance with Section 3 of DSA Procedure 08-03.

Please contact me at (916) 324-4390 if you have any questions regarding this notice. For questions regarding the Seismic Mitigation Program funding and Office of Public School Construction (OPSC) procedures, please contact Ms. Hannah Konnoff at (916) 375-4037 or Ms. Tasha Brennan at (916) 375-4138.

Sincerely,

William Van Woert

William Van Woert
DSA Senior Structural Engineer

cc: D. Murakawa-Leopard, Beverly Hills Unified School District
Les Tso, Kanda & Tso, Structural Engineers
Hannah Konnoff, Project Manager, Office of Public School Construction
Doug Humphrey, Regional Manager, DSA Los Angeles Regional Office

511 Mission Street
South Pasadena, California
91030-3035

Kanda and Tso Associates consulting structural engineers

telephone
[626] 441 1211

fax
[626] 441 1011

KANDA | TSO

April 27, 2015
Mr. Tim Buresh
Beverly Hills Unified School District
255 South Laskey Drive
Beverly Hills, Ca 90212

Subject: **El Rodeo School Seismic Assessment**
Beverly Hills Unified School District

Dear Mr. Buresh:

In response to your request for an assessment of seismic safety relative to the above cited campus buildings, we submit this letter summarizing our findings based on the work that we have performed to date, including correspondence with other parties.

Kanda and Tso Associates serves as the project's consulting structural engineers, in collaboration with project architect, HMC Architects. Since 1990, most of the firm's work has been devoted to serving the public school sector. The writer has over 30 years experience in the profession and is thoroughly familiar with Field Act and Division of the State Architect (DSA) standards and procedures. Similar seismic mitigation projects under his charge are either underway or has been completed at other school districts.

Over the past four years, the firm has become familiar with the campus and its buildings through a variety of tasks and activities. These include:

- Numerous site visits for the purposes of condition assessment and data collection
- Review of a report entitled "Seismic Risk Evaluation Beverly Hills K-8 Schools", prepared by MHP Structural Engineers, Inc., dated Nov. 11, 2008
- Review of a geohazard report prepared by Leighton Consulting, Inc., dated Mar. 2, 2015
- Study of available record drawings from the 1927 original construction through the most recent 2000 voluntary retrofit
- Attending preliminary design meetings with the District, Architect and DSA
- Preparing eligibility evaluation reports in accordance with DSA Procedure 08-03 as part of Phase 1 of the Seismic Mitigation Program (SMP)
- Preparing an evaluation and design criteria report in accordance with the aforementioned procedure as part of Phase 3 of the SMP
- Preparing a data collection program and reviewing obtained results
- Preparing preliminary seismic analyses
- Preparing preliminary seismic retrofit plans

The present campus consists of five primary buildings, A, B, C, D and E. The following is a brief description:
Building A – Auditorium and Administration, erected 1927
Building B – K-1, erected 1927
Building C – Classrooms and Cafeteria, erected 1927

Building D – Classrooms and Gymnasium with shower/lockers, erected 1962
Building E – Classrooms and Library, erected 1966

Because Buildings A, B and C are physically attached together, without separation joints, they are considered a singular structure and will be referred to as Building ABC herein.

The scope of this assessment concerns structural safety of the building structures, including its cladding as they relate to strong ground motion from seismic events. Other non-structural components, mechanical, electrical and plumbing systems are excluded from this scope.

Based on the work performed and the documentation compiled, conditions that present a high potential for catastrophic collapse have been identified, which is a qualification to be eligible for Proposition 1D retrofit funding under the Seismic Mitigation Program. DSA's concurrence with the Phase 1 reports represents their confirmation of the potential hazards identified.

While the geologists opine in the aforementioned geohazard report that there is no evidence to suggest active faults underlie the campus, they go on to state that "strong ground shaking has and will occur at this site." The ground motions described in the report translates into large horizontal forces - - forces the structures were never designed for. The geologists also cite the existence of blind-thrust faults that occur throughout the Los Angeles basin. These types of faults have been known to also generate high vertical accelerations, which presents additional concerns described below.

Subjected to these seismic conditions, the structures are at risk, as initially identified in the 2008 MHP report. The report assigned a risk category for each building, 1 through 5, with 1 as the highest risk. They rated Building ABC with a Category 1 as the highest risk with significant life safety hazard. Building D was rated a Category 3 as moderate risk with some life safety hazard. Building E was rated a Category 4 as low risk and is unlikely to have life safety hazard.

The following hazards have been identified as the most significant, to be addressed as part of the SMP work, currently in progress:

1. Catastrophic collapse caused by separation of the walls from the floors and roof due to inadequate or lack of wall-to-framing connections:
   Affected structure: Building ABC
   a. Without adequate wall anchors, the supporting walls of the building could pull away from the horizontal framing when subjected to strong ground shaking. Their separating from the framing results in loss of both vertical support for the floors and roof; and structural stability. In this event, both walls and framing would likely collapse.

2. Falling hazards:
   Affected structure: Building ABC
   a. These include building cladding and suspended components. Of particular concern is the vulnerability of the plaster ceiling over the auditorium. It was initially identified in the MHP report as a significant life safety hazard. Strong earthquakes with high vertical accelerations could be detrimental for these types of assemblies. For example, the 1994

Northridge earthquake, which was a blind-thrust earthquake, generated excessively high vertical accelerations. This action, coupled with strong horizontal shaking, may have caused ceilings to collapse in at lease one area high school auditorium. It prompted the School District to retrofit other auditorium ceilings that were at risk district-wide. These high accelerations have a tendency to intensify the hanger loads on ceiling wires; causing them to break loose if they are not properly installed. Based on recent visits, many of the ceiling hangers were found to be non-compliant at El Rodeo School. Refer to the photo appendix for conditions.

b. As part of the data collection program, an investigation was performed by a specialty contractor to determine how cast stone panels, which decorate the exterior façades of Wings A and C, are attached. An investigation of a cast stone window grill did not find metal anchors attaching it to the parent structure. However, during a subsequent visit, a light gage steel channel resembling what could have been part of an anchorage assembly was visible. But, a metal piece which would ordinarily connect the channel to the stone was missing. Perhaps the original design intended for this connector, but was never installed. It was also discovered that the grill itself is a single unit and is attached to separate stone pieces that make up the surrounding frame with four steel wires, embedded in mortar. Photos depicting these conditions are in the appendix.

   On April 25, a follow-up investigation was performed by the contractor, with the writer and Mr. Dan Helphrey of Totum present. The purpose of the second visit was to check another area to see, if possibly, anchors exist elsewhere. The location selected was at the upper deck of the bell tower. A small portion of concrete wall and mortar were removed on the inside face of the tower. The contractor made an opening, large enough to see the interface between the structure and the stone. It revealed the presence of an anchorage system similar to those found in present day veneer assemblies. It consists of a hooked metal tab, embedded into the mortar joint of the stone and was fitted into a light gage metal channel, similar to the one observed at the grill. The anchor was found in two locations, both at the bed joints of the stone pieces. The anchors were heavily rusted and came loose when exposed. See photo appendix for conditions.

   Based on this discovery, it is relatively certain that the architectural cast stone assemblies were post-installed as precast concrete units and are presently attached to the structure using an antiquated veneer anchorage system.

   It is our opinion that while these metal anchors indeed exist, the degree of deterioration observed greatly diminishes their capacity to hold the heavy stone to the building; and must rely only on the bond strength of the mortar. Even if they were in good condition, their capacity to hold the stone to the building would not be enough for the force levels generated by the ground motions cited in the geohazard report. This deficiency presents a significant life safety hazard as the bond between the stone and structural substrate could easily break loose, causing the stone pieces to fall out. The failure of the attachment could have serious ramifications, causing severe injury or death. Their falling off the building could also block building exits and restrict passage to safety.

c. Falling clay roof tiles due to inadequate attachment can cause similar injury or prevent passage to safety as noted above with stone panels.
d. HMC has already prepared drawings to address the deficiencies relative to the auditorium ceiling. Additional drawings will be prepared to mitigate the other falling hazards.

3. Inadequate strength of the concrete walls to resist lateral in-plane and out-of-plane loads: Building ABC and D
   a. The original campus structures were installed in 1927, pre-dating the Field Act and its rigorous requirements for seismic design and continuous construction inspection. Historically, structures erected in that era were designed for gravity loads with minimal consideration for earthquake effects. Seismic resistant technology as we know it today simply did not exist at that time.
   b. Record drawings show that the walls of the 1927 building to be reinforced with only 3/8-inch steel bars in both vertical and horizontal directions, which was standard for concrete walls for that era. Not only are they substandard relative to modern codes, they are not adequate to resist the required seismic forces for both in-plane and out-of plane actions.

4. Inadequate strength of floor and roof diaphragms, chords and collectors: Building ABC
   a. These important components of the lateral force resisting system serve to transmit lateral load to the vertical resisting elements of a structure, i.e. the shear walls.
   b. Preliminary analyses indicate that these components do not have the required strength to perform this action, due to the massive seismic weight of the structure, coupled with extreme ground motions.

As the SMP work continues on, we are in support of the District's plan to relocate the students from the existing buildings. This would provide for a single phased construction activity with shorter duration over a cumbersome, multi-phased arrangement; and would protect the occupants from possible harm if a major earthquake were to occur prior to the completion of the retrofit. Because of the potential hazards identified, we recommend the relocation of the occupants to occur as soon as possible, especially from Risk Category 1 buildings.

If you have any questions, please contact the writer.

Very truly yours,

KANDA and TSO ASSOCIATES

Leslie Tso, S.E., President/CEO

PHOTO APPENDIX

Photo 1
Non-compliant ceiling wire

Photo 2
Ceiling wire with inadequate number of turns (DSA requires 3 tight turns)

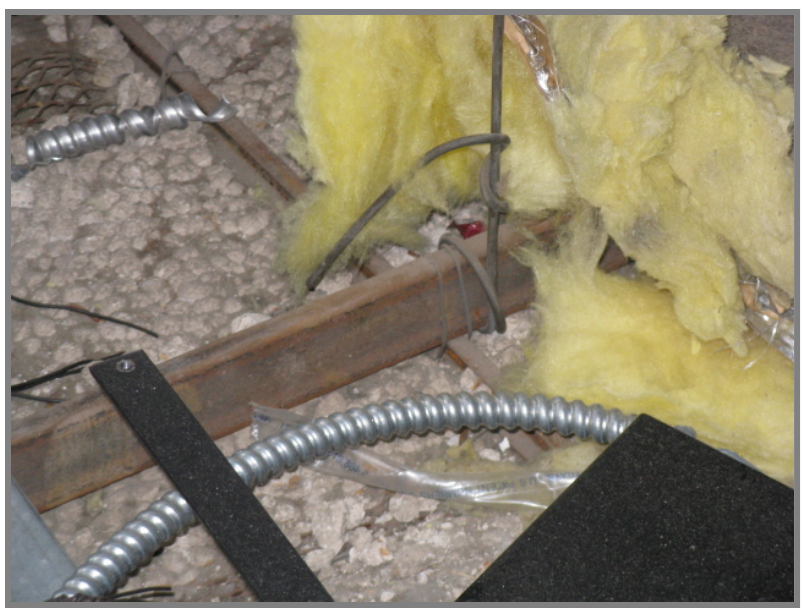


Photo 3
Excess weight on ceiling caused by construction waste and debris

Photo 4
Initial investigation of cast stone grill

Photo 5
Evidence of steel channel embed indicating possible anchor assembly (attached only to the concrete wall)

Photo 6
Cast stone grill with wire tie embedded in mortar

Photo 7
Metal anchor found in mortar joint of cast stone, separated from metal channel embed

Photo 8
Hooked metal anchor found in mortar joint of cast stone

Date: June 11, 2014

To: Tim Buresch and Eldon Gath

From: Miles Kenney
Kenney GeoScience

Subject: **Preliminary assessment of seismic ground shaking response associated with local geologic factors for school sites in the City of Beverly Hills**

Preliminary findings of an assessment of seismic ground shaking response associated with local geologic factors for school sites in the City of Beverly Hills are provided. The five schools include, El Rodeo, Beverly Hills, Hawthorne, Beverly Vista and Horace Mann. Chris Madden (Earth Consultants International) and Miles Kenney PhD, PG (Kenney Geoscience) performed this evaluation. Discussions with Tim Buresh also led to evaluating some seismic parameters. This short study was motivated by a question from Tim Buresh regarding why Beverly Vista school experience much more damage due to ground shaking compared to Horace Mann school located only one kilometer to the east.

Potential seismic parameters evaluated include:

- Instrumental shaking amplification maps from the United State Geological Survey (USGS)
- Location of red-flagged structures from the 1994 Northridge Earthquake
- Local near surface (surficial) geologic units
- Location of local faults
- Regions of historical subsidence
- Region of artesian wells and near surface groundwater from 1905 data

Findings

Ground shaking intensity map associated with the 1994 Northridge Earthquake

A ground shaking intensity map provided by the USGS (website) for the 1994 Northridge earthquake indicates that the City of Beverly Hills was located in a region designated as very strong to lower severe (Plate 1). However, this region was not located in a region exhibiting the strongest ground motions during this earthquake, which are approximately outlined in Plate 1 by black dashed lines. Red-flagged structures identified after the earthquake are also shown on Plate 1.

Red-flagged buildings associated with the 1994 Northridge Earthquake

The Beverly Vista and Horace Mann schools are located within a zone of red-flagged structures identified after the 1994 Northridge Earthquake (Plate 2). The Beverly Vista school resides along the western edge of the red-flagged structures area and the Horace Mann school resides near the center. The El Rodeo, Hawthorne and Beverly Hills High schools do not reside in the zone of red-flagged buildings (Plate 1).

Surficial Geology

The Beverly Hills High School primarily resides on well indurated mid to late Pleistocene sediments that are over 200,000 years old. Older structures of the El Rodeo school reside on sediments over 200,000 years old; however, younger structures reside on Holocene to late Pleistocene sediments (Qay2 on Plate 2). The Hawthorne, Beverly Vista and Horace Mann schools reside on Holocene age sediments that are considered unconsolidated (Plate 2). The Horace Mann school is located on the generalized contact between areas that have flooded historically (Qay2 of the USGS OFR 97-256; east side of school) and relatively older undifferentiated Holocene alluvial sediments to the west (unit Qay1).

Fault locations

The Beverly Hills High and El Rodeo schools overly faults associated with the Santa Monica Fault Zone in Century City (Parsons, 2011; KGS, 2012, 2013; 2014 in preparation; LCI, 2012a, 2012b). Interpreted faults associated with the Newport-Inglewood fault zone likely occur immediately east of Beverly Hills High school along Moreno Drive (KGS, 2013). The Hawthorne school is not underlain by any published faults; however, it may reside near proposed "cross-faults" in the western Hollywood Basin (Plate 2; KGS, 2014 in preparation). The Beverly Vista and Horace Mann schools are located near the surface projection of the east-west trending blind San Vicente Fault Zone. It is unknown if this fault zone is active or inactive, but may be active based on an evaluation of recent earthquakes by Madden (ECI, 2014, in preparation). These two schools are also possibly located within the Newport-Inglewood fault zone if the fault extends this far north. Erickson and Spaulding (1975) and Wright (1991) proposed that strands of the Newport-Inglewood fault zone occur in this region as shown on Plate 3. If true, then Beverly Vista school is located between two strands and the Horace Mann school is located east of the fault zone.

There is a weak correlation associated with the location of red-flagged buildings overlying the San Vicente fault zone. A relatively dense concentration or red-flagged buildings trending approximately east-west occurs over the east-west trending San Vicente fault zone in the region of the Beverly Vista and Horace Mann schools. If increased ground shaking in this reason is associated with the fault, it suggests that seismic waves were "channeled" up the fault zone or by stratigraphic structure toward the surface. These processes have been proposed along the western Santa Monica Fault west of Highway 405 (City of Santa Monica; see Graves et al., 1998 and Goa et al. 1996). It should be pointed out that the San Vicente fault zone extends to depths of at least a 1000 meters below the surface (Plate 2).

Regions of historical subsidence

Regions of subsidence associated with historical fluid withdrawal often occur in sub-basins, hence possibly delineated basins that can trap seismic surface waves increasing ground shaking intensity and duration both of which can increase structural damage. Numerous subsidence studies have been conducted in the region and most show subsidence delineating the Hollywood Basin located south of the Santa Monica Mountains, north of the North Salt Lake Fault, and east of the Cheviot Hills. Subsidence data from the Castle and Yerkes (1976) is shown on Plate 2 showing subsidence in the central and eastern Hollywood Basin located northeast and east of the City of Beverly Hills.

The five school sites, based on the Castle and Yerkes (1975) data suggests that they may reside in a sub-basin with the Hawthorne, El Rodeo and Beverly Hills school residing along the edge of the basin, and the Beverly Vista and Horace Mann residing in the middle.

Region of artesian wells and near surface groundwater from 1905 data

Near surface groundwater can increase seismic wave magnitude and duration, both of which can increase damage associated with ground shaking. Trifunac and Todorovska (1997) indicate that some regions that experience increase ground shaking (well above what would have been estimated) due to near surface groundwater during the 1994 Northridge Earthquake. It was beyond the scope of this study to investigate the depth of groundwater at the various school sites, but an initial first order evaluation can be conducted by evaluating a map provided by Mendenhal in 1905. He published a seminal groundwater paper of the northern Los Angeles Basin detailing regions of pumping plants (saturated conditions at the surface), region of artesian wells, and regions where pumping was identified near the surface. The local extent of these regions are identified by Mendenhal (1905) are shown on Plates 2 and 3. Hence, it is in these areas that groundwater, at least historically, was very close to the surface. Based on evaluation of numerous water well monitoring sites in the region indicates that groundwater in the region is often within 100 feet of the surface.

There is a weak correlation between the location of red-flagged structures and the region of shallow groundwater as mapped by Mendenhal (1905; Plate 2). Most of the red-flagged structures do occur in or near mapped areas exhibiting historical shallow groundwater. If this correlation is correct, then it suggest that shallow groundwater may contribute to increased ground shaking related damage at the Beverly Vista and Horace Mann school sites.

Perched groundwater, meaning groundwater at shallower depths than the “water table” can also increase ground shaking damage during a major earthquake. Hence, increased landscape watering can increase ground shaking locally at a site. It may be productive to evaluate groundwater and soil conditions at each school site in terms of seismic shaking hazards. Some water monitoring wells occur on the El Rodeo school site, and possibly Beverly Hills High school. It is unknown if the other school sites have similar data.

Discussion

Most of the seismic shaking data evaluated in this study (intensity maps and location of red-flagged buildings) are associated with the 1994 Northridge Earthquake. The epicenter of this earthquake was nearly 20 kilometers north of the City of Beverly Hills. The magnitude of ground shaking south of the Santa Monica Mountains including the City of Beverly Hills was much larger than what would have been predicted with empirical ground shaking models prior to the quake. It was a strong lesson that local geology (topography, groundwater, fault locations, alluvial basins) can greatly modify anticipated ground shaking. It is likely that because the 1994 Northridge Earthquake was nearly 20 kilometers away, that heavy damage south of the Santa Monica Mountains was likely localized in certain areas with “favorable” geologic conditions. One function of the location of heavily damaged areas was the direction of the traveling seismic waves. In other words, if another regional earthquake were to occur in a different direction, then these seismic wave may behave much differently in the northern Los Angeles Basin than the 1994 Northridge Earthquake. A good example of this is if the San Andreas Fault or San Jacinto Fault were to rupture, both of which are considered “over due”.

If a local major earthquake were to occur, then it is likely that the density of red-flagged buildings would likely be more concentrated in alluvial basins, however, there would also be a considerably higher concentration across the region than what was observed during the 1994 Northridge earthquake because the seismic source is essentially “underfoot” and does not require local structures to assist in amplifying waves and duration of shaking.

It should also be pointed out that roughly half of the major earthquakes in southern California historically have occurred on structures (faults) not previously fully recognized. Examples include the 1987 Whittier “narrows” Earthquake, the 1994 Northridge Earthquake, and the 1992 Landers Earthquake.

Hence, it may be prudent, particularly for school sites, to prepare for “the worst” case scenario. For example, to prepare for the possibility of a major local earthquake occurring on an unknown fault.

These issues have been raised because all five school sites in the City of Beverly Hills will at some point in the future likely experience severe ground shaking well above that experienced during the 1994 Northridge Earthquake. In addition, that it is essentially impossible at this time to fully evaluate the magnitude of shaking these sites will experience in the future during regional and local major quakes with great certainty.

Depending on the local geologic conditions of each site, ground shaking will likely vary among the various school sites. Based on the evaluation of this study, it seems likely that the Beverly Vista and Horace Mann school sites likely exhibit the relatively largest hazard due to their location in a region of potentially shallow groundwater, within an alluvial basin, and occurring above the structural San Vicente Fault. The Hawthorne school site located on alluvium associated with the Hollywood Basin and likely in a region of numerous faults (Hollywood, Santa Monica, possible cross-faults) is a close second to the Beverly Vista and Horace Mann school sites. Beverly Hills High and El Rodeo schools are located on relatively dense sediments, which will decrease the relative ground shaking compared to other school sites. This is supported by the paucity of red-flagged structures near these schools associated with the 1994 Northridge Earthquake.

Comparing the Beverly Vista and Horace Mann school sites

This short study was initially motivated by the question regarding potential geologic reasons that the Beverly Vista school experience considerably more damage than the Horace Mann school located only 1 kilometer to the east. This is a very difficult question to answer and with the available data can only be speculated. The data provided in this report suggest that these two sites exhibit similar key geologic/seismic criteria: These include:

- Both occur within a zone of relatively dense red-flagged buildings from the 1994 Northridge Earthquake.
- Both occur within an area of basin alluvium (Qay1 and Qay2 on Plate 2).
- Both occur above the San Vicente Fault, which in association with potential deeper alluvial sediments south of the fault could produce increased ground shaking from regional earthquakes.

The two school sites also exhibit some geologic conditions that could be considered contrary to the level of damage the two schools exhibited during the 1994 Northridge Earthquake. These include:

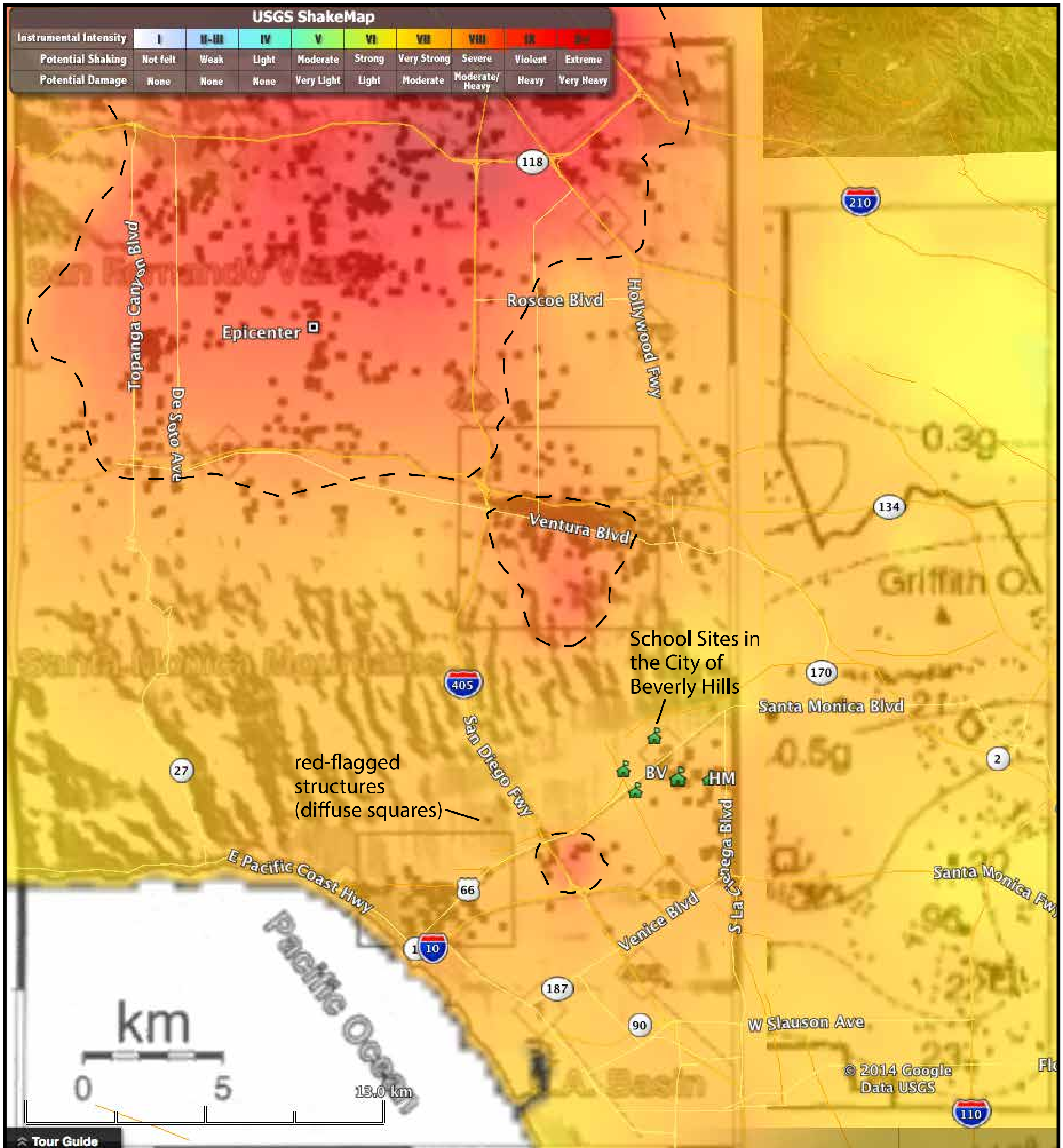
- The Horace Mann school site is located within the region of mapped shallow groundwater shown by Mendenhal (1905) whereas the Beverly Vista school is not. Additional groundwater studies may be warranted.
- The Beverly Vista school is located at the edge of the relatively dense red-flagged buildings of the 1994 Northridge Earthquake whereas the Horace Mann school is located essentially in the center of this zone.

One potential geologic difference between the Beverly Vista and Horace Mann school sites is that the former may be located within the Newport-Inglewood fault zone whereas the Horace Mann school site may exist east of the fault zone (Plates 2 and 3). The extension of the Newport-Inglewood fault this far north is speculative; however, if it does, it is possible that increased shaking could occur between various strands of the Newport-Inglewood fault zone associated with trapped waves, or maybe variations in bounded groundwater.

Other explanations for variation in site damage could include:

- Variations in design (engineering design and/or quality of seismic retrofits).
- Possible increase landscape watering at Beverly Vista school compared to Horace Mann school or some other geologic parameter that would increase the Beverly Vista school seismic site response to seismic waves. In other words, some local geologic factor may have increased the seismic wave amplitudes and/or duration at Beverly Vista compared to Horace Mann that is currently unknown.
- Although difficult to predict, it is possible that multiple surface waves “trains” may have collided near the Beverly Vista school to produce relative high magnitudes of shaking and this process did not occur at the Horace Mann school site.

MDK



Groundshaking intensity map from the USGS (website) associated with the 1994 Northridge Earthquake. Groundshaking USGS data overlies red-flagged structure data (■) of Trifunac and Todorovska (1997).

Kenney GeoScience

CLIENT:
PRIMESOURCE PM LLC

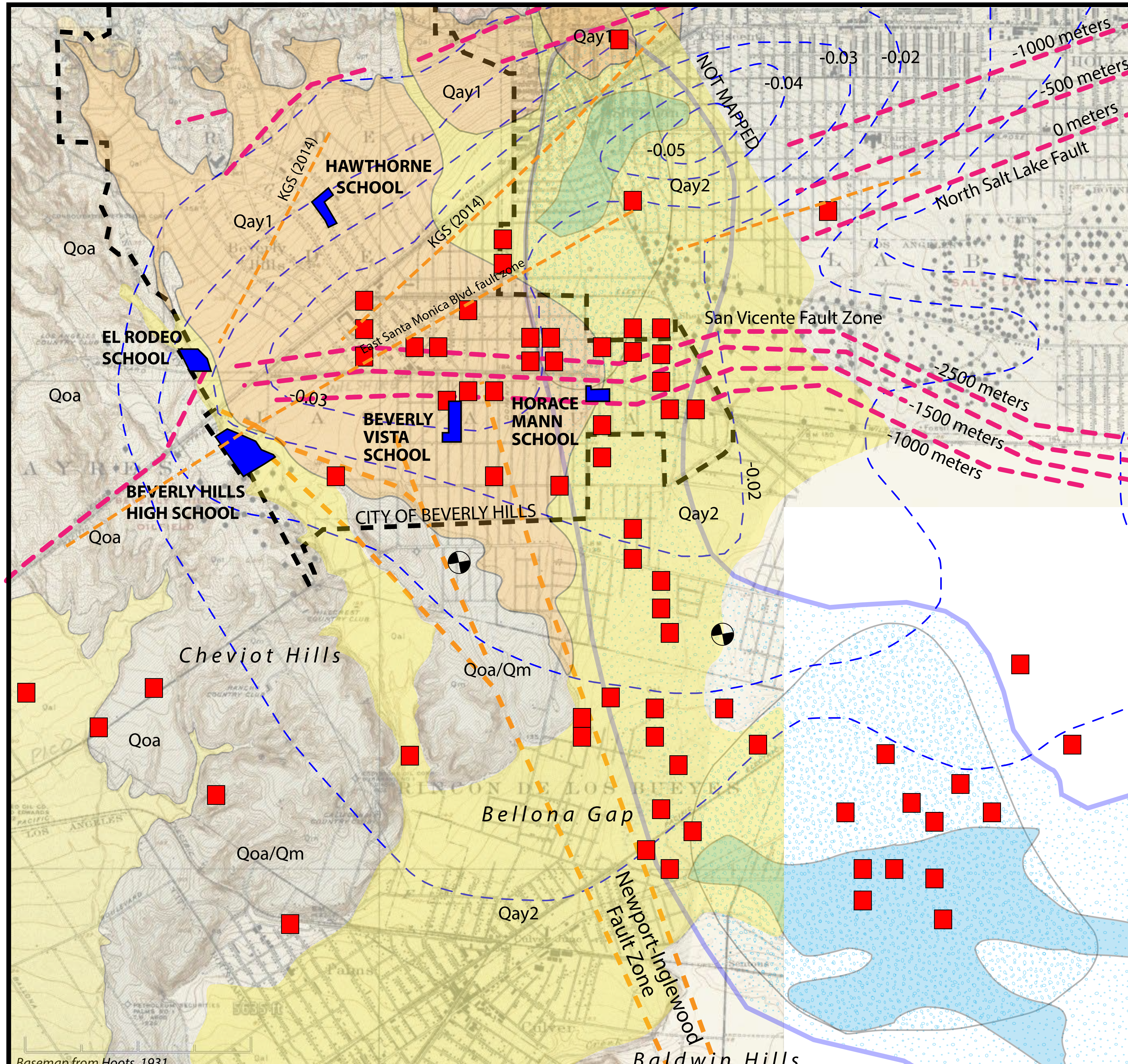
REPORT:
PRELIMINARY EVALUATION OF POTENTIAL SEISMIC HAZARDS AT CITY OF BEVERLY HILLS SCHOOL SITES

JN 723-11

Date: JUNE, 2014
Drafted by: MDK

PLATE 1

Map created by Chris Madden (ECI)



SYMBOL DESCRIPTIONS

Symbol	Description
BEVERLY VISTA SCHOOL	Approximate location of school within the City of Beverly Hills
Qay2	Approximate limits of unconsolidated and uncementd alluvium (Holocene) that are flooded historically. USGS OFR 97-256.
Qay1	Approximate limits of unconsolidated and uncemented alluvium (Holocne) - undifferentiated. USGS OFR 97-256
	Very approximate location of red-flag structure associaited with the 1994 Northridge Earthquake. Data from Trifunac and Todoravska (1997)
-2500 meters	Approximate location of fault by Tsutsumi et al. (2001). Structure contour depths in meters.
	Approximate location of proposed faults by KGS (2014)
-0.05	Approximate location of subsidence conours from Castle and Yerkes (1976). Contour intervals in feet.
	Approximate limits of region or original artesian wells from Mendenhal (1905). See Plate 1 for symbol descriptions.
	Approximate location of strike-slip focal mechanism. See Plate 1 for magnitude, year of quake and source of data.

N

~5000 feet

Scale

Kenney GeoScience

CLIENT:
PRIMESOURCE PM LLC

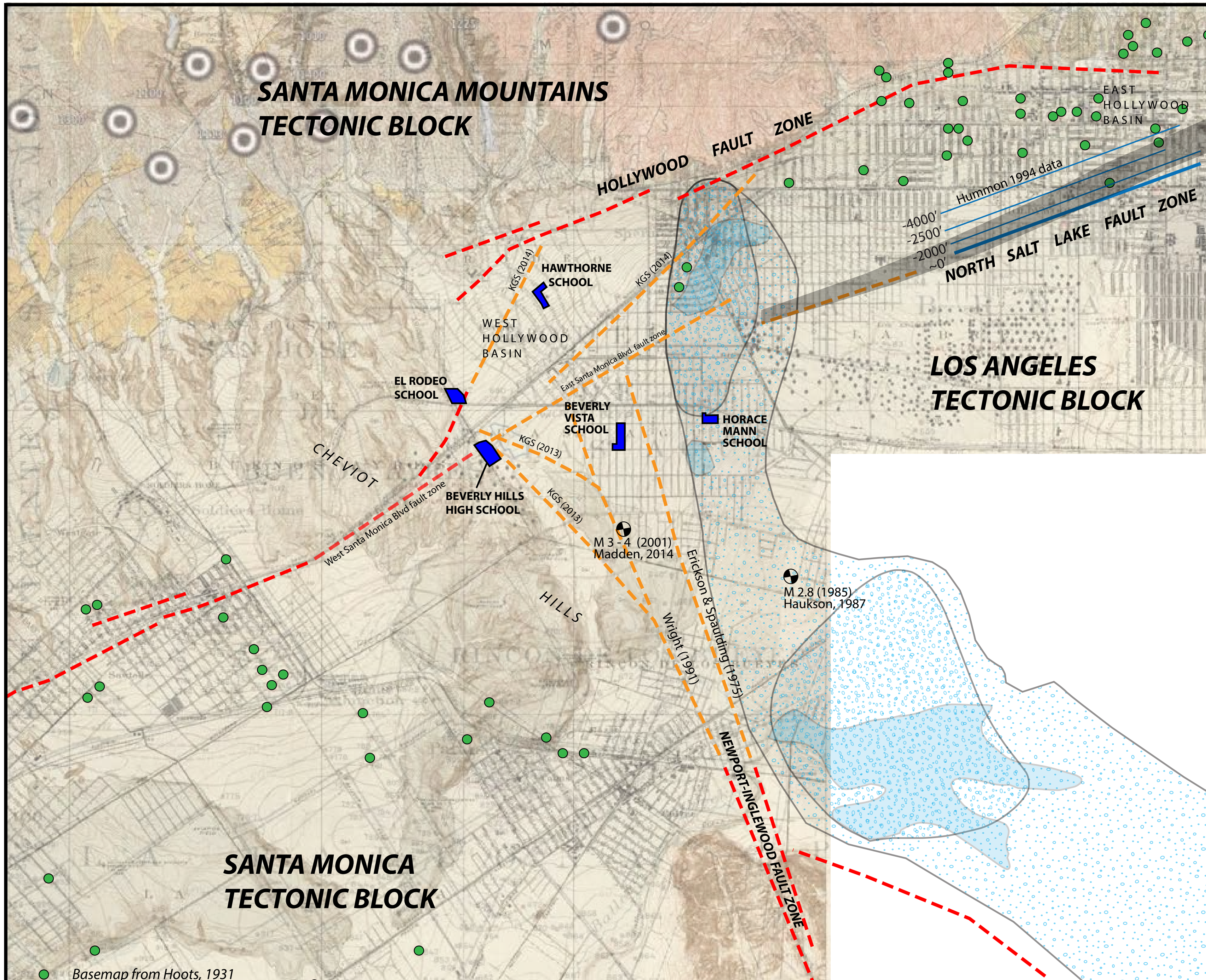
REPORT:
PRELIMINARY EVALUATION OF POTENTIAL SEISMIC HAZARDS AT CITY OF BEVERLY HILLS SCHOOL SITES

JN 723-11

Date: JUNE, 2014

Drafted by: MDK

PLATE 2



SYMBOL DESCRIPTIONS

Approximate location of published fault in the near surface

Postulated fault (this study) in the near surface

Approximate epicenter location of historical earthquake focal mechanism. Magnitue, year of event and data source provided. Focal mechanisms show strike-slip faulting.

Approximate region of historical subsidence proposed to be associated with the North Salt Lake Fault (Hill et al., 1979).

-2000'
0'
Approximate location of structure contours for the North Salt Lake Fault from Hummon (1994). Depth in feet below the surface.

Mendenhal, 1905 Groundwater data (WSIP Report No. 139)

Region of pumping near surface groundwater (saturated)

Region of artesian wells in 1905

Approximate region of original artesian wells.

Location of pumping plants

N

5000 feet
scale

Kenney GeoScience

CLIENT:
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PLATE 3

7.0 Conclusions and Recommendations

Nearly every building included within this seismic study was constructed prior to the introduction of modern seismic design procedures in building codes, which were first added to the Field Act in 1978. This building vulnerability, combined with the significant seismic hazards affecting the sites (primarily strong ground motion), result in the deficiencies identified.

The seismic studies of the subject buildings have identified numerous seismic deficiencies that represent life-safety hazards, as defined by ASCE 31-03 criteria. However, some of the buildings have limited deficiencies in only certain elements, and will require less seismic strengthening than others. The overall significance of the risk was estimated and captured by placing each building into a relative risk category (using the 1-5 scale described in Section 4.0 and repeated in Section 7.2 below) based on the identified hazards and deficiencies in the building construction. The building risk category is indicated in the Recommendations Summary in Section 7.2.

The seismic deficiencies identified by the evaluation are described below for each building. The rough, order-of-magnitude cost to seismically strengthen the buildings has been estimated in the summary table. The strengthening cost is dependent on many factors. These factors include:

- Desired Performance Level: Upgrading the seismic force-resisting systems of existing buildings to current code criteria is typically cost prohibitive. Most often, a desired performance level, such as Life-Safety, or Immediate Occupancy, is chosen. While reducing the risk to life in buildings is of utmost concern, some level of damage control is often desirable, particularly in facilities that need to be operational after a large earthquake. The incremental cost to provide some level of additional damage control can vary, but is sometimes relatively small. Any seismic strengthening should, as a minimum, meet *Life Safety* performance level per the requirements of ACSE 41-06 "Seismic Rehabilitation of Existing Buildings" or requirements of Chapter 34 "Existing Buildings", 2007 California Building Code.
- Construction Phasing: If the seismic strengthening is completed in conjunction with other building upgrades (such as architectural, mechanical/electrical, ADA, communications, etc.), the cost will be much lower than if the seismic strengthening is completed as an independent phase.
- Architectural Impacts: Often times, various strengthening options are available. Some options may have architectural impacts, but will be less expensive to implement. A reasonable balance between historic significance/architectural appearance, and cost to implement should be adopted.

The accuracy of cost estimates for seismic strengthening work is also directly related to the depth of the analysis and to development of detailed strengthening plans for a cost estimator or contractor to evaluate. More accurate cost estimates can be achieved by developing a complete set of preliminary strengthening plans. The strengthening costs provided herein address only the structural construction costs alone, and do not include modernization costs or other soft costs.

7.1 Conclusions and Recommendations

Based on the deficiencies identified in our evaluation, the following strengthening measures are recommended:

7.1.1 El Rodeo School

Building A, B, & C

Based upon the deficiencies identified in Section 6.0, strengthening of the building should consist of the following:

Wing A

- Remove and replace the heavy plaster ceiling above the seating area of the auditorium (and any other areas) with a lighter ceiling system. Alternatively, it may be feasible to isolate and brace the plaster ceiling to resist seismic forces.
- Add steel chords at the concrete roof diaphragm above the auditorium along the east and west sides.

Wing B

- Add positive wall anchorage at the roof level in limited locations.
- Verify the adequacy of the steel strapping and connections that tie the wood canopy to the building (south side of the building).

Wing C

- Add additional positive wall anchorage at the roof and elevated floor levels in limited locations. Strengthen the expansion anchors at existing steel tube struts if required.

All Wings

- Strengthen the diagonally sheathed wood roof and floor diaphragms where overstress conditions occur.
- Add additional lateral force resisting elements such as concrete or CMU shear walls, braced frames, and/or strengthen existing concrete elements by fiber-wrapping or infilling in select, limited locations (along with associated foundation work).
- Remove and replace the heavy concrete façade elements (Art Stone) with lighter elements, or re-install existing elements with proper anchorage to the concrete substrate.
- Other minor miscellaneous seismic strengthening upgrades (such as equipment anchorage, emergency gas shut-off valves, removal of URM and hollow clay tile walls (if occurs), etc.)

Building D

Based upon the deficiencies identified in Section 6.0, strengthening of the building should consist of the following:

- Additional transverse plywood sheathed wood stud shear walls at the first and second floors and additional corridor plywood sheathed wood stud shear walls at the second floor should be added to reduce the stresses in the diaphragm and overstressed shear walls.
- Drag elements including wood beams or blocking and steel straps should be added to adequately drag the lateral forces from the roof and second floor diaphragms into the shear walls.
- New out-of-plane anchorage of heavy concrete walls at second floor level should be provided. Anchorage should be spaced at a maximum of 4 feet on center. Total effected length of wall is approximately 100 linear feet.

Building E

Based upon the analysis noted in Section 6.0, strengthening of the building does not appear to be necessary.

7.1.2 Hawthorne School

Building A

Based upon the deficiencies identified in Section 6.0, strengthening of Building A should consist of the following:

- Where straight-sheathed diaphragms occur at the roof and floor levels, new plywood sheathing should be added over the existing straight sheathing.
- Positive anchorage between the heavy timber roof trusses and the central concrete corridors should be provided.
- The existence of fiber-wrapping at the corridor side of the short, captive concrete column piers should be verified, and new fiber wrap should be added where it does not occur.
- Evaluate and strengthen concrete "piers" or "posts" that extend from the continuous grade beams or stem walls into the soil. These elements may be vulnerable to shear and/or flexural failure.
- Additional concrete or steel wall anchorage ties should be provided at roof and floor levels where required. Additionally, the shear transfer mechanism between the roof and floor diaphragms and perimeter concrete walls should be improved.
- Remove and replace the heavy plaster ceiling above the seating area of the auditorium (and any other areas) with a lighter ceiling system. Alternatively, it may be feasible to isolate and brace the plaster ceiling to resist seismic forces.

Buildings B and C

Based upon the deficiencies identified in Section 6.0, strengthening of Buildings B and C should consist of the following:

- At Building B additional lateral force-resisting elements or strengthening of the existing lateral force-resisting elements will likely be required in the longitudinal direction of the building. This may include fiber-wrapping of the column piers at the central corridor, or other means of strengthening to preclude brittle shear failure when subjected to lateral earthquake loading.

- A Building C, new plywood sheathing should be added over the existing diagonal sheathing. In addition, a new wall anchorage system should be installed around the perimeter of the roof (and possibly floor) structure.
- Buildings B and C may require additional strengthening elements, which could only be determined from a review of structural drawings, or after a destructive testing program to determine the details of the existing construction.

Building D

Based upon the deficiencies identified in Section 6.0, strengthening of Building D should consist of the following:

- The connections of the double channel "X" braces to the steel gusset plates and of the gusset plates to the steel beams and columns should be strengthened by exposing the connections (removing finishes) and field welding the braces and gusset plates to add significant additional capacity.
- The steel columns at the braced frames should be strengthened by welding on steel angles, channels, or plates to increase the column capacities.
- At the Northeast side of the building at the discontinuity where the braced frames step back and are supported by the tapered steel girders at the second floor level, the tapered steel girders should be strengthened and/or the shear transfer mechanism into the floor diaphragm should be strengthened.

Building E

Based upon the deficiencies identified in Section 6.0, strengthening of Building E should consist of the following:

- The continuity or development of the short anchored joists should be upgraded by adding wood blocking in line with the tied joists and strapping across to form continuous continuity ties across the building.
- The tall concrete exhaust stack should be strengthened, removed, braced, or reduced in height. Further investigation would be necessary to determine the exact level of deficiency. This would require destructive and/or non-destructive testing if drawings can not be located.

Building F

Based upon the deficiencies identified in Section 6.0, strengthening of Building F should consist of the following:

- Additional lateral force-resisting elements such as steel moment or braced frames, or new plywood shear walls should be added. In addition, it would be beneficial to add new plywood sheathing over the existing diagonally sheathed roof diaphragm.

Building G

Based upon the deficiencies identified in Section 6.0, strengthening of Building G should consist of the following:

- Additional lateral force-resisting elements, likely consisting of new plywood shear walls should be added at each face of the building. In addition, it would be beneficial to add new plywood sheathing over the existing diagonally sheathed roof and floor diaphragms.

Building H

Based upon the deficiencies identified in Section 6.0, strengthening of Building H should consist of the following:

- Limited upgrades to chords and/or drags at the roof diaphragm and to select wood stud shear walls.

Building J

Based upon the deficiencies identified in Section 6.0, strengthening of Building J should consist of the following:

- The roof diaphragm will require overlaying with plywood (if straight sheathing exists), and lateral force-resisting elements will need to be added to at least two building elevations (either frames, or new plywood shear walls that in-fill some of the perimeter openings).

Building K

Based upon the deficiencies identified in Section 6.0, strengthening of Building K may be required and would consist of the following:

- Limited upgrades to chords, drags, and/or diaphragm nailing; and limited upgrades to perimeter plywood sheathed shear walls. In the event the building lateral force-resisting system utilizes steel frames, more extensive seismic strengthening may be required.

7.1.3 Horace Mann School

Building A

Based upon the deficiencies identified in Section 5.0, strengthening of Building A should consist of the following:

- Strengthen the diagonally sheathed wood floor diaphragms where overstress conditions occur by adding new plywood sheathing over the existing sheathing. Add shear transfer elements to allow the diaphragm shears to be transferred into the concrete shear walls.
- Add wall anchors at the floor levels between the steel trusses and the concrete walls in both the parallel and perpendicular framing conditions.

- Add additional lateral force resisting elements such as concrete or CMU shear walls and/or strengthen existing concrete elements by fiber-wrapping or infilling in select, limited locations (along with associated foundation work).
- Hollow clay tile partition walls (if occurs) should be removed or encapsulated to prevent collapse.

Building B

Based upon the deficiencies identified in Section 5.0, strengthening of Building B should consist of the following:

- Add additional lateral force resisting elements such as concrete or CMU shear walls and/or strengthen existing shear walls by fiber-wrapping or adding shotcrete. Significant associated foundation work may be required, as the building is founded on a deep pile foundation system with concrete grade beams between piles. Access to the building will be difficult, due to the proximity of adjacent buildings.
- Strengthen the connections between the pre-cast balcony rail panels and the edge of the concrete roof and floor slabs by adding additional steel brackets, or remove all of the pre-cast panels and replace with a lighter railing system.

Building C

Based upon the deficiencies identified in Section 5.0, strengthening of Building C should consist of the following:

- Add limited concrete wall infill or strengthen existing concrete elements by fiber-wrapping in select, limited locations including the north wall of the building at the basement level.
- Remove and replace the heavy plaster ceiling above the seating area of the auditorium and above the foyer (and any other areas) with a lighter ceiling system. Alternatively, it may be feasible to isolate and brace the plaster ceiling to resist seismic forces.
- Verify the adequacy of the wood roof diaphragm and wall anchorage, and complete additional strengthening if required. Review of strengthening drawings (currently not available) or limited destructive investigation would be required to complete this evaluation.

Building D

Based upon the deficiencies identified in Section 5.0, strengthening of Building D should consist of the following:

- The bolted connections from the steel X-braced frames to the second floor steel beams should be strengthened to add significant additional capacity.
- The shear transfer mechanism at the braced frames in the transverse direction should be strengthened by adding new steel elements parallel to the direction of lateral loads. Currently, the load is perpendicular to the supporting steel beams, with no designated element parallel to the load.

- The discontinued braced frames supported by a series of steel beams should be strengthened by increasing the stiffness of the system, and/or additional frame elements should be added to increase the stiffness and redundancy of the upper (second) floor structure.
- The insulating concrete filled roof diaphragm should be locally strengthened where required; alternately, if new frame elements are added, the diaphragm stresses could be reduced to acceptable levels.

7.1.4 Beverly Vista School

Building B, C, D, & E

Based upon the analysis noted in Section 6.0, strengthening of the buildings does not appear to be necessary. Other minor miscellaneous seismic strengthening upgrades (such as equipment anchorage, emergency gas shut-off valves, etc.) should be addressed.

7.2 Recommendations Summary and Estimated Costs

The Recommendations Summary table summarizes the recommended seismic strengthening measures for each building, and indicates the relative risk category (using the 1-5 scale below) based on the identified hazards and deficiencies in the building construction. As previously noted, the Risk categories are defined as follows:

RISK CATEGORY	DESCRIPTION OF RISK	LEVEL OF RISK
1	**Building Appears to have a Significant Life-Safety Hazard**	**Highest**
2	**Building Likely has a Life-Safety Hazard**	**High**
3	**Building Possibly has a Life-Safety Hazard**	**Moderate**
4	**Building is Unlikely to have a Life-Safety Hazard**	**Low**
5	**Building is Very Unlikely to have a Life-Safety Hazard**	**Lowest**

The Recommendations Summary table also provides rough, order-of-magnitude cost estimates to reduce the life-safety hazards identified by our evaluation. The cost estimates are based on the following qualifications:

- The seismic strengthening will be completed in conjunction with architectural and other modernization of the buildings.
- The modernization and strengthening work will not exceed 50% of the replacement cost of the buildings, and will therefore not trigger seismic strengthening in full conformance with current code requirements.
- The seismic strengthening will be designed to a Life-Safety performance level.
- The cost estimates are for the structural construction costs alone, and do not include modernization costs, design or permit fees, or construction administration costs. No contingency costs have been included.

It should be noted that the non-structural costs associated with strengthening Horrace Mann Building B are particularly significant, due to very limited access, the nature of the strengthening, and building configuration. These issues are discussed in Appendix A.

RECOMMENDATIONS SUMMARY TABLE

School	BHUSD Bldg Designation	Risk Category	Building Area (square ft)	Recommended Seismic Upgrades	Estimated Cost Per Square Ft	Estimated Construction Cost (1)
El Rodeo	A, B, & C	A = 1 B = 2 C = 2	69,000	Add wall anchorage. Additional shear walls or frames. Bracing heavy "art stone" façade elements. Remove & replace of plaster ceiling at Auditorium Add plywood at wood roof and floor levels.	$49.28	$3,400,000
El Rodeo	D	3	24,500	Add wood stud shear walls. Add wall anchorage at second floor. Add drag elements at the roof and second floors.	$36.73	$900,000
El Rodeo	E	4	23,600	None	$0	$0
Subtotal El Rodeo School		**--**	**117,100**	--	--	**$4,300,000**
Hawthorne	A	1	41,500	Add new plywood sheathing over existing straight sheathing at roof and floors. Add and improve wall anchorage. Verify fiber-wrapping and add new as needed. Remove and replace plaster ceiling at Auditorium.	$50.60	$2,100,000
Hawthorne	B	1	8,700	Add or strengthen lateral force-resisting elements. No drawings available, further review required.	$34.48	$300,000
Hawthorne	C	1	5,000	Add plywood sheathing, add wall anchorage. Possibly add lateral force-resisting elements. No drawings available, further review required.	$35.00	$175,000
Hawthorne	D	1	25,200	Strengthen existing X brace frame welded connections, strengthen columns and beams. Possibly strengthen girders under discontinuous frames, & shear transfer.	$39.68	$1,000,000
Hawthorne	E	2	3,500	Add blocking and strapping to short anchored joists. Strengthen, remove, or brace stack.	$25.00	$87,500
Hawthorne	F	2	8,400	Add new plywood shear walls or frames, add plywood sheathing.	$50.00	$420,000
Hawthorne	G	3	2,400	Add new plywood shear walls and sheathing.	$41.67	$100,000
Hawthorne	H	3	2,700	Upgrade chords, drags, & possibly shear walls.	$22.22	$60,000
Hawthorne	J	2	1,200	Overlay roof diaphragm and add either frames or new plywood shear walls.	$50.00	$60,000
Hawthorne	K	3	12,100	Upgrade chords, drags, perimeter shear walls, and/or diaphragm nailing. No drawings available, more extensive seismic strengthening may be required.	$29.75	$360,000
Subtotal Hawthorne School		**--**	**110,700**	--	--	**$4,662,500**

RECOMMENDATIONS SUMMARY TABLE

School	BHUSD Bldg Designation	Risk Category	Building Area (square ft)	Recommended Seismic Upgrades	Estimated Cost Per Square Ft	Estimated Construction Cost [1]
Horace Mann	A	**1**	45,300	Add new plywood sheathing at floors. Add shear transfer elements. Add wall anchors. Add concrete or CMU shear walls and/or strengthen existing by fiber-wrapping or infilling. Remove or encapsulate hollow clay tile partition walls as needed.	$47.50	$2,150,000
Horace Mann	B	**1**	21,500	Add concrete or CMU shear walls and/or strengthen by fiber-wrapping or adding shotcrete. Significant foundation work appears to be required. Add steel brackets, or remove and replace pre-cast panels at all floor levels. Site access is very restricted due to proximity of adjacent buildings.	$97.67	$2,100,000 [2]
Horace Mann	C	**1**	10,000	Add wall infill or strengthen by fiber-wrapping. Remove and replace suspended plaster ceiling. Verify wood roof diaphragm and anchorage, add limited strengthening if required after further detailed review.	$40.00	$400,000
Horace Mann	D	**2**	31,100	Strengthen bolted connections at second floor braces. Add new steel lateral elements. Improve braced frame support at second floor. Strengthen roof diaphragm.	$25.72	$800,000
Subtotal Horace Mann School		--	**107,900**	--	--	**$5,450,000**
Beverly Vista	B	**4**	n/a	None	$0	$0
Beverly Vista	C	**4**	n/a	None	$0	$0
Beverly Vista	D	**4**	n/a	None	$0	$0
Beverly Vista	E	**4**	n/a	None	$0	$0
Subtotal Beverly Vista School		--	**n/a**	--	--	**$0**
Grand Total (to be strengthened)		--	**335,700**	--	--	**$14,412,500**

[1] See qualifications in section 7.2

[2] See Appendix A, Appendix to Seismic Evaluation for Horrace Mann School Building B-"The Rotunda" for discussion of associated costs.

8.0 Limitations

The seismic risk evaluation was performed by MHP on behalf of Dougherty + Dougherty Architects LLP for the purpose of evaluating the structural integrity of the buildings and determining the seismic risk at the project sites.

Physical testing was not performed and is considered outside the scope of this assignment. Intrusive testing was neither authorized nor performed.

The scope of work for the seismic review was based on standards developed and outlined by MHP, Inc. Differences, problems, and/or code violations were noted where observed; however, it is possible that areas containing deficiencies, physical inadequacies, or code and other regulatory violations may be present but were not observed at the time of the limited inspections. The recommendations and cost estimates provided in the report are intended to serve as general guidelines to be used in future repair, maintenance, and capital improvement decisions. The implementation of any recommendations will require specific details, plans, and specifications to be prepared by a licensed engineer or architect. Detailed cost estimates can be made based on the specific details and plans.

The information presented in this report has been developed in accordance with the above limitations, using that degree of professional care and skill ordinarily exercised under similar circumstances by engineers using the standards of practice and care normally exercised in the design and evaluation of investment-grade buildings in the local marketplace. No other warranty, express or implied, is made.

This report is subject to the limitations set forth above and is for the exclusive use of Dougherty + Dougherty Architects LLP and Beverly Hills Unified School District. Use by others is authorized only after acknowledging and accepting the limitations stated and upon the express written permission of MHP.

By:

Lance Kenyon

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Partner

STRUCTURAL ENGINEERS

SEISMIC RISK EVALUATION

BEVERLY HILLS HIGH SCHOOL
241 SOUTH MORENO DRIVE
BERVERLY HILLS, CALIFORNIA

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December 13, 2007
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TABLE OF CONTENTS

BEVERLY HILLS HIGH SCHOOL
241 SOUTH MORENO DRIVE
BERVERLY HILLS, CALIFORNIA

1.0 Introduction

At the request of LPA Inc. Architects, a seismic risk evaluation of the existing buildings at the Beverly Hills High School campus was conducted to identify school buildings that have a potential for suffering significant structural damage, and particularly those that present a life-safety risk, during future strong ground shaking that may affect the site. The information obtained from the assessment will allow the seismic risk of each building to be considered with other factors during the Master Planning process for the campus. For those buildings identified as having significant risk, potential strengthening schemes will be discussed, and rough cost estimates for recommended strengthening work will be developed.

The Beverly Hills High School campus was originally constructed in 1927 (Buildings B and H), while Building E was added shortly thereafter circa 1933. The buildings (B, H, and E) were seismically retrofit in 1936-1937 as a result of the devastation suffered by the City of Long Beach schools in the 1933 Long Beach earthquake. The swim-gym (Building F) was constructed in 1939-1940. New wings were added on the east and west ends of Building E circa 1967. Building A and the North Wing addition to Building B were constructed in 1967-1970. Buildings C and D were constructed at an unknown time, and are scheduled for demolition.

It is important to note that older buildings generally have a higher risk of damage during strong ground motion than newer buildings, as code requirements to address seismic forces were in their infancy during the 1930s through 1960s. It was not until July 1, 1978 when the 1976 Edition of the Uniform Building Code (UBC) was adopted that significant improvements in seismic design codes were required for school buildings. Since 1976, significant additional code upgrades have been adopted as a result of lessons learned in the Loma Prieta, Whittier, and Northridge earthquakes, and from University research and testing conducted in the past decades.

The scope of this review involves the detailed seismic evaluation of Buildings B, E, and H, and preliminary evaluations of Buildings A and F to determine if they pose a significant risk to occupants if subjected to strong earthquake ground motion. The seismic performance of the buildings is being evaluated per ASCE/SEI standards.

3.0 Seismic Evaluation Methodology

The purpose of this seismic evaluation is to identify critical structural elements in the project buildings that, when subjected to earthquake induced forces and displacements, may result in a significant life-safety or collapse hazard. Structural behavior representing a significant life-safety hazard includes yielding or failure of structural elements that potentially could cause local collapse, and the creation of falling hazards at exit ways due to deterioration of heavy finishes or excessive deflections. The expected seismic performance of the selected buildings was evaluated following procedures in ASCE 31-03 "Seismic Evaluation of Existing Buildings" (formerly FEMA 310). The detailed evaluations look at each structure's ability to meet a *Life Safety* level of performance, as defined by ASCE 31-03, for the designated earthquake hazard level.

3.1 Structural Analysis

For each structure evaluated, linear static or linear dynamic analysis procedures are used. The scope of this review involves the detailed seismic evaluation of Buildings B, E, and H, and preliminary evaluations of Buildings A and F. The detailed analysis of Buildings B, E, and H and Preliminary analysis of Buildings A and F starts with a defined earthquake ground motion force level, typically reported as mapped spectral accelerations expected for the specific site. The value of the spectral accelerations account for the general seismicity of the area and potential for strong ground shaking, proximity to known earthquake faults, and site soil conditions (to the degree they are known) including liquefaction and supporting soil or rock stiffness. The level of ground motion at each site is defined in accordance with the requirements of ASCE 31-03 and is based upon an earthquake hazard level that is equal to 2/3 of the Maximum Considered Earthquake (MCE). The MCE is defined as an earthquake hazard with a probability of exceedance of 2 percent in a 50-year exposure period (approximately 2500 year return period). See Section 4.4 for specific ground motion data used for each site.

To evaluate the building's response to the developed ground motion spectrum in an analysis, the building and primary lateral elements within are mathematically modeled by hand calculations that use general engineering principals (linear static) or, for the more complex structures, with two or three dimensional computer models (linear static or dynamic). The computer models are typically done using the SAP2000 or ETABS structural analysis software by Computers and Structures International, Berkeley, California.

Typically, in the linear static procedure, hand calculations are used to determine magnitude of lateral forces on individual elements within the building to investigate the elements adequacy to resist lateral loads. Lateral loads are calculated based upon the spectral accelerations at the fundamental period of the building and are applied statically. Distribution of the lateral load is relative to the mass distribution within the structure.

Where a linear dynamic analysis with computer modeling is used for the detailed evaluation, the earthquake response of the structure is approximated by first performing a modal analysis of the linear elastic model and then applying an appropriate response spectrum and damping level representing the input ground motion in accordance with ASCE 31-03. Periods and mode shapes are computed and earthquake forces calculated for key structural elements assuming structural behavior remains linear elastic. Typically a damping value of 5 percent for the structure is selected for dynamic analysis as a reasonably conservative estimate for post-yield behavior under the earthquake ground motions. Sufficient mode shapes are extracted, using the eigenvector analysis method, in order to achieve a minimum of 90-percent mass participation in both principal horizontal directions. The modal forces are combined using the complete quadratic combination (CQC) method. The dynamic forces obtained from the analysis are combined with appropriate dead and live load forces, with the resulting combined forces defined as the elastic demand force.

Whether calculated by the linear elastic computer model or by using general engineering principals, the elastic demand (D) for each element is compared to the capacity (C) of that element, where the capacity is defined as the expected elastic strength at yield. Element actions are defined as Deformation-Controlled or Force-Controlled. Except for brittle-failure mechanisms (such as concrete or masonry wall anchorage), most actions are considered Deformation-Controlled. For elements governed by Deformation-Controlled actions (those members capable of inelastic behavior) the yield capacity of the member is multiplied by a component modification factor to account for permissible deformations beyond yield. These modification factors are referred to as *m*-factors. Acceptable *m*-factors for various component actions are defined in ACSE 31-03 and vary depending on the level of evaluation performance desired (e.g. *Life Safety* for 2/3 MCE). Acceptable element performance is denoted as when the element capacity multiplied by the appropriate *m*-factor is greater than or equal to the demand from the analysis. Another way to define acceptable behavior is when the ratio of demand to capacity (DCR = Demand/Capacity) for each element is less than or equal to the acceptable values of *m* for a given type of action and performance objective. For Force-Controlled element actions DCR must be less than or equal to unity (in essence *m* = 1.0).

4.0 Site Specific Seismic Hazards

The seismic hazard evaluation identifies earthquake effects at the site (e.g., ground shaking or ground failure) and quantifies the likelihood of their occurrence, irrespective of buildings or other improvements on the site. Seismic hazards include strong ground shaking, ground rupture due to faulting, seismically-induced settlement, liquefaction, and slope failure.

4.1 Strong Ground Shaking

Beverly Hills has significant seismic hazards, due primarily to the presence of the Santa Monica fault and other nearby faults. Future ground motion at a specific site is often estimated based on a probabilistic seismic hazard analysis (PSHA) considering the location, geometry, slip rate and maximum magnitude for active and potentially-active faults in the region, and the use of ground motion attenuation relations suitable for the type of faulting and the site soil profile. Ground motion at a site is often characterized in terms of peak ground acceleration (PGA). Based on published geologic reports and maps, strong ground shaking may affect the Beverly Hills High School site as the result of earthquakes likely to occur on the following active regional faults:

ACTIVE REGIONAL FAULTS			
Fault or Fault Zone	**Distance and Direction From Site**	**Recent Activity**	**Maximum Magnitude**
Santa Monica (A)	<1/2 mile NW	--	6.6
Hollywood-Raymond (A)	1.5 miles NE	--	6.5
Newport-Inglewood (A)	2 miles SE	1933 M6.3	7.1
Malibu Coast (A)	7 miles W	1989 M5.3	6.7
Puente Hills (A)	7 miles E	--	7.1
Upper Elysian (A)	7 miles NE	--	6.4
Verdugo (A)	11miles NE	--	6.9
Palos Verdes (A)	12 miles SW	--	7.3
Northridge (A)	13 miles NW	--	7.0
Sierra Madre (A)	15 miles NE	1991 M5.8	7.2
San Andreas (A)	37 miles NE	1857 M7.8	7.8

4.2 Fault Rupture

California Earthquake Fault Zones (EFZs), established by the State of California under the Alquist–Priolo Earthquake Fault Zoning Act (first enacted in 1973), are delineated around known 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 faulting are considered when estimating the degree of earthquake-related damage for existing facilities located within the fault or drag zone.

The site is not located within a California Earthquake Fault Zone (nearest EFZ on the Newport Inglewood Fault). The closest mapped active or potentially active fault is the active Santa Monica Fault located less than 1/2-mile from the site. 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.

4.3 Other Earthquake Hazards

Seismically induced settlement, 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 and hazard maps indicate subsoils at the site consist of older Quaternary alluvium near the northwest (higher) portion of the campus and younger Quaternary alluvium for the remainder (lower portion) of the campus. The depth to ground water at the site is greater than 20 feet. The site is not located within a California Seismic Hazards Zone (SHZ) for liquefaction (Beverly Hills Quadrangle, official map released 3/25/99). Regional information indicates a low to moderate liquefaction potential. Based on the available information, the seismically induced liquefaction potential at the site is considered to be low.

The site is not located within a California SHZ zone for landslide (Beverly Hills Quadrangle, official map released 3/25/99). The site consists of moderate slope with the high portion near the northwest portion of the campus and slope downward towards the east and south. Based on this information, the potential for earthquake-induced landslide or slope stability failure is low.

The site is not located adjacent to a coastal or inland body of water or downstream of a dam, and is therefore not subject to flooding by earthquake-related tsunami, seiche, or dam failure.

4.4 Site-Specified Ground Motion

Ground Motion at the project site was evaluated in accordance with the requirements of ASCE 31-03 (formerly FEMA 310). Earthquake ground motion is based upon an earthquake hazard level that is equal to 2/3 of the Maximum Considered Earthquake (MCE). The MCE is defined as an earthquake hazard based on a probability of exceedance of 2 percent in a 50-year exposure period (approximately 2500 year return period). At 2/3 of the MCE level, as used in these building evaluations, the design earthquake is very similar to the earthquake hazard based on a probability of exceedance of 10 percent in a 50-year period (approximately 475 year return period). The 475 year earthquake is the defined earthquake ground motion used by the 2001 CBC code for the design of new buildings. The MCE Response Spectrum is defined by two values obtained from 2002 USGS study for rock; S_S and S_1, the Short-Period Spectral Response Acceleration and Spectral Response Acceleration at one second, respectively. The

Ss parameter is defined at 0.2 seconds; both the S_S and S_1 are listed in the table below. Actual spectral *design* values are modified for Site Class D (stiff soil). The Peak Ground Acceleration for a 475 year return period (similar to 2/3 MCE), Short-Period *Design* Spectral Response Acceleration Parameter (S_{DS}), and *Design* Spectral Response Acceleration Parameter at one second (S_{D1}) are also summarized in the following table:

SITE SPECTRAL RESPONSE ACCELERATION PARAMETERS					
Ground Motion Level	**PGA**	**S_S**	**S_1**	**S_{DS}**	**S_{D1}**
2/3 MCE (Earthquake Hazard used for ASCE 31 Evaluation)	0.59g	1.246g	0.424g	1.246g	0.64g
MCE - 2% in 50 years (2475 year return period)	0.88g	1.868g	0.636g	1.868g	0.95g
10% in 50 years (475 year return period)	0.49g	1.230g	0.443g	1.242g	0.69g

5.0 Seismic Evaluation Results and Recommendations

As previously discussed, the structural evaluations of the buildings were completed using the approach contained in ACSE 31-03. To accomplish this, an evaluation of the seismic strength of Buildings B, E, and H was completed using a linear-elastic computer model or calculations based upon basic engineering principals and relative rigidities of lateral force resisting components to determine demands on specific elements of the structure. Only preliminary analysis was completed for Buildings A and F. The seismic demands were compared to acceptable values defined in ACSE 31-03 for a *Life Safety* performance level, incorporating calculated modified inelastic demand-to-capacity ratios (*m*-factors) to account for the ductility of specific element types. The conclusions reached using this evaluation procedure are used to determine deficiencies (if any) in the buildings' ability to maintain a Life Safe performance level when subjected to the defined earthquake hazard.

The results for each building evaluated are provided below.

5.1 Building A (Classrooms, Cafeteria, Parking)

For the evaluation of the roof diaphragms and concrete shear walls, earthquake demand forces were calculated using the equivalent lateral static force method described earlier. For the upper story steel moment resisting frames, general redundancy was evaluated. The following observations were made as a result of our equivalent force analysis of the building subjected to forces as specified by ASCE 31-03:

- The capacity of the existing gypsum fill roof diaphragm does not meet the requirements for a *Life Safety* performance level. The maximum DCR for the diaphragm significantly exceeds *Life Safety* values.
- A major concern with future earthquake performance of moment frame buildings involves the existing beam-to-column connections of the moment resisting frames (MRF's). The connections in this building, as detailed on the construction documents, are similar to those that performed poorly in the 1994 Northridge earthquake. Although many affected connections were not damaged, a wide spectrum of unexpected brittle connection damage did occur. The cause of the connection damage has been attributed to factors related to the configuration of the beam-to-column interface, and limitations in the material properties, workmanship and inspection and testing of these joints. The ability of existing welded steel moment frame buildings to resist earthquake ground shaking through inelastic behavior is now understood to be significantly less than that previously assumed prior to the 1994 Northridge earthquake.

  For the preliminary analysis of the steel moment-resisting frames, the redundancy of the frames was calculated. Redundancy of the lateral force resisting system is an important factor affecting the performance of buildings in an earthquake. Buildings with a low redundancy, i.e. few lateral force resisting elements to resist the seismic loads, tend to suffer more damage under earthquake ground motions. The layouts of the moment frames within the building have a moderately high level of redundancy as calculated by current code criteria. Therefore, even if several of the moment-resisting frame connections were to fail, it would not likely lead to collapse of the building. The life-safety risk of the steel moment frame portions of the building was judged to not be a significant life-safety risk, based on the preliminary analysis.

- The capacity of the existing concrete walls to resist in-plane lateral forces appears to marginally meet the requirements for *Life Safety* performance level, based on the preliminary analysis.

Based upon the results of the structural analysis, the predicted structural performance of the building does not satisfy all of the provisions of *Life Safety* as outlined in ASCE 31-03. Seismic strengthening of the gypsum roof diaphragm would be required to reduce the life-safety hazards associated with the building. The structure should be seismically retrofit, as a minimum, to meet ACSE 41-06 structural performance guidelines for *Life Safety*.

5.2 Building B (Domestic Science, Classrooms, Administration, Auditorium, Music & Arts, and Drama Lecture)

To evaluate the seismic performance of the Building B structures, the elastic demand forces of concrete shear walls were calculated from a two-dimensional mathematical model developed for each building with an exception of Auditorium building (three-dimensional model) using the structural analysis software ETABS. Based on the requirements of ASCE 31-03, a linear static analysis, adopting the equivalent lateral static forces, was conducted for two-dimensional models.

For the Classroom building, one-third of the exterior concrete shear walls in longitudinal direction were modeled as a representative segment of the lateral force resisting system and is a justified simplification due to the repetitive shear wall pattern along the building. A concentrated load was applied at each floor level to account for the earthquake load. The in-plane and out-of-plane demand forces at wall piers were compared with the specified capacities of reinforce concrete shear walls. Wood floor diaphragm stresses and anchorages to the walls were evaluated using the static seismic load procedure described above.

For the Administration building, the exterior concrete shear walls in both longitudinal and transverse directions were modeled. A concentrated load was applied at each floor level. The demand forces of shear walls were compared with their in-plane and out-of-plane strengths. Wood floor diaphragm stresses and anchorages to the walls were also compared with the specified capacities.

The same methodology applies for the evaluation of the Domestic Science, Art and Music and the Drama Lecture buildings. The in-plane and out-of-plane demand forces of concrete shear walls in both longitudinal and transverse directions were compared with their capacities. Diaphragm stresses of wood roof and floors, and anchorages of the walls to the horizontal diaphragms were evaluated. In addition, the capacity and detailing of the steel braced frames of the Drama Lecture building were considered.

For the evaluation of Auditorium building, a linear elastic dynamic analysis of a three-dimensional ETABS model was conducted with input ground motion consisting of elastic response spectra as specified in ASCE 31. The ground motion used in the analysis was based on 2/3 of the maximum considered earthquake (MCE) which is approximately equal to ground motion with a 10% probability of exceedance in 50 years. Then, the DCRs of concrete columns and shear walls were determined and compared to allowable *m*-factors.

The following observations were made as a result of either equivalent lateral static load or response spectrum analysis of Building B subjected to 2/3 of maximum considered earthquake (MCE) level as specified in ASCE 31-03:

Building B (General);

- Existing strengthening plans from 1936 do not indicate any positive attachment of the newer gunite to the original clay-tile walls. Positive attachment is required to prevent the tile from peeling away from the exterior. Further evaluation of the existing condition is necessary and may require localized destructive testing. If, as it appears, there is no direct attachment between the unreinforced tile and gunite layers of the walls, strengthening will likely be required.
- Where they occur, unreinforced hollow clay tile interior nonbearing walls were strengthened by replacing the plaster on one side of the wall with a thin (1" thick) layer of lightly reinforced gunite. The gunite is not adequate to prevent damage or potential failure of the walls. In a strong earthquake these walls may collapse and thus, are considered a life safety hazard. To mitigate the potential hazard these walls should be removed and replaced with light gage steel framed walls.
- Typical roof and floor diaphragms consist of 1x straight sheathing. In some cases the straight sheathing is covered by other wood flooring. Straight sheathed diaphragms acting alone are not permitted for use as earthquake load resisting elements in school buildings by the California Building Code (2007 CBC, Section 3417.1.5). These types of diaphragms have very little strength to resist in-plane shears due to earthquake ground motions. Where straight sheathed diaphragms occur, new plywood sheathing should be added.

Classroom building;

- The in-plane and out-of-plane flexural and shear capacities of the shear walls meet the requirements for *Life Safety* performance level.
- Wood floor and roof diaphragm stresses do not meet the requirements for *Life Safe*ty performance level. The demand-capacity-ratio (DCR) of the diaphragm stress is 16% higher than the allowable '*m*' values for Life Safety.
- Floor and roof diaphragm-to-wall anchorage does not meet the requirements for *Life Safety* performance level. The DCR at longitudinal wall anchorage is approximately twice the acceptable level for Life Safety.

Administration building;

- Both transverse and longitudinal shear walls comply with the requirements for *Life Safety* performance level.
- Wood floor diaphragm stresses do not meet the requirements for *Life Safety* performance level. The DCRs are 9% to 37% higher than the allowable '*m*' values for Life Safety.
- At the roof diaphragm, the horizontal steel x-brace capacities are adequate to meet *Life Safety* performance level.

- Floor and roof diaphragm-to-wall anchorage does not meet the requirements for *Life Safety* performance level. The DCRs are 81% and 57% higher than the acceptable value for Life Safety in transverse and longitudinal wall anchorages, respectably.

Auditorium building;

- The column capacities considering the axial-flexural interaction do not meet the requirements for *Life Safety* performance level. The DCRs of the columns at second floor are noted up to 80% higher than the allowable '*m*' values for Life Safety.
- The shear wall capacities do not meet the requirements for *Life Safety* performance level. The DCRs of the shear walls at second floor were 1.78 to 2.34 times higher than the allowable '*m*' values for Life Safety.

Art & Music building;

- The capacities of shear walls do not comply with the requirements for *Life Safety* performance level. The in-plane flexural of the longitudinal shear wall is exceeded by 18% of the allowable '*m*' values for Life Safety although the shear capacity is in the acceptable range.
- Floor diaphragm stresses do not meet the requirements for *Life Safety* performance level. The DCRs are 1.11 and 2.61 times the allowable '*m*' values for Life Safety in transverse and longitudinal loadings, respectively.
- Floor diaphragm-to-wall anchorage marginally complies with the requirements for *Life Safety*.

Drama Lecture building;

- In-plane and out-of-plane capacities of the reinforced masonry shear walls meet the requirements for *Life Safety* performance level.
- The brace members of the steel braced frames do not meet the requirements for *Life Safety* performance level. In addition, the configuration of the braced frames is that of a 'k'-brace. Although permitted for use at the time of construction, 'K'-braces are no longer allowed in this application due to the limited ductility of the system and the large lateral loads that can be imposed on the columns at the intersection of the column and the bracing member. The braced frames should be strengthened by increasing the capacity of the frames and by altering the configuration of the braces.
- Roof shear stresses of the blocked plywood diaphragm meet the requirements for *Life Safety* performance level.
- Roof diaphragm-to-wall anchorage marginally complies with the requirements for *Life Safety*.

Based upon the results of the structural analysis, the predicted structural performance of the building does not satisfy the provisions of *Life Safety* as outlined in ASCE 31-03. Therefore, seismic strengthening of many elements of the structures would be required to reduce the life-safety hazards associated with the building. The deficiencies identified above should be addressed, and the structures should be seismically retrofit, as a minimum, to meet ACSE 41-06 structural performance guidelines for *Life Safety*.

5.3 Building E (Gymnasium and Locker Rooms)

For Building E a detailed structural evaluation was completed using linear static evaluation method. Elements of the structure were evaluated with equivalent lateral force analysis, and for the longitudinal perimeter walls; a supplemental ETABS two-dimensional computer model was conducted to determine the distribution of lateral forces to the wall piers and lintels. In either case, elements or portions of the building were subjected to forces as specified by ASCE 31-03 (as described in Section 3.0). Included in the evaluation were the in-plane and out-of-plane wall capacities, adequacy of lateral wall anchorage, and roof diaphragm capacity.

The following observations were made as a result of our detailed analysis:

- Where they occur, unreinforced hollow clay tile interior nonbearing walls were strengthened by replacing the plaster on one side of the wall with a thin (1" thick) layer of lightly reinforced gunite. The plaster is not adequate to prevent damage or potential failure of the walls. In a strong earthquake these walls may collapse and thus, are considered a life safety hazard. To mitigate the potential hazard these walls should be removed and replaced with light gage steel framing.
- Straight wood sheathing was standard construction and conformed to code when the buildings were built; however, straight sheathing is less capable under seismic loads in transferring diaphragm shear forces and controlling deflections than current designs, which typically use plywood sheathing. Due to the marginal capacity of straight wood sheathing the straight sheathed roof diaphragm at the main gymnasium is overstressed in shear. Demand capacity ratios (DCRs) exceed the allowable *Life Safety* '*m*' values by 10% to 20%.
- The in-plane and out-of-plane flexural and shear capacities of the shear walls and piers meet the requirements for *Life Safety* performance level.
- The DCRs for the wall anchors attached to the wood framing at the roof and floor level are greater than the allowable '*m*' values for *Life Safety*. However, if the steel framing is considered, as the walls span horizontally between the steel members, the wall anchorage is deficient only at the roof level.
- At the locker rooms (east and west ends of Building E), wall anchorage provided is minimal and does not meet a Life Safety level of performance along the north and south elevations.
- For the locker rooms roof and floor shear stresses of the blocked plywood diaphragm meet the requirements for *Life Safety* performance level.

Based upon the results of the detailed evaluation, the predicted structural performance of the Physical Education buildings does not satisfy all of the provisions of *Life Safety* as outlined in ASCE 31-03. Therefore, seismic strengthening of several elements of the structure would be required to reduce the life-safety hazards associated with the building. The deficiencies identified above should be addressed, and the structure should be seismically retrofit, as a minimum, to meet ACSE 41-06 structural performance guidelines for *Life Safety*.

5.4 Building F (Gymnasium and Natatorium)

For the evaluation of the Swimming Pool building, the elastic demand forces of wood arch frames were calculated from a two-dimensional mathematical model of the typical wood arched frame using the SAP2000 computer program. The arched frame consisted of three individual frames whose components were made of a series of double curved-tapered sections. In the model, the tapered section was simplified to be a double linearly-tapered section. The connections between tapered sections along the longitudinal direction in a frame were assumed to be pinned such that no moment could be transferred. At the bolt groups interconnecting the three frames, equal constraint technique was used to restrain the relative displacement between the frames at the connections. To account for the earthquake load, a uniformly distributed lateral load based on the equivalent lateral static force concept per ASCE 31-03 was imposed on both sides of the building frame. The forces obtained from the lateral load were combined with appropriate dead and live load forces, and the resulting combined forces were defined as the elastic demand force. The framing member and the corresponding connection demand forces were compared with their capacities. The frame analysis was supplemented by linear static evaluations of selected elements of the lateral force-resisting system.

The following observations were made as a result of either equivalent lateral static load analysis of Building F subjected to 2/3 of maximum considered earthquake (MCE) level as specified in ASCE 31-03.

- The flexural capacity of the individual framing member of the building arched frame does not meet the requirements for *Life Safety* performance level. The DCRs are noted 12% to 23% higher than the acceptable '*m*' values for Life Safety.
- The bolted connections of the building arched frame marginally comply with the requirements for *Life Safe*ty performance level.
- Although straight sheathing is used for the roof diaphragm, the stresses in the diaphragm are below the allowable levels and therefore, the diaphragm meets a *Life Safety* level of performance (assuming the arched trusses act as lateral force-resisting elements for the building).
- In the longitudinal direction, lateral forces are resisted by two bays of steel "X"-rod bracing. The "X"-rod bracing marginally meets the *Life Safety* compliance level.

Based upon the results of the structural analysis, the predicted structural performance of the building does not satisfy the provisions of *Life Safety* as outlined in ASCE 31-03. It is recommended that the wood arched frames be seismically strengthened to provide adequate resistance under major earthquake event. The structure should be seismically retrofit, as a minimum, to meet ACSE 41-06 structural performance guidelines for *Life Safety*.

5.5 Building H (Cafeteria)

To complete the detailed evaluation of the roof diaphragms and concrete shear walls, earthquake demand forces were calculated using the equivalent lateral static force method described earlier. The following observations were made as a result of our equivalent force analysis of the building subjected to forces as specified by ASCE 31-03:

- Existing strengthening plans from 1936 do not indicate any positive attachment of the newer gunite to the original clay-tile walls. Positive attachment is required to prevent the tile from peeling away from the exterior. Further evaluation of the existing condition is necessary and may require localized destructive testing. If, as it appears, there is no direct attachment between the unreinforced tile and gunite layers of the walls, strengthening will likely be required

- Unlike Building B the unreinforced hollow clay tile interior nonbearing walls were removed and replaced with light gage steel studs and plaster as part of the 1936 seismic strengthening. These lighter walls are not a life safety falling hazard.

- Typical roof and floor diaphragms consist of 1x straight sheathing. In some cases the straight sheathing is covered by other wood flooring. Straight sheathed diaphragms acting alone are not permitted for use as earthquake load resisting elements in school buildings by California Building Code (2007 CBC, Section 3417.1.5). These types of diaphragms have very little strength to resist in-plane shears due to earthquake ground motions. Where straight sheathed diaphragms occur, new plywood sheathing should be added.

- The in-plane and out-of-plane flexural and shear capacities of the shear walls meet the requirements for *Life Safety* performance level.

- Wood floor and roof diaphragm stresses do not meet the requirements for *Life Safe*ty performance level. The demand-capacity-ratio (DCR) of the diaphragm stress is higher than the allowable '*m*' values for Life Safety.

- Roof diaphragm-to-wall anchorage does not meet the requirements for *Life Safety* performance level.

Based upon the results of the structural analysis, the predicted structural performance of the building does not satisfy the provisions of *Life Safety* as outlined in ASCE 31-03. Therefore, seismic strengthening of several elements of the structure would be required to reduce the life-safety hazards associated with the building. The deficiencies identified above should be addressed, and the structure should be seismically retrofit, as a minimum, to meet ACSE 41-06 structural performance guidelines for *Life Safety*.

6.0 Conclusions and Recommendation Summary

The buildings included within this seismic study were each built prior to the introduction of modern seismic design procedures in building codes, which were first added to the Field Act in 1978. This building vulnerability, combined with the significant seismic hazards affecting the site, result in the deficiencies identified.

The seismic studies of the subject buildings have identified numerous seismic deficiencies that represent life-safety hazards, as defined by ASCE 31-03 criteria. However, some of the buildings have limited deficiencies in only certain elements, and will require less seismic strengthening than others. Presented below are the seismic deficiencies identified in the evaluation. The cost to seismically strengthen the buildings is dependent on many factors. These factors include:

- Desired Performance Level: Upgrading the seismic force-resisting systems of existing buildings to current code criteria is typically cost prohibitive. Most often, a desired performance level, such as Life-Safety, or Immediate Occupancy, is chosen. While reducing the risk to life in buildings is of utmost concern, some level of damage control is often desirable, particularly in facilities that need to be operational after a large earthquake. The incremental cost to provide some level of additional damage control is sometimes relatively small. Any seismic strengthening should, as a minimum, meet *Life Safety* performance level per the requirements of ACSE 41-06 "Seismic Rehabilitation of Existing Buildings" or requirements of Chapter 34 "Existing Buildings", 2007 California Building Code.
- Construction Phasing: If the seismic strengthening is completed in conjunction with other building upgrades (such as architectural, mechanical/electrical, ADA, communications, etc.), the cost will be much lower than if the seismic strengthening is completed as an independent phase.

The accuracy of cost estimates for seismic strengthening work is also directly related to the depth of the analysis and to development of detailed strengthening plans for the cost estimator to evaluate. Very conceptual details and plans for strengthening the buildings are included in Appendix A. More accurate cost estimates can be achieved by completing more detailed evaluations (particularly for Buildings A and F) and more so, by developing a complete set of preliminary strengthening plans.

RECOMMENDATIONS SUMMARY:

Building A

Based upon the deficiencies identified in Section 5.0, strengthening of Building A should consist of:

- Remove and replace the gypsum diaphragm with new metal deck diaphragm. Roof area is approximately 80,000 sf.
- A detailed evaluation and/or strengthening design may identify other areas requiring strengthening including, isolated shear walls, the stair towers, etc.

Building B

Based upon the deficiencies identified in Section 5.0, strengthening of Building B should consist of:

- Strengthen existing wood sheathed roof and floor diaphragms by overlaying with new 3/8" thick Struct I plywood and renailing diaphragm. Approximately 61,000 square feet of diaphragm.
- Anchor original unreinforced masonry walls to existing gunite walls. Anchorage would consist of new dowels or bolts embedded through the masonry and into the gunite with epoxy. Total effected area of wall is approximately 50,000 sq ft.
- Strengthen or provide new out-of-plane anchorage of heavy masonry/concrete walls at roof and floors. Anchorage would be spaced at a maximum of 4 feet on center. Total effected length of wall is approximately 2,800 linear feet.
- Remove existing unreinforced (or lightly reinforced) hollow clay-tile partition walls and replace with walls of light gage steel framing and gypsum wallboard. Total effected length of wall is approximately 700 linear feet.
- Strengthen shear walls and columns at the Auditorium building and strengthen longitudinal shear wall at the Art and Music building. Strengthening could consist of adding new layer of reinforced concrete over the existing walls or by infilling open areas along the wall lines with new concrete walls. Additional foundation work should be anticipated.
- Provide new steel tube or pipe braces, replacing the existing "k" braces, in the two steel brace frames in the Drama Lecture room.
- A detailed strengthening design may identify other areas requiring strengthening, including isolated shear walls, columns, etc.

Building E

Based upon the deficiencies identified in Section 5.0, strengthening of Building E should consist of:

- Strengthen existing wood sheathed roof diaphragm at the main gymnasium building by overlaying with new 1/2" thick Struct I plywood and renailing diaphragm. Approximately 14,000 square feet of diaphragm.
- Anchor original unreinforced masonry walls to existing gunite walls in limited areas, if required. Intrusive investigation and/or destructive testing would be necessary to determine if strengthening is required. Anchorage would consist of new dowels or bolts embedded through the masonry and into the gunite with epoxy. Total effected area of wall is approximately 5,000 sq ft.
- Strengthen or provide new out-of-plane anchorage of heavy masonry/concrete walls at roof of main gymnasium and at north and south walls of locker room. Anchorage would be spaced at a maximum of 4 feet on center. Total effected length of wall is approximately 400 linear feet.

- Remove existing unreinforced (or lightly reinforced) hollow clay-tile partition walls and replace with walls of light gage steel framing and gypsum wallboard. Total effected length of wall is approximately 450 linear feet.
- A detailed strengthening design may identify other areas requiring strengthening, including isolated shear walls, columns, etc.

Building F

Based upon the deficiencies identified in Section 5.0, strengthening of Building F should consist of:

- Replace two of the middle wood truss arches with new steel arches. As an option to strengthening the trusses, it may be possible to strengthen the existing wood sheathed roof diaphragm by overlaying with new 3/8" thick Struct I plywood and adding new plywood to the end walls of the building.
- A detailed strengthening design may identify other areas requiring strengthening, including shear walls, columns, etc.

Building H

Based upon the deficiencies identified in Section 5.0, strengthening of Building H should consist of:

- Strengthen existing wood sheathed roof and floor diaphragms by overlaying with new 3/8" thick Struct I plywood and renailing diaphragm. Approximately 19,000 square feet of diaphragm.
- Anchor original unreinforced masonry walls to existing gunite walls. Anchorage would consist of new dowels or bolts embedded through the masonry and into the gunite with epoxy. Total effected area of wall is approximately 2,500 sq ft.
- Strengthen or provide new out-of-plane anchorage of heavy masonry/concrete walls at roof and floors. Anchorage would be spaced at a maximum of 4 feet on center. Total effected length of wall is approximately 400 linear feet.
- A detailed strengthening design may identify other areas requiring strengthening, including isolated shear walls, columns, etc.

7.0 Limitations

The seismic risk evaluation was performed by MHP on behalf of LPA Inc. Architects for the purpose of evaluating the structural integrity of the building(s) and determining the seismic risk at the project.

Physical testing was not performed and is considered outside the scope of this assignment. Intrusive testing was neither authorized nor performed.

The scope of work for the seismic review was based on standards developed and outlined by MHP, Inc. Differences, problems, and/or code violations were noted where observed; however, it is possible that areas containing deficiencies, physical inadequacies, or code and other regulatory violations may be present but were not observed at the time of the inspection. The recommendations and cost estimates provided in the report are intended to serve as general guidelines to be used in future repair, maintenance, and capital improvement decisions. The implementation of any recommendations will require specific details, plans, and specifications to be prepared by a licensed engineer or architect. Detailed cost estimates can be made based on the specific details and plans.

The information presented in this report has been developed in accordance with the above limitations, using that degree of professional care and skill ordinarily exercised under similar circumstances by engineers using the standards of practice and care normally exercised in the design and evaluation of investment-grade buildings in the local marketplace. No other warranty, express or implied, is made.

This report is subject to the limitations set forth above and is for the exclusive use of LPA Inc. Architects and Beverly Hills Unified School District. Use by others is authorized only after acknowledging and accepting the limitations stated and upon the express written permission of MHP.

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