

# Portland State University Lincoln Hall

## FEMA-178 Seismic Evaluation



Prepared For:  
**Soderstrom Architects, P.C.**  
1200 N. W. Naito Parkway, Suite 410  
Portland, OR 97209



Prepared By:  
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March 2000



# VLMK Consulting Engineers

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March 24, 2000

Mr. Cameron Hyde  
Soderstrom Architects, P.C.  
1200 N.W. Naito Parkway, Suite 410  
Portland, OR. 97209

RE: FEMA-178 Seismic Evaluation  
Lincoln Hall - Portland State University

Dear Mr. Hyde,

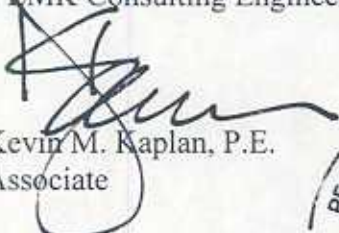
In accordance with our proposal, VLMK Consulting Engineers has completed a Seismic Evaluation of the Lincoln Hall Building located on the campus of Portland State University in Portland, Oregon.

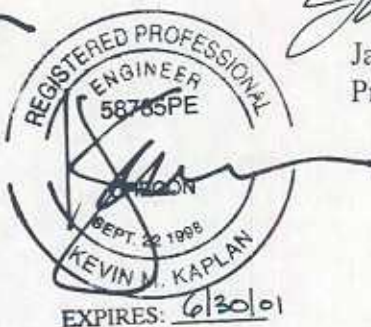
Following the FEMA-178 guidelines, a few deficiencies were identified in the building. Chief among these is a lack of lateral load resisting elements in several areas of the building. A number of nonstructural deficiencies were also identified largely relating to a need for bracing and restraint of building components and contents. In our opinion, the building represents a moderately high seismic risk and can expect to suffer significant damage following a large earthquake, including the possibility of collapse.

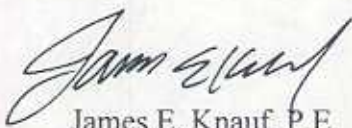
We appreciate the opportunity to be of service and look forward to working with you on this and other projects in the future. If you have any questions or concerns regarding this report or would like further information, please do not hesitate to contact us at your convenience.

Sincerely,

VLMK Consulting Engineers

  
Kevin M. Kaplan, P.E.  
Associate



  
James E. Knauf, P.E.  
Principal



# Portland State University Lincoln Hall FEMA-178 Seismic Evaluation

## SUMMARY

At the request of Soderstrom Architects, P.C., VLMK Consulting Engineers has completed a seismic investigation of the Lincoln Hall Building located on the campus of Portland State University. The report follows the format and guidelines outlined in the National Earthquake Hazards Reduction Program (NEHRP) Handbook for the Seismic Evaluation of Existing Buildings – Federal Emergency Management Agency (FEMA) 178 / June 1992. Information about the building was obtained from a review of the original construction documents, visits to the site, engineering analysis, material testing, and geological information.

Lincoln Hall, originally constructed in 1918, was first designed for use as Lincoln High School. Throughout the years the building has taken several roles, and is currently the home of the School of the Performing Arts at Portland State University. Based on the construction drawings obtained by VLMK Consulting Engineers, the building has gone through two major renovations. The first was in 1966 and consisted of expanding the existing basement level gymnasium to a lecture hall. Most of the building renovations done in 1966 were architectural in nature and did not significantly impact the existing structural system. The second significant renovation, done in 1974, impacted several portions of the buildings structural system. The main aspects of this renovation were to expand the main auditorium and backstage area, expand the band practice room in the basement, and to modify the existing lightwell areas to create more usable space. Most of the modifications done to the structural system at this time did not involve increasing the lateral capacity of the building. The lateral capacity was actually compromised during this remodel due to significant holes being added in the existing concrete floor diaphragms for the new band room and orchestra pit.

In an earthquake, "shaking" ground motion causes forces to act horizontally within a building. In Lincoln Hall, the primary lateral resisting elements are its concrete floor diaphragms and various concrete and unreinforced masonry shear walls. In a building of this type the seismic horizontal forces are first resisted by the roof and floor diaphragms which then transfer the force to the shear wall system. The shear walls then transfer the forces down to the foundation. Adequate strength and connections are required between the respective members for this system to perform. The concrete and unreinforced shear walls act together as a system, each taking a portion of the lateral force based on their specific relative rigidity within the entire system. The more rigid the wall is within the lateral system, the more force the wall will attract and have to resist. The majority of the building's shear walls occur at the perimeter of the building and are comprised of a concrete shear walls at the basement level and unreinforced masonry shear walls between the floors above the first floor

level. There are also two interior rectangular shaped groups of shear walls that are comprised of unreinforced masonry. Several of these walls are overstressed as outlined in the structural deficiency portion of the report to follow. Mitigating these overstressed shear walls would require the addition of new lateral load resisting elements, most likely in the form of new concrete shear walls. An option to adding several new lateral elements would be replacing the heavy hollow clay tile interior walls with lighter steel stud walls. This would significantly decrease the building's seismic mass thereby decreasing the seismic design force on the building. Some new lateral elements would still be required, although the quantity of these would more than likely be decreased. Additionally, if the interior hollow clay tile walls were replaced with steel stud walls, new braced frame elements could be incorporated in the new walls.

A number of nonstructural deficiencies were also identified in the building and can generically be categorized by a significant need for restraint. From file cabinets and shelving units, to ceilings, light fixtures, stage materials and piping - each needs to be braced and secured to withstand lateral movement.

Although earthquakes and the associated damages they cause are not easily quantifiable, the potential for significant damage of the structure in a large seismic event, in our opinion, is moderately high. Mitigation of many of the structural deficiencies present in the building could be achieved by providing additional lateral elements. Note that the amount of upgrade can be adjusted to meet different performance levels, which range from collapse prevention to immediate occupancy following an earthquake.

Portland State University  
Lincoln Hall  
FEMA-178 Seismic Evaluation

At the request of Soderstrom Architects, P.C., the following Seismic Evaluation has been prepared for the Lincoln Hall Building located on the Portland State University Campus in Portland, Oregon (See Photographs #1 through #6). The evaluation follows the guidelines outlined in the National Earthquake Hazards Reduction Program (NEHRP) Handbook for the Seismic Evaluation of Existing Buildings – Federal Emergency Management Agency (FEMA) 178 / June 1992.

### **REPORT SCOPE AND LIMITATIONS**

The primary purpose of an evaluation conducted under the NEHRP Handbook, commonly referred to as FEMA-178, is to identify life safety hazards posed by the building, or a particular building component. As described in the FEMA-178 document, "A building does not meet the life-safety objective of this handbook if, in an earthquake, the entire building collapses, portions of the building collapse, components of the building fail and fall, or exit and entry routes are blocked preventing the evacuation and rescue of the occupants." Consequently, the report scope is largely limited to identifying those items that relate to life safety. Strictly speaking, it does not address issues such as code compliance and damage control, nor does compliance with the report evaluation ensure that the building will be serviceable following an earthquake. Additionally, the scope of this report does not include designs to correct any items or components that are determined to be deficient. Nevertheless, some comments relating to these issues are provided in order to help the reader better understand the building's condition and plan for future upgrades. In addition, the report also contains a review of many of the non-structural elements (those components which are not part of the building's lateral force resisting system), which can pose a hazard to people in and around the building during an earthquake.

## PREPARATION SOURCES

Preparation of the Seismic Evaluation was based upon information obtained from several different sources that included:

- (1) "Lincoln High School" architectural and structural construction documents designed by Whitehouse and Fouilhox Architecture, provided by Portland State University, dated 1918.
- (2) "Portland State College - Old Main Basement Lecture Room" architectural and structural construction documents designed by Campbell, Miller, Michael Architects, provided by Portland State University, dated March 9, 1966.
- (3) "Portland State University -1974 Alteration" architectural and structural construction documents designed by Wegroup Architects and Planners, provided by Portland State University, dated February 4, 1974.
- (4) "Lincoln Hall" existing floor plans by Portland State University, provided by Portland State University, dated September 17, 1997.
- (5) Several visits to the site for observation and investigation.
- (6) Discussions with facilities personnel familiar with the building and its history.
- (7) Discussions with geotechnical engineers familiar with the existing geotechnical conditions at the building's site.
- (8) Currently available geological mapping for the area.
- (9) Brick shear, concrete compression tests, and clay tile shear tests results, tests performed by Carlson Testing, Inc.

## BUILDING TYPE, CONSTRUCTION, AND HISTORY

The Lincoln Hall building is a three-story unreinforced masonry building with a daylight basement covering a footprint area of approximately 37,820 square feet (See Figures A-0, A-1, A-2, and A-3). The roof and floor diaphragms consist of a multitude of different structural slab types with varying thickness and span. However, the majority of the slabs utilize a system of 3" to 10" deep one-way span clay tile form with 2" of concrete slab. Slab reinforcing constructed between the clay tile joints. A combination gravity frame system using both concrete and steel beams and columns provides the vertical support for the floor slabs. Three large steel trusses, pre-fabricated out of double-angle members, provides the vertical support for the large center span of the roof slab. The foundation (See Figure S-0) is comprised of conventional spread footings with concrete retaining walls around the perimeter of the building. The exterior walls are constructed with plaster finishes over multi wythe unreinforced masonry walls.

On the South side of the building, additional exterior retaining walls create two light wells into the basement space. Along the other three elevations, exterior basement windows are present near the basement ceiling level. Interior non-structural walls are comprised of plaster finishes applied over hollow clay tile walls.

Based on the original Lincoln High School drawings, as well as other resources, the original building was constructed in 1918. In March of 1966, the first of two major remodels was done to the then Portland State College. This remodel included the alteration to the existing basement gymnasium to create a new lecture hall. As part of this renovation, interior architectural walls were added to change the shape of the rectangular gymnasium area to its current configuration. Structurally, this remodel consisted of altering the existing slab on grade to achieve the current sloped lecture hall floor. This revision was accomplished by removing a portion of the existing slab on grade and replacing it with a lower sloped slab on grade. Additionally, a portion of the existing slab on grade was covered with structural fill and a new slab on grade was constructed.

In 1974 the second remodel was done to the building. The main intent of this renovation was to modernize the building for the changing needs of the School of the Performing Arts. The biggest change during this renovation was to the main auditorium. These changes involved removing the existing balcony at the second floor and replacing it with a new sloping "stadium" type slab on steel deck. This new slab is continuous from the back of the auditorium to the front and is supported by typical structural steel beam and girder framing bearing on the top of steel pipe columns and/or on unreinforced masonry shear walls. The steel pipe columns are supported at the top of the slab by concrete beams at the first floor. The center portion of the east exterior wall and chimney was removed and replaced with a new structural steel framed wall placed farther east resulting in an expanded backstage area. Other major structural changes included removing large sections of the first floor slab to expand the ceiling height of the band practice room in the basement, and to create a new orchestra pit for the main auditorium.

Several other architectural and/or minor structural changes were also done during this remodel. Such changes included opening up the space in the interior light wells to create more classroom space and new entry stairs into the main auditorium. A new elevator and associated elevator pit was also added in the northeast corner of the building. The projection room in the basement lecture room was completely remodeled, as well as many of the interior non-bearing partition walls. Several new openings were also added in the existing floor slabs and un-reinforced masonry shear walls.



## **CONCURRENT VAULT AND UTILITY STUDY**

In addition to this FEMA - 178 report other modifications and upgrades are planned for Lincoln Hall. Please refer to "Portland State University - Lincoln Hall Vault and Utility Study", dated February 3, 2000 issued by Soderstrom Architectes, P.C.

The purpose of the report noted above was to provide Portland State University with an analysis of the current condition of the electrical and mechanical systems of Lincoln Hall, as well as the structural recommendations for the reconstruction of the Broadway Avenue sidewalk vault. The report presents documentation of existing conditions, problem descriptions, solutions, and recommended priority items. This report is related to the FEMA - 178 report only in the fact that these reports have similar timelines and are briefly interrelated. This reference is noted only so that the reader can be aware of additional upgrade work that is in progress at Lincoln Hall.

**Table 1 – Typical Building Construction and FEMA-178 Type Classification**

<b>Building Element</b>	<b>Description</b>
Approx. Square Feet	37,820 (Building Footprint)
Roof System (See Figure A-4) (See Figure S-4)	Mixed levels and varying types of concrete slab systems (listed below) supported by structural steel "I" or wide flange shaped beam and girder framing. All original structural steel framing was encased in concrete. Interior roof over the auditorium utilizes (3) 7'-0" deep double angle steel trusses. Slab systems consisting of either 3" to 10" clay tile with 2" of concrete slab, 3-1/2" solid reinforced concrete slab, or 3-1/4" & 3-1/2" reinforced lightweight concrete slab over 2" metal deck.
Third and Second Floor System (See Figure A-2) (See Figure A-3) (See Figure S-2) (See Figure S-3)	Same as Roof system as indicated above, neglecting the (3) steel trusses.
First Floor System (See Figure A-1) (See Figure S-1)	Same as Roof system as indicated above, neglecting the (3) steel trusses. Additionally, (4) reinforced concrete beams of varying size along with (4) reinforced concrete columns that vary in size support the floor slab above the lower auditorium. The sizes of these members are as indicated on Figure S-1.
Basement (See Figure A-0) (See Figure S-0)	Slab on grade system consisting of slab depths of 4 to 5 inches in thickness.
Vertical Load Bearing System	A combination of steel "H" or wide flange shaped structural steel columns, reinforced concrete columns, and masonry/concrete walls. All original structural steel framing was encased in concrete.
Foundation System (See Figure S-0)	Conventional concrete spread footings.
Lateral System	Varying thickness unreinforced masonry shear walls. Reinforced concrete shear walls from the basement to first floor elevation.

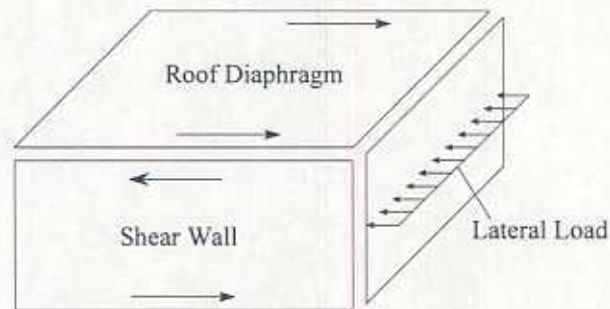
The Building Type Classification used by the FEMA-178 document for this type of building is a Type 15 – Unreinforced Masonry Bearing Wall Building. Additionally, the evaluation statements associated with the Building Type Classification for the Basic Building System has been utilized.

## SEISMIC LOAD PATH

The seismic load path in any building is simply the means by which loads, acting laterally on the building, are transferred from the roof and each floor level down to the foundation. The following paragraphs explain this concept in a generic sense and then specifically discuss the load path present in Lincoln Hall building.

### General Explanation

Loads such as wind and earthquake are typically considered to act on a building in a lateral (horizontal) direction, hence the name lateral load. To understand how loads are transferred through the building in a simplified case, consider the one story illustration shown below. A lateral load is applied perpendicular to a wall. The wall (concrete and unreinforced masonry walls at Lincoln Hall) then spans, much like a beam, between the foundation and roof (or floor in a multi-story building). As a result, a portion of the load is then transferred into the roof. The roof (called a diaphragm), spanning between the walls on each side, in turn transfers the load in the form of shear. The shear load is then transferred through the walls down to the foundation and into to the ground, completing the lateral load path.



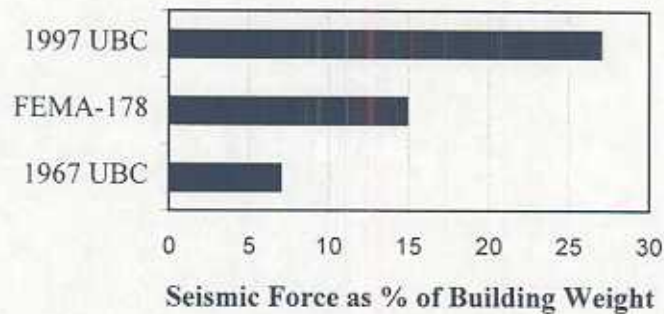
Although these load paths become more complex in larger and multi-story buildings, the basic underlying concepts still apply. A complete load path must be defined for any building. If a portion of this path is missing, the building has the potential to collapse. For instance, in the illustration above, if the lateral load resisting elements (shear walls) were removed, it is clear that the structure would be unstable and easily collapse. Try removing two sides of a cardboard box and then pushing on it. The same type of argument might also be made if the connection between the diaphragm and roof was missing or lacked adequate strength. Again, various degrees of instability would be introduced.

### Specific Description

The lateral load resisting elements utilized in Lincoln Hall building are the culmination of unreinforced masonry and reinforced concrete shear walls lining the perimeter of the building. In addition, original interior light wells forming two rectangular shaped groups of shear walls oriented longitudinally in the east and west direction aid in resisting lateral loads. Each of the four rigid diaphragms are continuously connected to these walls through a

combination of concrete encased structural steel framing, and slab reinforcing dowels. Refer to see Table 1 for a summary of these systems.

Calculations of lateral seismic forces are a function of a building's weight. In the intervening years since the original building's construction, the opinion and knowledge of seismic design has changed and improved significantly. Correspondingly, the code prescribed seismic design forces have increased substantially to reflect this new gained philosophy and knowledge. The following graph illustrates the changes in required seismic forces, as they relate to this structure expressed as a percentage of the building's weight. The change, for instance, between the 1967 (7%) and 1997 (27%) Uniform Building Code (UBC) seismic design force levels represent an increase of well over 300% as shown below.



### QUICK CHECK

One of the biggest problems with this structure is the fact that none of the past modifications to this building have put any improvement into its seismic load resisting capacity. Many of these changes have resulted in removing significantly large sections of the masonry shear walls and concrete diaphragms as well as adding seismic mass to the structure. In addition, many of the prior modifications have altered the structure such that some lateral load resisting elements of the building are subjected to a larger portion of the building's design lateral load than the building's original design intended. This is coupled with the increased seismic design force levels.

Another difficulty with the seismic analysis of this building is lack of design and construction information available. Much of the necessary existing information required to accurately analyze the overall lateral resisting capacity of the building and its elements was not commonly detailed on structural or architectural drawings during the time period when it was designed.

Due to the size and geometric complexity of the building, a dynamic response spectrum analysis method was used to evaluate the building. This was accomplished using the ETABS computer aided modeling program, which simulated the dynamic effects of a seismic event. The response spectrum methods used as a basis for this report are as outlined in Section 1631 of the 1997 Uniform Building Code with forces scaled to match the FEMA - 178 force levels. The computer program is able to distribute the lateral forces to the various shear elements based upon their relative rigidities. Similarly, overturning moments resulting from

shear loads were also investigated for several lateral load resisting elements based upon their contribution to the overall lateral load resisting system for a particular building floor level.

## EVALUATION STATEMENTS

In preparing a Seismic Evaluation with the FEMA-178 guidelines, a series of true or false statements are evaluated for a given building. A 'True' response to a statement indicates that a given item satisfies the FEMA-178 guidelines. 'False' responses, however, indicate an area that does not meet the guidelines and should be corrected and/or further evaluated.

Therefore, the statements discussed below were either found to be 'False', could not be answered definitively within the scope of this report, or merited further comment. A complete listing of the evaluation statements is given in Section E. It is suggested that they be reviewed by the reader not only to gain a more complete understanding of the strengths and weaknesses of the structure, but also to better understand some of the specific concerns which should be avoided in the future, particularly with regard to nonstructural items.

### Building Systems

- **Load Path:** The seismic load path for each of the principle directions is described above in the section "Seismic Load Path." The Lincoln Hall building appears to have a complete load path for seismic force affects from any horizontal direction that serves to transfer the inertial forces from the mass to the foundation.
- **Redundancy:** Redundancy is a fundamental characteristic usually required for good earthquake performance. Redundancy is difficult to quantify or codify, but when it is present, it makes for better ultimate performance since it tends to mitigate high demand / capacity ratios. The building should be provided with a redundant system such that a failure of a single member, connection, or component does not adversely affect the lateral stability of the structure. By inspection, Lincoln Hall has several lateral load-resisting elements in each of the principle directions making for a generally redundant structure. In addition, the building includes several hollow clay tile interior walls that would increase the redundancy of the building even though these walls were not included in the analysis of the buildings lateral system.

### Configuration

- **Weak Story:** A weak story exists when the story strength of a particular level is less than 80% of the strength of the story above. By inspection, the Lincoln Hall building has significant strength discontinuities in the vertical load resisting elements. In particular, this occurs in the east and west directions between the first and second floors where openings occur in the original interior light well walls. This subject is further discussed in item #1 of the Structural Deficiencies section.
- **Soft Story:** A soft story exists when the lateral stiffness of a particular level is less than 70% of that in the story above or less than 80% of the average stiffness of the three stories above. In the Lincoln Hall building a soft story does exist in the two interior shear

walls between the first and second floor. This subject is further discussed in item #1 of the Structural Deficiencies section.

- **Geometry:** A geometrical irregularity exists when setbacks in the horizontal dimension of the lateral force resisting system occur in more than 30% of a story relative to adjacent stories. In the Lincoln Hall building there does not appear to be any significant geometric irregularities.
- **Mass:** A mass irregularity exists when there is a change in effective mass of more than 50% from one story to the next, including light roofs. In the Lincoln Hall building there do not appear to be any significant mass irregularities.
- **Vertical Discontinuities:** A vertical discontinuity exists when a particular lateral resisting element is not continuous throughout the various levels of the building. This commonly occurs in the condition of a discontinuous wall. Without vertically continuous elements the lateral force has to be developed through either supporting columns or diaphragms to a connection element that is not in the same vertical plane. The potential hazard occurs when the transferring element is not designed for any of the additional overturning forces that occur due to this separation in vertical planes. Vertical discontinuities exist in the four major corner shear walls of Lincoln Hall in the east and west directions. These shear walls step back approximately four feet at the first floor. The result is that the first floor diaphragm is required to transfer the lateral loads between the elements. The FEMA – 178 “Quick Check” specifies that the vertical forces, including the added overturning and P-delta forces, be increased by a factor of  $Cd/2$  (1.5 minimum).
- **Torsion:** A well balanced system with respect to torsion is one where the lateral load-resisting elements are equally spaced such that the center of mass for the building is relatively close to the center of rigidity between the elements. Significant torsion is defined as any condition where the distance between the center of mass is greater than 20% of the width of the structure in either major plan dimension. With a building subject to torsion there can be significant rotation in the floor or roof diaphragm. The hazard associated with this rotation is in the columns that support the diaphragm, especially if the columns are not part of the lateral-force-resisting system. The tops of the columns are forced to move laterally with respect to the bottoms of the columns at the floor level below. Such columns are often designed without regard to these movements. Any building, with almost any amount of torsion, can be designed to meet code forces, but buildings with severe torsion are not likely to perform well in earthquakes. By inspection and calculations, due to the number and locations of lateral force resisting elements similar to those discussed in the redundancy description, the building is not subject to a significant amount of torsion.
- **Adjacent Buildings:** There is no immediate adjacent structure that is less than half as tall or has floors or levels that do not match those of the building being evaluated. A neighboring structure is considered to be “immediately adjacent” if it is within 2 inches multiplied by the number of stories away from the building being evaluated. The

associated problem of buildings being located close together is damage in an earthquake caused by the buildings behaving differently and pounding into each other.

### Materials and Conditions

- **Deterioration of Steel:** There is no evidence of significant visible rusting, corrosion, or other deterioration in any of the lateral load resisting systems. However it may be possible to have some corrosion of the reinforcing steel in the concrete retaining walls forming the Boiler Room on the east side of the building at the basement level, due to the visible signs of water leakage in that area. (See Photographs #7 through #14 taken in the Boiler Room area).
- **Deterioration of Concrete:** There is no evidence of significant deterioration of concrete in any of the lateral load resisting systems. However, there is evidence of concrete deterioration visible in several locations at the sidewalk slab over the Boiler Room at the east of the building at the basement level. Additionally, there are visible signs of water leakage in this area (See photographs #7 through #14 taken in the Boiler Room area).
- **Concrete Wall Cracks:** There is no evidence of significant concrete wall cracks in any of the lateral load resisting systems. However, there is evidence of concrete wall cracks visible in locations in the Boiler Room on the east of the building at the basement level. (See photographs #7 through #14 taken in the Boiler Room area).
- **Masonry Joints:** The mortar cannot easily be scraped away from the joints by hand with a metal tool and there are no significant areas of eroded mortar. In general, mortar joints appear to be in good condition. Additionally, as part of this report, several brick shear tests were taken. See the Test Data report section for the results of these brick shear tests.
- **Masonry Units:** There is no visible deterioration of large areas of masonry units. In general, masonry units appear to be in good condition. Additionally, as part of this report, several brick shear tests were taken. See the Test Data report section for the results of these brick shear tests.
- **Cracks in Infill Walls:** There are no diagonal cracks noted in the infilled walls that extend throughout a panel or are greater than 1mm wide.

### Concrete Shear Walls

- **Shearing Stress Check:** The building fails the Quick Check method of testing the shearing stress in a few of the lower level concrete shear walls. Specifically, the few overstressed lower level basement walls occur on the west and east sides of the building.
- **Overturning:** The building has shear walls with  $h_w / l_w$  ratios of greater than 4 to 1. Additionally, a few shear walls were checked for overturning and determined to be inadequate.

### Unreinforced Masonry Shear Walls

- **Shearing Stress Check:** The building fails the Quick Check method of testing the shearing stress in the unreinforced masonry shear walls. Several shear walls fail the shear stress criteria in both building directions, several areas of the building and in all floor levels. Specifically, a few of the more overstressed walls occur in the interior shear walls between the first and second floor levels where openings occur at the original light wells due to forces in the east and west directions.

### Capacity of Foundations

- **Overtipping:** The building's foundation capacities will require further investigation. The foundations are expected to need some remedial work, mostly relating to the need for increased bearing capacity due to the increase in the seismic design lateral loads. The Lincoln Hall building's ratio of the effective horizontal dimension, at the foundation level of the seismic resisting system, to the building height (base/height) exceeds 1.4Av. Additionally, the shear and moment capacity of the foundation elements should also be further investigated. The existing soil bearing capacity should be determined as a part of this more detailed investigation.
- **Sloping Sites:** The building is set on a sloping site exceeding one-half story.

### Geologic Site Hazards

As previously noted, no original soils report was obtained for this report. Current DOGAMI maps were reviewed for the site's potential Relative Liquefaction Hazard. Listed below are geologic hazards that are believed to exist at the site, as well as a brief explanation of the existing geotechnical conditions per the above noted information.

- **Liquefaction:** Liquefaction susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance do not exist in the foundation soils at depths within 50 feet under the building. According to DOGAMI maps, the site is located in an area where the Relative Liquefaction Hazard potential is low.
- **Slope Failure:** The building site is sufficiently remote from potential earthquake induced slope failures or rockfalls to be unaffected by such failures and is expected to be capable of accommodating small predicted movements without failure. The building location is on a site that is located in an area where slope failure problems have a low probability.
- **Surface Fault Rupture:** Surface fault rupture and surface displacement at the building site is anticipated. The proximity of the known active Portland Hills Fault is within 1/4 mile from the building site.



## Nonstructural Elements

An overview of the nonstructural elements was performed with areas of deficiency or concerns noted below. The evaluation statements (See Section E) provide additional insight on the types of items which are of concern and should be avoided.

- **Partitions:**

- a. Unbraced hollow clay tile partition walls exist throughout the building. These hollow clay tile partition walls are brittle and subject to shattering.
- b. Interior partition walls that extend full height are not detailed to accommodate the expected interstory drift of the building during a seismic event and may fail creating means of egress concerns.
- c. The tops of partitions that only extend to the ceiling line do not have adequate lateral bracing. These partition walls may overturn or buckle due to lack of lateral bracing in a seismic event.

- **Ceiling System:**

- a. The suspended ceilings, plaster ceilings, and ceiling supported lighting fixtures in several areas are believed to be without lateral bracing, aside from being attached to structural walls and interior partition walls. Lay-in ceiling tiles or plaster ceilings can drop out of the grid and can hazards at areas of egress. (See Photographs #15 and #16)

- **Light Fixtures :**

- a. Light fixtures are supported independently of any ceiling suspension system, but are not laterally braced to the structure. Swaying due to not being braced may cause the light and/or the fixture to break after encountering other building components. (See Photographs #17 and #18)
- b. Although the diffusers on most of the fluorescent light fixtures do not have specific safety devices to restrain them from falling, the configuration for the typical fixtures appears to provide adequate constraint.
- c. Pendant light fixtures should be replaced or braced. (See Photographs #17 and #18)

- **Parapets, Cornices, Ornamentation, and Appendages :**

- a. Cornices, parapets, and other appendages that extend above the highest anchorage level or cantilever from exterior wall faces and other exterior wall ornamentation appear to be well anchored to the structural system. The exact connections of these parapet cornices and appendages or connection capacities could not be verified from the existing construction drawings and should be further investigated. (See Photographs #37 through #40)

- **Buildings Contents and Furnishings:**

- a. In several areas of this building, the contents represent a significant hazard to occupants during an earthquake. Items stored on shelves if not restrained can become a falling hazard. (See Photographs #19 through #21)
- b. Tall, narrow (having a height-to-depth ratio greater than 3) storage racks, file cabinets, lockers, refrigerators, bookcases should be anchored to the floor and/or adjacent walls. These tall and narrow items can become falling hazards in an earthquake. (See Photograph #22)

- **Mechanical and Electrical Equipment**

- a. Mechanical equipment in some locations is not anchored to the structure. Significant equipment in some locations is not braced or anchored to the slab. These items can become loose or fall. (See Photographs #24 and #36)

- **Piping**

- a. Several pipe runs, especially in the attic spaces, are not adequately braced. Fire sprinkler branch and main lines should be braced. These systems should be operational after an earthquake. (See Photographs #25 through #29)

- **Ducts**

- a. Mechanical ductwork in some locations is not laterally braced. Suspended equipment that is not adequately braced is more susceptible to damage than floor, roof, or wall mounted equipment. (See Photograph #23)

## DEFICIENCIES AND POTENTIAL CORRECTIONS

The following paragraphs summarize the major deficiencies identified in the building and discuss, in broad stroke, possible means for correcting them. Specific design and preparation of seismic upgrading and retrofit plans is beyond the scope of this report. Deficiencies are divided into two classifications, structural and non-structural. Structural elements are the buildings primary load carrying systems. Non-structural elements are those items that do not directly support the building loads, and include the contents and components such as partition walls, exterior finish systems, etc. In deciding on an appropriate level of seismic upgrade to be included in remodel plans, a balance should be achieved between the two classifications.

### Structural Deficiencies

1. The largest structural deficiency is lack of adequate shear walls throughout the building. During the building's history, the amount of lateral load resisting shear walls has been reduced while the building's weight has been increased. Several of the openings that exist in the major shear walls create "soft story" and "weak story" deficiencies in the building's lateral load resisting system. This is mostly evident at the interior unreinforced masonry shear walls between the first and second floors. The original openings in the interior shear walls between the roof and the second floor were filled during the 1974 remodel. These elements, now possessing more stiffness, attract more seismic force. Consequently, a large amount of the total base shear of the building for these floor levels is now being resisted through these interior shear walls. This same load, in turn, has to then be transferred down through the walls between the first and second floors where the walls have been greatly reduced. These overstressed walls are the single largest deficiency of the structure.
2. Secondly, another structural deficiency with respect to the shear walls is their inadequacy to resist overturning moments. This deficiency relates to item number 1 above as the overturning forces are magnified due to increased forces on the walls above. The past modifications of these existing shear walls during the building remodels results in a large array of tall slender walls to resist increased forces. The guidelines of FEMA - 178 specifies a height to width ratio of no more than 4 to 1 to prevent overturning. Several of the shear walls in Lincoln Hall fail this guideline.
3. Yet another structural deficiency that exists is the lack of ductility in the building. All of the masonry shear walls are unreinforced. This increases the risk for sudden, brittle type failures. Mitigating this problem to achieve better building performance would be rather costly and difficult due to the number of walls present. The most practical way to achieve this would be to introduce a more ductile lateral resisting system.
4. Finally, another significant deficiency is that lateral forces in the east and west directions of the building are primarily resisted by the strength of four unreinforced masonry shear walls located at the corners of the building. As discussed earlier, these walls have

vertical discontinuities at the first floor level. The deficiency occurs in that the two side shear walls that support the upper walls are overstressed due to both shear and axial loads. This is occurs without the amplification increases specified by the FEMA – 178 guidelines.

In summary, the lack of adequate lateral load resisting elements is the basis for most of the structural deficiencies noted above. It is suggested that new lateral elements be added to the building to lower stresses to the existing lateral load resisting shear walls. These added elements would most likely be in the form of new concrete shear walls. Braced frames could also be utilized, though locating these would be more difficult due the amount of windows present in the building. The building floor plan has several areas where new shear walls could be added without changing the layout of the existing spaces. Additionally, as noted above in the summary, replacing the heavy hollow clay tile interior walls with lighter steel stud walls would significantly decrease the seismic design force on the building. It is expected that if this option is utilized, some new lateral load elements would still be required.

### **Non-structural Deficiencies**

1. In many areas of the building, contents could represent a very real hazard to occupants during an earthquake. Tall bookshelves, file cabinets, storage shelving, and lockers that are not secured represent falling and egress hazards. These items should be well anchored to the walls and/or floors to prevent them from overturning and/or falling.
2. Mechanical equipment, piping, ductwork, and water heaters are un-braced and/or unrestrained. Proper anchorage should be provided for larger equipment, and necessary bracing should be provided in accordance with current mechanical, electrical, and plumbing codes and current practice.
3. Fire sprinkler branch lines and other piping in several locations are not laterally braced. These piping lines should be braced such that the fire protection and other system remain operational after an earthquake.

## **ESTIMATED POTENTIAL SEISMIC UPGRADE COSTS**

The following paragraphs attempt to estimate the potential seismic upgrade costs following the guidelines outlined in the National Earthquake Hazards Reduction Program (NEHRP) Handbook for the Typical Costs for Seismic Rehabilitation of Existing Buildings – Federal Emergency Management Agency FEMA - 157 / July 1988. Both the Summary (Volume 1) and the Supporting Documentation (Volume 2) documents of the FEMA - 157 were used to establish this potential seismic upgrade cost estimate.

Typical costs associated with seismic rehabilitation of existing buildings is based on costs associated with projects that are representative of buildings constructed from a given structural material and are based on the "best available data." One of a kind projects such as the rehabilitation of a complex, historic structure, or special engineering facilities are not considered typical within the scope of this study. The seismic rehabilitation of the Lincoln Hall project is a one of a kind project as noted above. It should be clearly understood that this estimate is just that, our estimate of what the potential seismic upgrade costs could be, based upon our "best available data."

### **Direct Costs and Indirect Costs**

FEMA - 157 guideline defines Direct Costs, in a sense, as the bill received by the owner from the contractor for the work. In addition, the FEMA - 157 guideline defines Indirect Costs as societal dislocations and consequences. While indirect costs are difficult to quantify, such costs cannot be ignored. However, no indirect costs were included for the purposes of this estimate as they are outside the scope of this report.

### **Cost per Square Foot**

The most meaningful typical cost is a cost per square foot by structural type because structural type exerts the greatest direct influence on the amount of work required. Additionally, Unreinforced Masonry Buildings represent the largest building type sampled in the FEMA - 157 guideline. Also, per the guideline, a 10% premium is recommended for painting and patching. Finally, adjustments have been made for construction activities expected, number of stories of the building, building footprint, project location, date of original construction, and inflation.

Based on the above factors we estimate the potential seismic upgrade costs to be between \$15.00 and \$25.00 per square foot. With an existing building footprint of approximately 135,000 square feet. We estimate that the potential seismic upgrade costs could be on the order of \$2.0 to \$4.0 million dollars.

## CONCLUSIONS

Determining a precise quantitative level of risk for a given structure is difficult and beyond the scope of analysis and evaluation for a FEMA-178 report. However, a qualitative assessment of this building, in our opinion, places it in a moderate category of risk for considerable damage and possible collapse during a significant seismic event.

As discussed, the major deficiency in the structure is the lack of lateral resisting elements. The various alterations that have been done to this building have resulted in decreasing its strength and lateral load carrying capacity. The consequences of this carry the potential to have significant damage to the building during a serious seismic event. This concern is compounded by the potential of having large congregations of people in the auditoriums during such an event.

Many non-structural deficiencies were identified in the building, mostly with regard to the need for contents and equipment to be restrained and/or braced. While these elements in themselves can cause injury to occupants or may result in a loss of services within the building following an earthquake, they do not represent a significant threat to life safety. However, failure of non-structural elements can block means of egress and/or trap people, thereby indirectly precipitating potentially more serious consequences.