Portland State University
Millar Library
Structural Seismic Evaluation

KPFF Project No. 96541.01
Client Supplemental Contract No. 90-96-65-03

July 27, 2000

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Portland State University
Facilities Department
Post Office Box 751
Portland, Oregon 97207-0651

Prepared By:
KPFF Consulting Engineers
111 SW Fifth Avenue, Suite 2500
Portland, Oregon 97204-3628
1.0 INTRODUCTION

Located on the campus of Portland State University, Branford Price Millar Library is bound between S.W. Harrison Street and S.W. Hall Street (if they were extended) to the north and south, and between S.W. Park Avenue and S.W. 10th Avenue (if they were extended) to the east and west. Millar Library is a five story structure with two basement levels. The building serves as the University's primary student library with reading areas and some administrative office spaces.

The building was constructed in two major phases circa 1956 and 1988. The structural system of the 1966 original library is concrete waffle slab construction with concrete columns supported on spread footings. Three shear cores encompassing two stairs and an elevator bank serve as its primary lateral resisting system against wind or earthquake. The 1988 addition uses a panel joint and girder system with concrete columns also supported by spread footings. The lateral system of the addition is composed of perimeter concrete moment frames. Typically, a 2" seismic joint separates the original building from the addition. For purposes of FEMA-178 designations, the construction of the 1966 original construction would be classified as Building Type 9: Concrete Shear Walls, and the 1988 addition would be classified as Building Type 8: Concrete Moment Frames.

2.0 EXECUTIVE SUMMARY

The building's structural lateral load resisting components were evaluated to determine their capacity to resist earthquake ground motion. The general seismic evaluation was performed using the criteria of FEMA-178. The seismic evaluation of the existing structure reveals that in a major earthquake, the overall building appears to be substantially adequate to prevent collapse of the structure. Some of the shear walls and floor diaphragms could be overstressed at the lower floors and would see some damage in a major earthquake. Significant damage could occur in the moment frames for a long duration major earthquake, which would require the structure to undergo several cycles of motion. Some localized damage could also occur at the interface between the 1966 original construction and the 1988 addition. Various non-structural components could also be expected to sustain some localized damage.

3.0 SCOPE

KPFF has been retained by the State of Oregon to conduct a general structural seismic evaluation of the Portland State University, Millar Library the NEHRP - Handbook for the Seismic Evaluation of Existing Structures, FEMA-178, June 1992 was used as the basis of our assessment of this building as it relates to seismic hazards. The bibliography in Section 8.0 contains a list of resources used in developing this assessment.

Our evaluation includes a limited field reconnaissance to observe the general physical status of the building and the site, a review of available design drawings of the original structure, assessment of significant structural deficiencies observed, and discussion of structural strengthening concepts to rehabilitate serious deficiencies.

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Observations, analyses, conclusions and recommendations contained in this report reflect our best engineering judgment. Concealed problems with the construction of the buildings may exist that cannot be revealed through our review. KPFF, therefore, can in no way warrant or guarantee the condition of the existing construction of the building and the building site.

4.0 Observations

Our observations of the Millar Library are drawn from a review of available design drawings of the original construction, a walkthrough of the structure, and our experience with structures of similar construction. The following sections present our comments of our review of the available documents and site visit, respectively.

Document Review

A list of the documents reviewed is contained in the Bibliography of this report. These documents were reviewed to provide us with insight into the types of structural systems used in this structure. Our observations are presented in two different subsections. The first are observations about the general construction of the building and the second are observations pertaining to the potential seismic performance of the building.

Millar Library Construction Phases

General Structural Observations

Millar Library was constructed in two phases, representing more or less one-half of the block for each phase.

1966 Original Construction

The first phase circa 1966 was designed by architects Skidmore, Owings & Merrill and structural engineers Cooper & Rose & Associates. This five story above grade structure, with a basement and sub-basement level, comprises the western half of the current library. Its overall plan dimensions are 100' x 200'. The floor construction is a waffle slab system with 30' x 30' x 14" deep pans and a 3" slab. The typical structural grid was 27' x 27' with 3' x 3' interior columns and 9' x 9' solid heads for shear. The exterior columns of the original construction were 6'-8" x 1'-10" with...
3’ x 6’ solid heads. The lateral system is comprised of three shear wall groups enveloping the north stairwell, the south stairwell, and elevator banks to the east end of the original construction. The columns and shear walls are supported on spread footings beneath the sub-basement. The typical above grade floor to floor height is 12'-0" except at the first floor which had 17'-0" floor height. The design concrete strength (f_c) was 3000 psi for the waffle slab and substructure, and 5000 psi for the columns and shear walls. The building envelope consists of a precast concrete fins and glass cladding system. As with many other buildings on the campus of Portland State University, Millar Library was designed for future additional floors. The drawings indicate that the roof was designed as the future sixth floor.

1988 Addition

The second phase circa 1988 was also designed by architects Skidmore Owings & Merrill, and the structural engineer was Moffatt Nichol & Bonney, Inc. This five story above grade structure, with only a basement level (no sub-basement), has an overall plan dimension of 200’ x 105’, with a 62'-6" radius semicircular courtyard containing a large, historic tree and opens onto the park blocks to the east. The floor construction consists of concrete pan joists and girder system. The typical joists are 3'-0" on center with 12' pans and 4-1/2" slabs. The lateral system consists of concrete moment frames arranged in a C-shape at the north and south ends of the addition. The moment frame columns transition into concrete bearing walls at the ground floor. The bearing walls and interior columns are supported on spread footings. The floor elevations match the original construction with a 2" seismic joint separating the 1988 addition from the 1965 original construction from the second floor to the roof. The design concrete strength (f_c) was primarily 4000 psi, with 3000 psi for the substructure. The semicircular main entrance at the courtyard is clad in a glass curtain wall system, with the remainder of the addition’s exterior clad in a mix of precast concrete panels, brick, and glass. As the with the original building, the drawings indicate that the addition was designed for three future floors and a penthouse, which would be laterally resisted by steel moment frames.

Site Reconnaissance

On Thursday, October 2 1997, KPFF representatives walked through Millar Library and reviewed the general condition of the structure. As with most finished and occupied buildings, most of the structure is concealed and not accessible or visible. The primary objectives of this reconnaissance were to evaluate the structure exposed to view; to look for signs of overstress, settlement, or deterioration; and to become generally familiar with the building and its construction. Additionally, an attempt was made to verify, to the extent possible, that the construction of the building structure was consistent with the design represented on the original drawings. No attempts were made in this review to perform materials testing or exploratory demolition to evaluate the existing construction.

Our review of the building began at the roof of Millar Library. The building roof is relatively flat and possesses very little drainage slope. About an 1-1/2' of ballast covers the entire roof. To the east over the 1988 addition, a masonry parapet extends approximately 4'-4" above the roof. Only steel handrails demarcate the roof's perimeter over the 1965 original building. The elevator machine room may be accessed from the roof. The elevator machines are supported on vibration isolators which are anchored to the structured slab.

Descending to the other floors, several rows of bookshelves approximately 8' in height were arranged with reading areas generally placed around the perimeter of the building to take advantage of the natural lighting. Although the shelves appeared to be very flexible to forceful shaking by an individual, most of them were anchored to the slab.
and had an internal bracing system of steel rods. In the original section of the library, the waffle slab structure was exposed to form a soffited ceiling, whereas in the addition, gypsum drop panels hid the structure. The exposed structure appeared to be in good condition. The 2" seismic joint between the original building and addition was readily apparent by a metal strip. Library stacks and reading areas are also present in the basement.

The sub-basement primarily housed various mechanical equipment. Large networks of pipes and fluorescent lighting were generally hung directly from the exposed ceiling structure. Some pipes were laterally braced a couple places along its length. Some machinery was set on vibration isolators that were anchored to the slab. Looking up from the elevator pit, the elevator beams were connected to the shear walls. Looking up at the underside of the basement floor system where the original building and addition interface, the original construction appears to have been chipped away with the concrete for the addition poured against the roughened surface.

Walking outside along the perimeter of the building, the structure appears to be in fairly good condition with the exception of water stains and cracks in the piers and some flexural cracks in the beams. The ground floor window system also appeared to be isolated from the structure.

5.0 STRUCTURAL EVALUATION

The building structure's lateral load resisting components were evaluated to determine their capacity to resist earthquake ground motion. The general structural seismic evaluation was performed using the criteria of the FEMA-178, Handbook for the Evaluation of Existing Buildings, utilizing an effective peak ground acceleration of 0.3g.

In this document the base shear, or the total seismic force on the building, is calculated by a prescribed formula accounting for geographic seismicity, the type of building structure, its stiffness, and its overall mass. The base shear is distributed to each story based on a weighted proportion of the floor's mass and height above the ground. The structure is analyzed with the distributed story forces to determine the demand, or the strength required, of each component of the structure. The demand on each structural component is compared to the capacity of the existing structural element. For a given structural element, a demand-capacity ratio (DCR) is the demand divided by the capacity of the existing element and is a relative measure of how much, more or less, is required of the structure in its current condition. A DCR of 1.0 means the demand is equal to the element's existing capacity. A DCR of more than one means the structure is required to resist more than it is likely capable. For example a DCR of 2.0 means the element is required to resist a force twice its existing strength. A DCR of less than one means the structure has reserve capacity.

Existing Reinforced Concrete Shear Walls

The reinforced concrete shear walls are the primary lateral force resisting structural elements utilized to resist wind and earthquake in the 1966 original construction. The shear walls were generally continuous from the roof to the foundation, although in some instances walls were offset a few feet, particularly around the shear walls enveloping the stairs to the north and south. The walls were reinforced with at least 0.0025 times the gross cross sectional area of the walls. A small portion (4%) of the seismic induced forces were also assumed to be resisted by the piers.

With the discovery of higher geographic seismicity reflected in current building codes adopted by the City of Portland, higher earthquake ground motions have also increased the demand recognized for existing structures. Because the original structure was designed for at least one additional future floor, the current building inherently
exhibits greater capacity than it might otherwise have shown. As shown in Table 5.1, only at the lower levels did the demand exceed the capacity of the shear walls. All of the shear walls in the north-south direction appear to have some insufficiencies in capacity. In the east-west direction, only the demand on the shear walls at the elevator bank exceeded capacity. Some redistribution of induced forces may occur to limit extensive damage.

Table 5.1

<table>
<thead>
<tr>
<th>Level</th>
<th>North Stair N-S</th>
<th>N-S E-W</th>
<th>South Stair N-S</th>
<th>E-W</th>
<th>Elevator Bank N-S</th>
<th>E-W</th>
<th>Piers N-S</th>
<th>E-W</th>
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<td>0.7</td>
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Reinforced Concrete Moment Frames

In the 1988 addition to Millar Library, the primary lateral resisting system is composed of reinforced concrete moment frames arranged in a C-shaped pattern at the north and south ends.

1988 Addition Moment Frames

The moment frames were continuous from the roof to the ground floor and transitioned into concrete bearing walls to the foundation. Soft story effects are potentially more problematic in moment frames than shear walls because of amplification in forces due to column slenderness and P-Delta effects. For the moment frames in the 1988 addition these effects were negligible; however, the soft story did have the effect of increasing the forces at the second floor rather than the ground floor, particularly for moment frame A, as shown in Table 5.2.
Table 5.2
DCR's for 1988 Addition Moment Frames

<table>
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<th>Level</th>
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<th>Column Moment</th>
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<td>0.3</td>
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<td>1</td>
<td>0.5</td>
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</tr>
</tbody>
</table>

As with the 1966 original phase, the 1988 addition was also designed for additional floors, which is reflected in the low demand to capacity ratios. Although not particularly pressing because of the low DCR’s, several other issues concerning the detailing of these concrete moment frames should be noted. As shown in the table, the reserve capacity for shear was less than the reserve capacity for flexure. Whereas, the failure mode for shear may be brittle with a sudden loss in strength, flexural insufficiencies are ductile with lower, but sustained strength. The tie spacing at the beam and column ends and joint shear capacities are inadequate to prevent rapid deterioration of the member ends where maximum forces are present. Also shown in the table, column reserve capacity for flexure is less than the beams, particularly at the second level for frame A.

Diaphragms

Generally the concrete diaphragms are assumed to be adequate to act as rigid diaphragms to transmit story forces to the lateral resisting elements. For Millar Library this was particularly true because of the diaphragms were highly redundant, stiff, structural slabs. One area of concern, however, is the necked down portion in the 1988 addition that is bounded by the curving exterior façade on one end and an internal stair on the other. For this section the DCR is 2.5. A re-evaluation of the moment frames indicate that the frames will be adequate to resist a 30% increase in seismic forces to each frame due to the redistribution of torsional forces on the building. Significant damage to the stair should be expected should such a redistribution of forces occur.

Pounding

A 2" seismic joint is inadequate to prevent the 1988 addition from hitting or pounding against the 1966 original building. Some damage to the slab at the interface should be expected in a significant seismic event.

Foundations

Both the shear walls and moment frames have concrete spread foundations. Because the moment frames transition into bearing walls, the forces are distributed throughout the length of the bearing walls. For the shear wall foundation, no indication of the design drawings was provided on the soil bearing capacity and was therefore...
assumed to be the same as for the addition. Based on this assumption, most of the shear wall foundations had significant inadequacies. Additional settlement may occur to the foundation. Furthermore, a redistribution of forces will take place to transmit additional lateral forces back through the ground floor, basement floor, and sub-basement floor slabs.

**Strengthening Concepts and Construction Cost Estimate**

As noted above, the buildings lateral load resisting system generally possess adequate strength to meet the requirements imposed by the FEMA-178 earthquake loading criteria. The limited locations in the shear walls, moment frames, diaphragms and foundations, where overloading may occur, in themselves would not be particularly beneficial to strengthen. Substantial, life-safety performance would be expected, considering anticipated load redistribution and potential overstrength factors.

The criteria for the FEMA-178 seismic loading is, however, less than that which would be required for new construction under the current building code. If conformance with the loading criteria of the current code or some other higher seismic performance level was desired, then a general strengthening of the building would be necessary, in areas where the reserve capacity is less than approximately 18% (i.e., a DCR of greater than 0.85). A general structural strengthening concept for a building of this type would for the most part consist primarily of the addition of concrete to the existing shear walls to supplement their strength and augmenting the specific areas of the diaphragm judged to be deficient either through the addition of straps or other means of anchorage. Costs for a general seismic strengthening might be consistent with those identified by FEMA-156, while costs for a strategic strengthening would be less, corresponding to the lower overall enhancement achieved.

Construction costs to seismically strengthen a building's structural system vary significantly between specific projects and are affected by many other associated factors. These costs could be viewed in two categories: Direct and Indirect. Direct costs principally consist of the construction, materials, and labor (contractor overhead and profit included) required for the seismic strengthening and professional and permit fees. Indirect costs would include such things as financing, occupant interruptions/relocation, increased rents, change in property value, loss of revenue, administration, time value of funds, etc…. In order to identify a planning program cost for the seismic strengthening required to mitigate the earthquake hazards of this building, FEMA publication 156, *Typical Costs for Seismic Rehabilitation of Existing Buildings, December 1994*, was used.

**1966 Original Construction**

Using the procedures contained in FEMA-156 for Option Two cost estimation, the mean probable structural strengthening cost for the 1966 original construction type of building would be $18.20 per square foot. The worksheet for this cost estimation is contained in the Calculation section of this report. The cost identified herein is the Direct cost and is developed from a life-safety performance objective, in 1997 dollars considering a 4% inflation rate with a moderate range (50%) and a resulting lower to upper bound cost range of $8.92 to $37.50 per square foot.

**1988 Addition**

For building types similar to the 1988 addition, the mean probable structural strengthening cost would be $20.84 per square foot. The worksheet for this cost estimation is contained in the Calculation section of this report. The cost

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identified herein is the Direct cost and is developed from a life-safety performance objective, in 1997 dollars considering a 4% inflation rate with a moderate range (50%) and a resulting lower to upper bound cost range of $10.21 to $42.94 per square foot.

6.0 GENERAL CONCLUSIONS AND RECOMMENDATIONS

The FEMA-178 evaluation identifies that the building structure does not comply with a number of seismic evaluation criteria. The specific areas where seismic performance deficiencies were identified, are as follows:

Building Systems

- Soft story. The structure has sufficient capacity to withstand the additional induced forces from soft story effects.

- Pounding. The seismic joint is probably insufficient to prevent local damage to the floor slabs between the 1966 original construction and 1968 addition.

- Foundation. The foundation beneath the shear walls may be insufficient based upon the assumed soil bearing strength of the 1968 addition; however, the shear walls do have sufficient capacity to resist overturning forces and transmit those forces back through the floor diaphragms.

Shear Walls

- Shear stress. The shear walls of the lower floors are overstressed.

- Coupling Beams. Beams over doorways are not detailed as coupling beams; however, some additional ties and longitudinal reinforcement was provided.

Moment Frames

- Shear failures. Although the frames have less shear capacity than flexural capacity, residual capacity exists because the building was designed for higher induced seismic loads from future additional levels.

- Strong column/weak beam. In some cases the beams had more residual capacity than the columns.

- Column tie spacing. Tie spacing is greater than FEMA-178 spacing limits in some columns which may affect the anticipated ductile frame behavior.

- Stirrup spacing. Stirrup spacing over the potential plastic hinge region near the joint is greater than spacing limits which may affect the expected ductile frame behavior.

- Joint reinforcing. Although column ties extend through the beam-column joints, the joint shear capacity may be insufficient should plastic hinges occur in the beams and may affect the expected ductile frame behavior.
Diaphragms

- Openings at shear walls. Although large openings in the floor diaphragms were provided for the elevators, the diaphragms are still sufficient to transmit story forces to the shear walls.

- Plan irregularities. Although no plan irregularities exist as defined by Code, a potential plastic hinge may develop at the necked down portion of the 1998 addition. The redistribution of the additional torsional forces may be adequately resisted by the moment frames.

Non-Structural Systems

- Partitions. The partitions and fixed glazing in some areas do not appear to have been detailed to accommodate the potential interstory drifts that would occur under ground shaking.

- Ceiling systems. Although lay-in tiles and gypboard ceilings are used in the building some bracing of the ceiling system was provided.

- Lighting fixtures. The light fixtures are not independently braced from the ceiling system.

- Cladding, glazing, and veneer. The connection details of the present window system for the 1988 addition are unknown. Other glazing systems do not appear to be sufficiently isolated to accept interstory drifts.

- Building contents and furnishings. Some bookcases, filing cabinets, and other furnishings were not adequately braced.

- Elevators. The elevator control panels appeared to only be anchored at its base.