PSU-ONDINE BUILDING
1912 SW Sixth Avenue
Portland, Oregon

STRUCTURAL SEISMIC EVALUATION

FINAL REPORT
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Submitted by:
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1.0 INTRODUCTION

The PSU-Ondine Building located at 1912 SW 6th Avenue and constructed in 1966 is a 15-story residential building (1st floor commercial) with a basement. There is a 3-level parking structure, including basement, attached on the east side of the building. The 1st floor of the parking structure (middle level) was remodeled for commercial use in the early 1970’s. The top level (roof) of the parking structure matches with the 2nd floor of the tower section and the bottom level matches with the basement of the tower section.

The building is constructed of cast-in-place concrete walls, columns, beams, joists and slabs, with the main lateral load-resisting (earthquake) system comprised of reinforced concrete shear walls. This report is an evaluation of the general structural conditions of this structure for resisting lateral forces imposed by earthquake forces. For the purposes of FEMA-178 designations, this building would be classified as Building Type 9, Concrete Shear Walls.

2.0 EXECUTIVE SUMMARY

The building, excluding the attached parking structure, does not have adequate strength to resist either FEMA-178 earthquake forces (as adopted by the City of Portland) or UBC Seismic Zone 3 earthquake forces. No structural strengthening is required in the attached 3-level parking structure; however, structural earthquake strengthening measures to meet FEMA-178 requirements for the tower portion of the building would consist of:

- Concrete and reinforcement added to the shear walls.
- Concrete and reinforcement added to link beams of panel walls.
- Concrete and reinforcement added to selected columns at the ends of shear walls.
- "Drag Strut" or force collecting member added at the 4th floor for wall F3-G3.
- Steel plate or reinforced concrete added to selected areas of floor diaphragms at 2nd and 4th floors to increase diaphragm strength.
- Foundations enlarged to reduce soil bearing stresses.

The location and embedment of existing joist and slab reinforcement must be field verified to determine its ability to resist forces at joist/slab to wall interfaces. The structural evaluation calculations and recommendations contained in this report assume this reinforcement is installed in manner capable of developing the reinforcing strength.

<table>
<thead>
<tr>
<th>BUILDING CODE AND ORDINANCE COMPLIANCE SUMMARY</th>
<th>COMPLIANCE</th>
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<tbody>
<tr>
<td>1. UBC Seismic Zone 3</td>
<td>Does not comply</td>
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<tr>
<td>2. FEMA-178</td>
<td>Does not comply</td>
</tr>
<tr>
<td>3. Dangerous Building Ordinance</td>
<td>Dangerous Building</td>
</tr>
<tr>
<td>Pre - March 22, 1995 Version</td>
<td></td>
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<tr>
<td>4. Dangerous Building Ordinance</td>
<td>Not a dangerous building</td>
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<tr>
<td>Post - March 22, 1995 Version</td>
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3.0 SCOPE

KPFF has been retained by Michael & Kuhns Architects to conduct a general structural seismic evaluation of the PSU-Ondine Building. We have reviewed only the structural elements resisting lateral forces. 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 the seismic hazards. We have also compared the building to the earthquake force levels required by the Uniform Building Code, 1994 Edition (UBC 94). 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 structures, assessment of significant structural deficiencies observed, and development and evaluation of alternative structural strengthening schemes to rehabilitate serious deficiencies.

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 PSU-Ondine Building are drawn from a review of available design drawings of the original construction, a walk-through 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 to 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.

General Construction Observations

The PSU-Ondine Building is a 15-story building with a basement and a 3-level parking structure (including basement) attached on the east side of the building. The top level (roof) of the parking structure matches with the 2nd floor of the tower section and the bottom level matches with the basement of the tower section. The tower section is approximately 160,000 square feet and the parking section is approximately 40,000 square feet. The tower section up to the 4th floor has a building dimension of 184'-4" in the north - south direction and 80'-1" in the E-W direction. The E-W dimension steps back to 50'-2" at the 4th level through the roof. The attached parking structure measures 127'-6" in the north - south direction and 117'-8" in the east - west direction. The northern face of the parking structure is 14'-6" further north than the northern face of the tower section. The drawings show that all concrete elements of the building were to be constructed with 3,000 psi cast-in-place concrete and the yield strength of the reinforcing steel used was 40, 50 or 60 ksi, depending on the application. The original structural design drawings were prepared by Thomas R. Mackenzie and are dated February 10, 1966.
The interior columns, elevator shaft, and north and south stairs are supported on spread footings. The exterior walls are supported by continuous footings.

There are two types of wall classifications in the building—shear walls intended for the resistance of lateral forces and miscellaneous or "load-bearing" walls intended to support gravity forces only. The walls A1-A3, E1-E2, H1-H2, G3-F3, H2-G2, G2-F2 and H3-G3 are designated as shear walls. All of these walls extend the entire height of the building, except for G2-H2 and H3-G3 which extend from the foundation to the 2nd floor. The walls vary from 8 inches to 12 inches thick and are reinforced in both the vertical and horizontal directions. The vertical reinforcing shown on the drawings is less than the minimum required by both FEMA-178 and UBC 94.

All other walls in the structure are load-bearing walls, including all the walls in the parking section of the building. These walls are typically 8-inch thick and are reinforced in both the vertical and horizontal directions. The load-bearing walls in the parking structure and along grid line 4 (1st - 3rd) floors are substantial in size and, therefore, also act as shear walls. The drawings indicate that the wall along grid line 4 does not have the minimum reinforcing required by both FEMA-178 and UBC 94.

The columns of the building are cast-in-place concrete with vertical and tie reinforcing. The vertical reinforcing steel is spliced with mechanical splices up through the 6th floor and compression lap splices from the 7th floor through the roof. Given the age of the building, we have assumed that the mechanical splices are compression only splices; therefore, the columns do not have appreciable tensile strength.

The concrete beams in the building span between the panel walls in the north-south direction with various sizes ranging from 14" x 48" to 8" x 68". They are reinforced with continuous straight reinforcing bars.

The floor slabs of the tower section of the building are a 17-inch deep one-way concrete pan joist system with a 3-inch thick slab (14-inch pan depth). The joists are reinforced with reinforcing steel and the slabs contain 6 x 6 x 5/5 welded wire fabric (WWF). The floor diaphragm of the parking structure is a 17 1/2-inch deep two-way joist system with a 3 1/2-inch slab (14-inch pan). The system is reinforced similar to that of the tower section. Between grid lines 4 - 8 and N - P of the parking structure, the floor diaphragm is a one-way pan joist system similar to that in the tower section of the building.

Seismic Performance Observations

Lateral earthquake forces imposed on the building are resisted by the concrete shear walls. The floor diaphragms connect the shear walls and transfer forces to them. The drawings indicate that the joist reinforcement extends and hooks into the walls. This may develop the reinforcing and would assist in the transfer of forces between the floor diaphragms and walls. We have assumed that the WWF is developed into the walls although the drawings do not indicate if the WWF is hooked into the walls at end spans. Both the deformed bar and WWF reinforcing should be verified for embedment in the walls by x-ray or other testing.

The lack of shear walls on the 2nd floor along the west face of the building is defined in the code as a vertical irregularity. At this story, the forces resisted by the shear wall above the 2nd floor must be transferred through the floor diaphragm to other shear walls. This increases the load on the diaphragm and other shear walls.
There is an elevator shaft running the entire length of the shear wall F3-G3; therefore, little or no slab is attached along the shear wall. The beams framing into the wall from the North and South side are likely not strong enough to drag loads into the wall.

Site Reconnaissance

On Wednesday, February 28, 1996, KPFF representatives walked through the PSU-Cordine Building and reviewed the general physical condition of the structure. As with most finished and occupied buildings, much of the structure is concealed and not accessible or visible. The main objective of this reconnaissance was to evaluate the structure exposed to view; to look for signs of overstress, settlement, or deterioration; and 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.

The review of the visible structural elements showed no signs of overstress, settlement or deterioration. From this observation, the building can be said to be of generally good condition.

5.0 STRUCTURAL EVALUATIONS

The building structure and components were evaluated for their resistance to lateral earthquake loading. The general structural evaluation of the building components was performed using the criteria of FEMA-178 and UBC 94. The forces calculated with UBC 94 were 1.25 times the FEMA-178 forces in the north-south direction and 1.55 times the FEMA-178 forces in the east-west direction. The difference in the magnitude of the multiplier above is a result of the FEMA-178 method which calculates the forces separately for each direction, while the UBC 94 static method uses the same forces in both directions.

Shear Forces in Shear Walls

The vertical reinforcement in the shear walls is less than the minimum of 0.0025 times the cross-sectional area of the wall. Because of this, FEMA-178 (5.1.7) requires that the forces be increased by 25 percent (or the wall capacities reduced by 20 percent) for the shear force check.

The panel walls on the east and west faces of the building were analyzed as complete walls with punched window openings. The weak link in these walls were the coupling beams between the panels. The walls were found to have inadequate shear force capacity at the 11th floor and below for the FEMA-178 load level, and at the 12th floor and below for the UBC 94 load level.

All other shear walls are inadequate for FEMA-178 and UBC 94 forces for most of the height of the building. Tables 1a and 1b below show which walls are inadequate and which floors this occurs for FEMA-178 and UBC 94 criteria, respectively. A demand to capacity ratio greater than one (bold) indicates the amount the element is overstressed.

The concrete walls in the parking section have demand to capacity ratios much less than 1.00 indicating they have adequate strength to resist both FEMA-178 and UBC 94 earthquake forces.