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RE: *Structural Evaluation*
Renton Public Library

We have completed our structural evaluation of the Renton Public Library. Our evaluation is based on the original construction drawings, visual observations made during a site visit and our experience with similar structures. No sub-surface investigations to confirm conformance with the original drawings or tests to determine actual material strengths have been implemented. The purpose of our evaluation is to determine the expected seismic capabilities of the structure as well as the allowable floor load capacity for potential re-adaptation of the building.

Building Description

The library was originally designed in 1965 and is a single story structure that spans over the Cedar River. The original floor structure is constructed with precast and cast-in-place concrete elements and measures approximately 121 feet by 169 feet. The floor area was enlarged in a remodel designed in 1986 with an addition to the east that measures 18 feet by 72 feet and with the enclosure of the original porch to the north that measures 11 feet by 80 feet. The roof structure is framed with wood members and is solely supported by 28 concrete columns. A portion of the roof over the stack area occurs at a higher elevation than the surrounding roof and was originally designed to accommodate a future mezzanine.

Observations

The structure appears to be in good condition. No indications of significant foundation settlement or deterioration of the structure has been noted. Tight, horizontal cracks can be found around the perimeter of many of the columns. The cracks are consistent with flexure in the columns that may have been caused by seismic or other lateral force generating events but do not appear to represent significant damage or degradation of the structure. There are a number or repaired cracks in the masonry walls that form the administrative spaces. The walls are non-structural and are reported to have been damaged by a gas explosion in the mechanical room. Vertical gaps up to ¼" in width can be found between the concrete columns and the abutting brick walls that form the eastern addition. A wider crack can be seen in the soffit at the overhang at the southeast corner of the addition. A possible explanation for the apparent separations is described later in this report.

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Floor Capacity

Our review of the original structural drawings indicates that the floor structure is designed for a live load of 60 psf in the central reading room where the precast tee-beams span 80 feet between column gridlines, 125 psf in the stack area east of the reading room where the tee-beams span 44 feet, and 100 psf in the remaining floor and walkway areas. Our calculations suggest that the floor structure has somewhat higher capacity but a precise load calculation cannot be made without detailed information regarding the pre-stressed reinforcement used in the fabrication of the tee-beams. The structural drawings specify the location and required effective force of the reinforcement but do not indicate the mechanical properties of the reinforcement that determine the ultimate material strength. Such would be determined by the tee-beam fabricator and shown on the shop drawings which are not a part of the design drawings made available for our review. Our calculations indicate that the floor load capacity of the central reading room is at least 70 psf and the other floor areas are capable of supporting a floor load of at least 125 psf.

Seismic Evaluation

Our evaluation of the seismic capabilities of the library structure is in accordance with ASCE 31-03, the current methodology for the seismic evaluation of existing structures. A life safety (as opposed to immediate occupancy) performance level has been used in conjunction with a design level earthquake that has a 2% probability of exceedence in 50 years which is consistent with current building code seismic design criteria. Conformance to ASCE 31-03 is not required unless the building is substantially altered or subjected to a higher occupancy.

Seismic and other lateral loads imposed on the building structure are resisted at the roof level by the concrete columns that are cantilevered vertically from the floor structure. The column reinforcement does not provide sufficient ductility for the column to withstand significant overload without a loss in strength that may be sudden and complete. Current seismic design provisions therefore limit the capacity of concrete members that are not reinforced to provide ductility in comparison to previous building codes. Our calculations indicate that the potential seismic load demand is 3.0 times the lateral load that was likely used for the original design of the library columns and approximately 2.5 times the expected column capacity.

The inertial forces generated by the building in a seismic event are collected in the plywood roof diaphragm and transferred to the lateral load resisting columns by the various roof framing components and their interconnections. Our calculations indicate that the roof structure is adequate to transfer seismically induced forces to the columns with the probable exception of the transfer mechanism for forces perpendicular to the roof beams. The drawings indicate that the roof beams span north to south and are bolted to a steel saddle with anchor bolts embedded into the top of the columns. The roof diaphragm is nailed to blocking that is nailed to the roof beams which provides a direct path for lateral loads parallel to the beams. Lateral loads perpendicular to the beams are apparently transferred to the columns by the connection of the roof trusses that span between beams. The drawings indicate that the bottom chord of the roof trusses immediately adjacent to each column are connected to the underside of the beam such that the beam may transfer transverse lateral load to the column. The drawings do not contain sufficient information to evaluate the strength of the joists to transfer load from the top chord of the joist (that is connected to the roof diaphragm) to the bottom chord of the joist (that is connected to the beam) or to evaluate the strength of the connection of the bottom chord to the beam because the joist and connection components are proprietary and were designed by the joist manufacturer. In consideration of the substantial increase in seismic design load described above with respect to the columns, it is likely that the joist and connection components are similarly deficient in capacity.

The brick walls are not considered a part of the building structure in that they are not designed to support building loads. However, they may be considered a seismic hazard to the building occupants if they have insufficient strength or insufficient lateral restraint to remain stable during a seismic event. The drawings indicate that the exterior brick walls are 8" thick and are adequately reinforced vertically and horizontally to remain intact during a seismic event. Partial height walls are designed to cantilever from the floor structure and full height walls are designed to be braced at the ceiling level with an attachment to the bottom chord of the roof trusses. All of the exterior walls appear to be adequately braced with the exception of the north wall. The north wall is full height and is oriented parallel to the roof trusses. The drawings indicate that the top of the wall is attached to framing spanning between the bottom chords of the roof trusses which are laterally braced by cross bridging provided by the truss manufacturer and spaced at 8 feet. The strength of the bridging is unknown but is likely inadequate to develop the wall restraint required by current seismic standards.

The drawings indicate that the interior walls that form some of the partitions in the administrative area are 6" thick and are reinforced (with joint reinforcement) in the horizontal direction only. In most instances the interior brick walls form small rooms (such as the mechanical room) and the walls are mutually braced by virtue of the short distance between wall corners. In some instances (such as the wall between cataloging and staff) where the wall length between intersecting walls or corners exceeds about 12 feet, the walls may be considered seismically unstable and a life safety risk to the building occupants. The ceiling restraint is only of marginal benefit due to the height of the wall and lack of vertical reinforcing.

We also note that the brick walls are incompatible with the cantilevered concrete columns as the lateral force resisting system for the building. As vertical cantilevers, the concrete columns are relatively flexible and will necessarily displace laterally at the top of the column as lateral loads are applied. By comparison, the brick walls are very rigid within the plane of the wall and will displace only a small amount as lateral loads are applied parallel to the wall. The brick walls that are attached to the roof framing will therefore initially resist the lateral movement of the roof, and in a design level seismic event eventually fail, before the cantilevered columns are activated. As discussed above, failure of the brick walls presents a life safety hazard to the occupants.

Where the brick walls are attached to the concrete columns, the walls will effectively stiffen the attached column (in the direction parallel to the wall). Those columns may become over-loaded as they assume a disproportionate share of the applied lateral load which will tend to degrade the overall strength of the lateral load resisting structure at a much faster rate than if the columns shared the load more equally. The southeast column of the eastern addition is a likely example in that it is one of only four columns that are attached to brick walls in both directions. We observed more horizontal cracks in that column than any of the other columns. The observed gap between the brick walls and the column as well as the crack in the soffit could be explained as a result of the subject column assuming a disproportionate amount of lateral load during the Nisqually earthquake.

Recommendations

We suggest an investigation of the condition of the roof framing members and attachments in the immediate vicinity of the southeast column of the eastern addition. The cracked soffit and our seismic analysis suggest that structural damage may be present. It appears that the area is accessible through the suspended ceiling. Any discovered damage should be evaluated and repaired as needed.

As stated earlier in the report, conformance to ASCE 31-03 or other seismic standard is not required unless the building is substantially altered or the occupancy is increased. The building may of course be strengthened as an elective upgrade. Our suggestions to improve the seismic performance are listed below and are ordered by priority.

- Enhance the strength of the lateral force resisting system. While many solutions are feasible, we suggest strengthening the interior eight columns in the central reading room with the application of a reinforced concrete cover (or "jacket") that is 32" in diameter. The column jackets appear to be a relatively inexpensive and unobtrusive method that is compatible with the existing lateral force resisting system. The 32" diameter matches the diameter of the pier below the column and should sufficiently stiffen the jacketed columns such that the load to the remaining twenty columns will be reduced to a level within their existing capacity. The attachment of the roof framing to the jacketed columns will need to be enhanced to accommodate the increased design load.
- Provide a vertical separation between the concrete columns and any abutting masonry walls of sufficient width to allow the columns to freely sway in response to lateral roof loads.
- Enhance the attachment of the north exterior wall to the roof framing.
- Remove interior masonry walls that are not within 12 feet of an intersecting wall capable of providing adequate lateral restraint. Such walls may be replaced with wood or metal stud walls. As an alternate to removal, the masonry walls may be strengthened with wood or metal studs that are adequately attached to the roof framing to allow the wall to span vertically between floor and ceiling.

Comments

The Renton Public Library is a unique structure with unique potential for re-adaptation. All of the walls, interior and exterior, are not required for structure so may be removed as needed to accommodate any re-configuration of space, or replacement of façade. Removal of the brick walls represents an improvement in the seismic performance of the building because it removes a potential hazard and removes an incompatibility with respect to the lateral force resisting columns. Replacement of the exterior walls also provides an opportunity to improve the thermal efficiency of the building which we understand has been identified as a priority for any re-adaptation or modernization scenario.

Lightweight insulation (such as batt or foam) may be simply added at the ceiling or roof level to further improve energy efficiency without the need to strengthen the roof framing. Added insulation at the floor level is comparatively more problematic due to the exposure of the floor structure if added below, or due to the weight of an additional topping slab as well as the change in floor level if added above, but is still feasible.

A mezzanine, of light framed construction, may be easily constructed below the high roof area as originally designed.

Although the drawings used for our evaluation do not contain the information needed to determine the maximum floor load that can be supported by the tee-beams, it is possible that the needed information can be obtained. If so, a floor live load capacity of 100 psf may be demonstrable which will accommodate most public occupancies including office, retail, dining, and public assembly. If the existing tee-beam capacity is found to be less than 100 psf, or if the needed information is not found, it is possible to strengthen the floor in the area of the long span below the reading room to achieve the required capacity. Although such strengthening may be more difficult than usual due to the sensitivity of the pre-stressed tee-beams and the presence of the river below, the floor strengthening would be limited to the longer tee-beams. The shorter spans to the east and west of the reading room as well as the girders, piers, and footings that comprise the rest of the floor structure all appear to have sufficient capacity to support floor live loads up to 125 psf which will accommodate most all non-industrial occupancies.

We understand a glass floor has been suggested to enhance the view of the river below, which is perhaps the library's most defining feature. Provided the 8" wide stems of the tee-beams that are spaced 8 feet apart are preserved, the floor slab between the stems could be removed and replaced in some areas to form long panels of glass floor.

Please contact us if there are any questions with regard to our evaluation or if further services are required. We are available to investigate the condition of the roof framing at the southeast column of the eastern addition. We are also available to further develop our seismic strengthening recommendations as needed to support any program studies or construction cost estimates.

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