10.0 STRUCTURAL EVALUATION

10.1 Code Analysis -- A major renovation of Patterson Hall would require that any and all modifications to the existing structure meet the 2003 International Building Code (IBC) requirements and applicable materials codes referenced within the IBC. Applicable code requirements are listed below.

10.1.1 Snow Load -- A ground snow load of 39 pounds per square foot (psf) is required in Spokane County with a minimum roof snow load of 30 psf. Snow drifting must be considered. Building Category III and Importance Factor I = 1.1.

10.1.2 Floor Live Loads -- Various floor live load requirements based on anticipated use from similar types of facilities are listed below. Live load reductions according to the 2003 IBC will be considered where allowed.

<table>
<thead>
<tr>
<th>Category</th>
<th>Live Load (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Classrooms</td>
<td>40</td>
</tr>
<tr>
<td>Offices</td>
<td>50</td>
</tr>
<tr>
<td>Partition Loads</td>
<td>20</td>
</tr>
<tr>
<td>Assembly Areas</td>
<td>100</td>
</tr>
<tr>
<td>First Floor and Exit Floor Corridors</td>
<td>100</td>
</tr>
<tr>
<td>Non-Exit Floor Corridors</td>
<td>80</td>
</tr>
<tr>
<td>Stairs and Landings</td>
<td>100</td>
</tr>
<tr>
<td>Storage Areas</td>
<td>125</td>
</tr>
<tr>
<td>Mechanical Rooms</td>
<td>150</td>
</tr>
</tbody>
</table>

10.1.3 Wind Design -- Basic wind speed of 85 MPH, Exposure B, Building Category III, Importance Factor I = 1.15.

10.1.4 Seismic Design -- Seismic Use Group II, Importance Factor I = 1.25, Site Class D (assumed), Seismic Design Category C.

10.1.5 Frost Depth -- 24 inches.

10.1.6 Allowable Soil Pressure -- 3000 psf as noted on the existing drawings.

10.2 Existing Framing -- The existing drawings were reviewed and an analysis of key components of the structural system were analyzed for compliance with current building codes. Any deficiencies are noted in each section below.

10.2.1 Roof Framing -- The roof is framed with a 1 ½” deep 20 gage steel deck spanning between open web steel bar joists at 4'-0” on center. Steel wide flange beams framing to wide flange columns support the steel joists. Total structural depth including deck, joists and beams is 22 ½ inches. The roof surface is warped with an average roof slope of ¼” per foot and a minimum roof slope of 3/16” per foot at the north end.

Joist spans vary between 11'-0” and 23'-0”. Joist capacity is the limiting criteria for allowable roof loads. Typically the joists have the capacity to support an 18 psf dead load and the required 30 psf snow load. Dead load capacity is adequate to allow for a built up roof, rigid insulation, joist and deck weight, a layer of gypsum board, a suspended ceiling and typical mechanical ducts and electrical conduit and lights. The existing steel beams are generally adequate for the above loads. Four roof beams, between grids A and B and on grids 5, 6, 14 and 15, exceed the allowable beam capacity by three percent. These roof beams should be reinforced if any additional load is added in this area during the renovation.
Over the majority of the roof, no additional capacity is available for adding mechanical units. Additional dead load capacity is available at the joists which span 11'-0", so units could be added in those areas. Roof beams on either side of these joists do not have additional capacity and would need to be reinforced. Reinforcement of the existing joists or the addition of wide flange beams between joists would be required to support mechanical units placed on other areas of the roof. A row of joists which span 22'-0" are adequate for the total required dead load plus snow load. However, under a full snow load of 30 psf these joists exceed the optimum deflection limit of span/360 by four percent. This would only present a problem if ceiling or mechanical systems hung from these joists cannot accommodate the joist deflection. Potential methods for reinforcing steel beams are discussed in a separate section below.

10.2.2 Second and Third Floor Framing -- The floor slab consists of a 5¼” total depth, reinforced concrete slab poured over a 1¼” deep, 24 gage steel deck. The slab is a cofar slab by Granco, a proprietary slab no longer manufactured, using steel deck with reinforcing welded to the high flutes of the deck for bending capacity of the slab. Additional steel reinforcement was added near the top of the slab over each beam. Total weight of the floor, including the concrete slab, metal deck, floor finishes, fire proofing, ceiling and an allowance for mechanical and electrical systems, is estimated at 72 psf. Apparent live load capacity of this slab is 100 psf. See the Code Analysis section above for required live loads. The slab has the capacity to support typical classroom or office loads including partitions and corridor loads required by the current code. Additional capacity is not present for storage room or library floor loads. Per the Granco literature, the existing slab has a two hour fire resistance rating. The slab is supported by steel wide flange beams spanning between steel wide flange girders supported by wide flange columns. Cofar shear connector loops were welded to the steel beams and girders to develop composite action between the steel shapes and the slab. Actual percentage of composite action was not able to be determined and we assumed the beams were shored when the slab was poured. Total structural depth of the slab and beams is 26 ¼” inches.

As noted on the existing drawings and confirmed by analysis, the beams and girders were designed for a 60 psf classroom live load plus a 20 psf partition load. Live load reduction was apparently not considered in the original design. A 20 percent live load reduction is allowed for beams spanning more than 34 feet and a 36 percent live load reduction is allowed for the girders. Based on current live load reductions allowed, the beams and girders can support a total live load of 100 psf. Additional capacity is not present for storage room or library floor loads. Potential methods for reinforcing steel beams to accommodate larger floor live loads are discussed in a separate section below.

10.3 Columns -- Existing interior columns are 8” deep by 8” wide “I” shaped wide flange steel columns located at the intersection of grid lines. Welded column splices occur one inch below the top of slab at each floor. Including allowable floor live load reductions, the interior columns are adequate for a floor live load of 90 psf at each floor and a roof snow load of 30 psf. With the current floor plan configuration of a central corridor loop flanked by classrooms, columns support an average floor live load less than the 90 psf. Potential methods for increasing column strength for larger floor live loads from assembly rooms, libraries and storage rooms are discussed in a separate section below.

Exterior columns are square or rectangular shaped steel tube columns located at 11’-0” on center around the perimeter of the building. The outside face of the tube column aligns with the outside face of the exterior masonry walls. Precast column covers bolted to the steel columns conceal this feature. Steel tube beams welded to the inside face of the columns support the roof joists and the floor slab at the east and west walls of phases I and II. At the north and south walls, roof beams at 22'-0” on center are welded to columns at each grid line and floor beams are welded to every
10.0 Structural Evaluation

10.4 Walls -- Existing exterior walls are 10” reinforced brick masonry consisting of an exterior and interior wythe of brick masonry with 2¾” +/- of cast in place grout between. Vertical and horizontal reinforcement is noted on the drawings and varies from floor to floor and wall location at the perimeter of the building. The existing drawings do not indicate the type or spacing of joint reinforcement connecting the masonry wythes. Existing building specifications that may provide this information are unavailable. For our analysis, we have assumed the cast in place concrete fill provides adequate joint reinforcement to develop composite action between the exterior and interior masonry wythes.

Typically the exterior walls do not support gravity loads. Masonry wall panels 8’-4” long are centered on the exterior columns and built around the existing building frame described in the column section above. These walls span vertically 14 feet maximum from the top of the ground floor retaining wall to the second floor level, from second floor to third floor and from third floor to the roof. Reinforcement dowelled from the wall into the slab at the second and third floor slabs and embed plates at the top of the masonry wall welded to a steel angle at the roof provide a connection between the wall and the building frame to resist wind and earthquake loads. Full height windows and spandrel panels occur between each masonry wall segment eliminating the need for any brick lintels. These walls are reinforced adequately to resist current wind and earthquake loads perpendicular to the wall. Wind and earthquake forces parallel to the wall will be addressed in Lateral Systems.

Walls at the east and west stairs span between stair landings and support the roof joists. These walls are reinforced adequately to resist current wind and earthquake loads perpendicular to the wall. The wall from the second floor intermediate landing to the roof spans vertically 20 feet. Any openings cut into this portion of the wall will require additional jamb reinforcement on each side of the opening. Potential methods to add an opening in the wall are discussed in a separate section below. Wind and earthquake forces parallel to the wall will be addressed in Lateral Systems.

10.5 Foundations -- Existing foundations consist of isolated concrete pad footings at interior columns and continuous concrete foundation walls and footings at the exterior walls and interior bearing walls at the elevator, mechanical area and stairs. Interior footings are typically located 8 inches below the top of slab and exterior footings are typically located at 6 inches below the top of slab. The ground floor slab is a reinforced concrete slab placed on top of a 3” concrete waste slab. Interior pad footings can support a live load of 100 psf at each floor in addition to a 30 psf roof snow load. Live load reductions have been considered for floor live loads 100 psf or less.

The first floor is typically 4’-0” below exterior grade and 6’-6” below exterior grade at the mechanical rooms on the north side. Exterior concrete foundation walls and footings were designed as retaining walls to resist the soil placed above the ground floor. All exterior columns are embedded in the concrete retaining wall and the column base plates bear on a grout bed directly on the footings. Additional reinforcement was provided in the continuous exterior footings for 8 lineal feet at each exterior column location. Based on distributing column loads 8 feet along the footing, the calculated soil bearing pressure exceeds the allowable soil bearing pressure stated on the existing drawings by 46 percent at the south wall and 16 percent at the north wall. Assuming the wall and continuous footing are adequately reinforced to distribute the column loads the 11 feet between columns, the footings at the south wall exceed the allowable soil bearing pressure by 34 percent. Using this same analysis, the footings along the north wall do not exceed the allowable soil bearing pressure. A visual inspection of the building did not reveal cracking that would indicate settlement due to an overstressed foundation. If the loads to the exterior columns are increased, the existing
footings should be underpinned with new footings. A geotechnical engineer could also be retained to examine the existing soil below the footings to determine if a higher allowable soil bearing pressure is warranted.

Water filtration into the building from the soils below was apparently a concern in the design and construction of the existing building. Exact concerns are unknown without a geotechnical report. An issue could be a high water table but more likely is perched water on the basalt layer below the building that does not drain. An extensive foundation drain system is in place below the interior footprint of the building and around the exterior foundations. Additional measures to prevent water infiltration included a 3 inch unreinforced waste slab at the interior poured between the footings and a waterproof membrane placed over the top of the waste slab and footings. Mechanical, electrical and structural system modifications which occur below the slab will damage the waterproof membrane. A geotechnical engineer should be consulted to determine the impact of cutting the waterproof membrane.

10.6 Lateral Systems -- Lateral systems are used in buildings to resist wind and earthquake forces. As noted on the existing drawings, lateral forces are resisted by exterior reinforced brick masonry shear walls. As the original building was constructed in two phases, the reinforced brick walls between phase I and phase II were considered shear walls for this analysis. Vertical and horizontal reinforcement in the walls, as noted on the existing drawings, meet the 2003 IBC requirements for an Intermediate Reinforced Shear Wall (R=4). The existing walls are adequate to resist wind and earthquake forces required per the 2003 IBC.

Modifications to the existing building could involve removal of some of the existing walls. Walls on the east and west ends of the building and the east and west walls between phases I and II are more heavily loaded than the north and south walls. Additional capacity is available in the third floor walls, so removal of 50 percent maximum of the third floor shear wall panels is possible assuming whole panels are removed. Other walls may be strengthened by pouring a concrete wall adjacent to the existing masonry wall or by using shotcrete on the surface of the existing wall. Dowels drilled into the existing masonry wall would be required to attach the new wall.

Lateral forces are delivered to the shear walls through horizontal roof and floor diaphragms. At the roof, the steel roof deck acts as the horizontal diaphragm. The existing drawings indicate weld size and spacing of the deck to each roof joist and to perimeter roof framing members. However, the existing drawings do not indicate side lap connection type or spacing between each roof deck panel. The amount of side lap fastening between deck panels is the limiting criteria of the capacity of the roof diaphragm. Roof diaphragm values for the Granco steel roof deck are not available. A similar product by another steel deck manufacturer was reviewed. Assuming side lap connections only at each joist, the deck has adequate capacity to develop the wind and earthquake forces required in the north and south direction but not in the east and west direction. Existing side lap connections can be visually determined after removal of the existing roofing and insulation. Additional side lap connections can be made to increase the capacity of the roof deck diaphragm as required. Small new roof deck openings within the interior of the building can also be accommodated by additional deck connections if required by analysis and existing deck connection conditions.

At the perimeter of the building, the roof deck is welded to steel angles. Embed plates at the top of the masonry wall welded to these steel angles provide a connection between the wall and the building frame to resist wind and earthquake loads. Along the east and west walls, steel tubes and angles welded to the columns tie the roof system to the building frame.

The reinforced concrete slab at the second and third floors is an adequate horizontal diaphragm. Reinforcement doweled from the wall and bent into the second and third floor slabs provides a connection between the wall and the building frame to resist wind and earthquake loads. Steel tubes along the east and west ends of the building and continuous steel reinforcement at the exterior slab
edge tie the floor system together. The floor diaphragm has adequate capacity to accommodate small new openings within the interior of the building.

10.7 Precast Elements -- A precast concrete parapet surrounds the perimeter of the roof and precast concrete column covers are attached to each building column. These precast elements are reinforced. The precast parapet pieces were formed in approximately 11'-0” lengths. Two embed plates cast in each piece are welded to each building column. The precast column covers were formed in lengths which do not correspond to building finish floor heights. At the free standing columns north and south of the exterior stairs, the column covers consist of two pieces joined at the center of the column face. Column covers are bolted to each building column at a maximum spacing of 6'-0” on center with a minimum of two connections per column. There does not appear to be any connection between each precast parapet element or between the precast parapet elements and the column covers.

The majority of the precast elements appear to be structurally sound. One column cover at the northwest corner of phase II (grid F and 12) shows extensive water damage. The reinforcement is corroded and the concrete has spalled completely off one edge. The precast column cover, at the free standing column at the south west corner of phase I, has a vertical crack approximately 1'-0” long running parallel to the cover joint. At the wall to precast column and parapet interface along the north and east sides of phase II, there is some discoloration which could indicate water damage. A closer inspection would be required to determine if it was surface discoloration or if the precast was damaged.

Although connected to the building columns, the precast elements are not required for the stability of the building structure and could be removed. Another method would be required to weather proof the columns and the top of the masonry wall. If the precast elements are to remain, the damaged pieces should be repaired or replaced.

10.8 Stairs -- The stairs appear to be cast in place concrete with brick pavers. Design requirements for stairs have not changed since the original design of the building.

10.9 Consideration of Potential Future Modifications -- Based on discussions with NAC Architecture and familiarity with past higher education facilities, potential changes to the existing structure were analyzed and concerns and recommendations are presented below.

10.9.1 Removal of Existing Exterior Masonry Walls -- One option to provide flexibility of interior spaces and exterior fenestration is the removal of the existing brick masonry walls. Except for the stair wells at the east and west ends of the building, the masonry walls do not support the gravity loads of the building. We anticipate using steel studs and brick veneer at the exterior to infill between the existing floor and roof framing. The existing concrete foundation wall is 10 inches wide at the footing but reduced to 7 inches wide at the top. A brick ledger angle or a concrete ledge will be required to support the veneer. At the stair wells, a new beam and column system will be required to support the roof and floor framing.

Braced frames constructed of tube steel members placed diagonally between existing building columns from the roof to the floor and between each floor level would become the new lateral system. To minimize an increase in roof and floor diaphragm stresses, interior braced frames in the north and south direction will be required. Quantity and length of exterior braced frames will be governed by being able to adequately connect the existing columns to the existing foundation walls for horizontal and uplift forces. At least 50 percent of the exterior wall will be required to be braced frames at the first and second floors, based on frames a minimum of 22'-0” long. Interior braced frames in the east and west direction will also be required. An additional concrete wall will need to be dowelled to and poured
adjacent to the existing exterior foundation walls to provide weight to resist uplift forces. The existing elevator shaft and machine room are constructed of concrete walls. Depending on modifications required to these areas, these walls could be included as shear walls to reduce the amount of braced frames required.

10.9.2 **Providing New Openings in the Existing Masonry Walls** -- Renovations often require new openings cut into the existing walls to accommodate larger windows or new mechanical systems. Steel angles or channels would be required at the top of each opening to support the weight of the masonry above. A steel tube or channel supported at each floor and at the floor or roof above would be required at the jamb on each side of the opening. Openings cut into existing masonry shear walls that remain may be strengthened by pouring a concrete wall adjacent to the existing masonry wall or by using shotcrete on the surface of the existing wall.

10.9.3 **Infill of Existing Wall, Roof or Floor Openings** -- Changes to the mechanical system and floor plan often require existing openings in the walls, roof deck and floor slabs be filled to match the adjacent surfaces. Wall openings can be infilled with studs or with masonry construction to match the existing wall. New roof deck matching the depth of the existing deck would be used to infill existing roof openings. A new concrete slab poured on steel deck would be used to infill existing floor openings. Total slab thickness would be determined based on load and fire rating requirements. Additional beams or joists may be required to support the deck or slab depending on the existing opening size. Existing roof and floor framing adjacent to existing openings were designed for a reduced load and may require reinforcement. Potential methods for reinforcing steel beams to accommodate larger loads are discussed in a separate section below.

10.9.4 **Providing New Openings in the Existing Floor Slabs and Roof** -- Changes to the mechanical system and floor plan often require new openings in the existing roof deck and floor slabs. New openings through the roof deck or floor slab which do not cut any supporting structural members will be reinforced with supplementary steel framing placed below the metal deck and connected to the existing steel framing. Larger openings which require the removal of structural supports will require the addition of beams, columns and footings around the opening to support the remaining existing framing.

10.9.5 **Floor Infill between Phase I and Phase II** -- To increase useable square footage within the existing building footprint, new second and third floor areas may be added by enclosing the open area between phase I and phase II. New floor beams will be supported by the existing columns which are adequate for the additional floor loads. Larger footings will need to be provided below the existing columns.

10.9.6 **New Mechanical Systems** -- Current mechanical systems are located on the north side of the building in mechanical rooms at the ground floor. Separate mechanical rooms were provided for phase I and phase II. Finish floor elevation in the mechanical rooms is 2’-6” lower than the typical first floor elevation. A 7'-4” by 8'-0” mechanical shaft open through the second and third floors is located west of the west elevator. To accommodate new mechanical systems, the existing rooms can be enlarged. New concrete retaining walls and footings would be required at the room perimeter to maintain the lower finish floor elevation in the mechanical area. Relocating units to the second or third floors or to the roof will require strengthening of the existing structural system to support the new equipment. Potential methods for reinforcing existing structural framing members are discussed in a separate section below.

10.9.7 **Increasing the Capacity of the Existing Structural System** -- As discussed in previous sections, the existing roof framing is inadequate to support heavier loads from snow drifting against taller elements or from roof mounted mechanical equipment. The existing second
and third floor slabs and floor framing are not adequate to support the 100 psf exit corridor load required at the second floor or heavier loads required for assembly areas, storage rooms and libraries.

Existing structural framing can be strengthened to support heavier loads. The addition of new beams between existing joists and below an existing roof deck or an existing concrete slab on metal deck decreases the span of the deck or slab and increases the amount of load that can be supported. When beams are added between existing beams, the amount of load supported by the existing framing is reduced which results in reserve capacity in the existing member. Steel plates, steel channels and “T” shaped steel members can be welded to the bottom flange of existing wide flange beams to increase beam capacity. Steel plates provide the least additional capacity. Steel “T” shapes will provide the most additional capacity but will reduce the available clearance below the beam.

Column capacity can be increased by the addition of an adjacent column on the same footing or by welding steel cover plates to the column. New columns and footings placed below existing beams is another method used to provide additional structural capacity in a beam. Footings which require additional capacity can be underpinned with larger concrete footings. Removal of existing columns will require new columns and footings to resupport the existing beam.