STRUCTURAL EVALUATION

OF THE

EDUCATIONAL FACILITIES AT

POWELL HIGH SCHOOL

POWELL, WYOMING

WJE NO. 2002.2667

29 August 2002

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EXECUTIVE SUMMARY

Wiss, Janney, Elstner Associates Inc. (WJE) has performed structural evaluations of five buildings on the Powell High School campus located in Powell, Wyoming. The purpose of the evaluations was to identify the modifications necessary to make each building structurally sound in compliance with the current building code, and to prepare an engineer's opinion of the costs to make these modifications. The buildings included in this evaluation were the two Gymnasium buildings (the original 1948 structure and the 1968 physical education addition), the Homemaking cottage, the Natatorium/Auditorium building, and the Classroom building.

These five buildings appeared to have been built in general conformance with the construction drawings that were prepared for each structure. Our review of these drawings, together with our analyses, indicated that the structural designs of these buildings were based on the appropriate gravity loads, including dead loads, live loads and snow loads, that were specified by the codes in effect at the time. Furthermore, the design live loads and snow loads prescribed by the current building code are essentially the same as those that were required at the time that each of these buildings was constructed. Our visual observations substantiated the findings of this review in that we observed virtually no distress or evidence of inadequacy in the gravity load carrying elements of the structures.

On the other hand, our review of the drawings, together with our observations and analyses, indicated that the design and detailing of these buildings for resistance to lateral forces, especially those that could result from a seismic event, are substantially inadequate by today's standards. However, it is important to note that, while the magnitudes of the code required lateral wind forces in Powell, WY are not much different today than they were when these buildings were constructed, the magnitudes of the lateral forces required by the current building code for the seismic design of buildings in this area are significantly greater than those required by the codes in effect at the time that each of these buildings was constructed. These changes are primarily the result of a greater understanding of the earthquake characteristics and seismicity of this region together with greater knowledge of the way various structural systems and materials behave when subjected to dynamic loading, as from a seismic event. Furthermore, the current building code does not permit the use of unreinforced masonry as part of the lateral force resisting systems in buildings in the seismic zone where Powell, WY is located.

It is our opinion that these structures possess significant inadequacies in their ability to resist the code required lateral seismic forces, and if a seismic event on the order of magnitude required by the current building code were to occur, structural failures would likely result, with the potential for loss of life. Strengthening is necessary to bring these buildings into compliance with minimum standards for life-safety and into compliance with current building code requirements. Because it is our understanding that a major renovation of the buildings is anticipated, we recommend, and the City of Powell Building Department will apparently require, that structural modifications and strengthening be made to bring the buildings into compliance with the 1997 UBC.

We have estimated that the costs to bring these buildings into compliance with the structural provisions of the 1997 UBC will be between $6,700,000 and $7,700,000. A breakdown of these costs is provided in Appendix A. This engineer's opinion of costs includes the following: contractor overhead and profit; architectural and structural engineering design fees; estimated fees for testing, inspections and structural observations; contingencies to cover unforeseen conditions and design contingencies to account for the conceptual nature of these recommendations. Not included are costs associated with the following: moving, transportation and storage of furnishings and equipment; the analysis and design of building modifications or upgrades that may be necessary (or desirable) to accommodate the recommended structural retrofits; design fees associated with a major renovation of these buildings; removal and abatement of hazardous materials such as asbestos; modifications to existing foundations that could be necessary depending on the findings of an updated subsurface soil investigation; seismic strengthening of non-structural building components; other considerations as discussed in Appendix A.

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WJE NO. 2002.2667

29 AUGUST 2002

INTRODUCTION AND SCOPE

Wiss, Janney, Elstner Associates Inc. (WJE) has performed structural evaluations of five buildings on the Powell High School campus located in Powell, Wyoming. Included in these evaluations were the following buildings:

- Gymnasium buildings (the original 1948 structure and the 1968 physical education addition),
- Homemaking cottage,
- Natatorium/Auditorium building,
- Classroom building.

The purpose of the evaluations was to identify the modifications necessary to make each building structurally in compliance with current building and seismic codes, and to prepare an engineer's opinion of the costs to make these modifications.

Personnel from WJE made on-site observations of the buildings on July 29, 30, and 31, 2002. During these inspections, we made visual observations of the existing conditions in order to verify various details of the construction, to confirm that the buildings were constructed in general conformance with the available drawings, and to identify areas of existing distress that may be the result of structural deficiencies or poor detailing. Mr. John Albrecht, Coordinator of Support Services for Park County School District No. 1, assisted us in our visual observations, and in selected locations he removed existing finish materials so that we could observe concealed construction details. We also met with Mr. Eric Buchan, Chief Inspection Official for the City of Powell, to discuss building code requirements for the buildings.

As part of our evaluations, we reviewed the following documents:

1. Miscellaneous drawings for a Gymnasium, School District No. 1, Powell, Wyoming, prepared by Cushing & Terrell, Architects and Engineers, dated June 25, 1948.
2. Miscellaneous structural steel shop drawings for a Gymnasium, School District No. 1, Powell, Wyoming, prepared by Illinois Steel Bridge Company and St. Paul Structural Steel Company, dated September and October 1948.
3. Drawing Sheets 1 through 7 for Homemaking Cottage, School District No. 1, Powell, Wyoming, prepared by Tresler-McCall & Associates, Architects and Engineers, dated May 1951.
4. Architectural Drawings 1 through 22 for Auditorium - Natatorium, School District No. 1, Powell, Wyoming, prepared by Tresler & McCall A.I.A., Architects and Engineers, dated October 1954.
5. Architectural Drawings 1 through 12 and Structural Drawings 13 through 15 for Senior High School, School District No. 1, Park County, Powell, Wyoming, prepared by Cushing, Terrell and Associated Architects, dated October 15, 1958.
6. Miscellaneous Architectural and Structural Drawings for Physical Education Addition to the Gymnasium, School District No. 1, Powell, Wyoming, prepared by Cushing Terrell Associates, Architects, Engineers, Planners, dated October 1968.

7. Seismic Appraisal of the 1948 High School Gymnasium Building, Powell, Wyoming prepared by Whitten & Borges, P.C., dated April 12, 2001.
8. Repair sketches prepared by Whitten & Borges, P.C. for the Type "T3" trusses at the roof of the Homemaking Cottage, dated March 23, 2001.

DESCRIPTION OF THE STRUCTURES

Gymnasium Buildings

The gymnasium buildings were constructed in two phases. The first phase, completed in about 1948, consisted of the main gymnasium building and included locker rooms, an upper mezzanine level with movable seating, and an entrance lobby. The second phase, completed in about 1968, consisted of two adjacent buildings and was referred to on the drawings as a Physical Education Addition. This addition provided an auxiliary gymnasium in one building, and additional locker rooms, offices, multi-purpose rooms, a storage room, and a mechanical room in the adjacent two-level building.

1948 Gymnasium Building

This building has plan dimensions of about 138 ft by 141 ft. The main roof has a height above the floor of about 41 ft, the height of the roofs over the mezzanines is about 30 ft, and the lobby roof has a height of about 11 ft. The building's basic structural load carrying system consists of steel frames with brick and glazed clay tile masonry walls.

The roof structures consist of 1x wood decking over 2x12 wood joists. At the high roof, these joists bear on the top chords of steel trusses that span approximately 114 ft between the north and south walls where they are supported by steel columns encased by the masonry. At the low roofs over the mezzanines, the wood joists are supported by the bottom chord of the steel trusses that span across the high roof and by steel beams and columns encased by the east and west exterior masonry walls. At the roof over the entrance lobby, the roof joists are supported by steel beams and columns encased by the south masonry wall of the gymnasium and by the south exterior masonry wall of the lobby.

The mezzanine structures consist of hardwood flooring on 1x wood decking over 12 in. deep wood joists and steel beams. This framing is supported by steel beams and columns encased by the east and west exterior masonry walls and by the interior masonry walls that separate the gymnasium from the locker rooms that are located beneath the mezzanines.

The main level floor is a concrete slab-on-ground. In the gymnasium area, this slab is covered with hardwood flooring. The building was constructed with concrete foundation walls and continuous footings. Concrete pipe tunnels were provided near the building perimeter, apparently to facilitate the building's mechanical systems.

1968 Physical Education Addition

This facility consists of two adjoining buildings connected to the north wall of the original gymnasium structure. The first is an auxiliary gymnasium building, with plan dimensions of about 91 ft by 112 ft. The second is a two-level building, housing locker rooms at the lower level with offices, multi-purpose rooms, and storage and mechancial rooms at the upper level, with plan dimensions of about 84 ft by 116 ft. The east wall of the auxiliary gymnasium coincides with the west wall of the two-level building over part of their length, and both buildings have a common roof height of about 30 ft above the floor level. The south wall of the two-level building coincides with the north wall of the original gymnasium building, and the floor in the 1968 addition is about 2 ft lower than the floor in the 1948 gymnasium.

The roof structures for both buildings consist of plywood sheathing over 2x6 wood roof joists supported on long span steel trusses. The trusses bear on concrete beams and columns at the exterior walls. Concrete masonry unit (CMU) infill walls with brick veneer were constructed between the concrete columns along these walls. In the two-story building, the upper level floor consists of a 3 in.

thick concrete slab on metal deck supported by steel joists, beams and columns. The main level floors are concrete slabs-on-ground. Both buildings were constructed with concrete foundation walls and spread footings.

Homemaking Cottage

This building was constructed in about 1951 and is a one-story structure with a full basement, comprising about 4,000 sf on each level. Design and construction of this building utilized conventional light-frame wood construction, and it resembles a typical residential structure.

The roof structure consists of 1x wood decking supported by wood trusses. The floor framing consists of hardwood flooring over 1x decking supported by 2x12 wood joists. The exterior walls utilized 2x4 wood studs with 1x exterior sheathing and brick veneer. Foundations consist of concrete walls and footings.

Natatorium/Auditorium Building

This structure was built in about 1956. The natatorium portion of this building has plan dimensions of about 114 ft by 92 ft, and the height of the roof above the floor/pool deck is about 28 ft. This area of the building includes a swimming pool, locker rooms, and offices at the main level, with bleacher seating provided at the mezzanine.

The auditorium portion of the building has plan dimensions of about 126 ft by 206 ft. This area of the building includes the auditorium and stage, main entrance lobby, bathrooms, classrooms, band room, and offices. Roof heights and floor elevations vary throughout this part of the structure, as do the roof to floor dimensions. For example, in the stage area, the floor to roof height is approximately 48 ft; in the auditorium itself, the floor to roof heights vary from about 32 to 38 ft; and in the entrance lobby the floor to roof height is approximately 11 ft.

The structural framing is fairly consistent throughout the entire building. The basic roof structures consist of 2 ft wide channel-shaped, precast concrete planks placed side by side to form the roof decks.

Each plank appeared to have a 2 in. thick flange with 6 in. deep by 2.5 in. wide stems. The span lengths of these planks vary throughout the building. The planks are supported by CMU bearing walls except at the roof over the swimming pool and auditorium space where they are carried by precast concrete girders supported on precast concrete columns. Brick veneer clads the exterior face of the CMU walls at some areas of the building. The main level floor slabs are concrete slabs-on-ground throughout the building and the foundations consist of concrete walls and footings.

Classroom Building

This building was constructed in about 1960. The classroom portion of this building is a three-story structure with plan dimensions of about 66 ft by 322 ft at the first level and about 68 ft by 330 ft at the second and third levels. The floor-to-floor heights are 12 ft, and the height of the roof above the main level floor is about 35.5 ft.

The two remaining portions of this building are single-story structures. One houses the cafeteria and kitchen and is located on the eastside of the three-story classroom building, at the north end. The floor in this area of the building is about 1.75 ft lower than the main level floor in the classroom area, and the floor to roof height is about 14 ft. The second single-story portion of the building houses the library, boiler room, and miscellaneous classrooms, shops and offices and is located on the eastside of the three story classroom building, at the south end. Roof heights and floor elevations vary throughout this part of the structure.

In the three-story classroom areas, the structural framing system consist of steel columns and beams which support cold formed metal deck. At the second and third levels, a 3 in. thick concrete slab was cast over the metal deck to form the floor decks. In the one-story areas of the building, the roof framing consists of metal deck supported by steel bar joists. These joists are supported in some areas by steel beams and columns and in other locations by masonry bearing walls. The main level floor slabs are concrete slabs-on-ground, and the foundations consist of concrete walls and footings.

REVIEW OF APPLICABLE BUILDING CODE REQUIREMENTS

During our 31 July 2002 visit with Mr. Eric Buchan, Chief Inspection Official for the City of Powell, we were informed that the 1997 Uniform Building Code (UBC), including the following load requirements, is currently adopted by the City of Powell, Wyoming.

- Roof snow load: 30 psf.
- Wind loading: 80 mph, Exposure B.
- Seismic Zone: 2B.

Mr. Buchan informed us that all existing buildings which undergo a major or complete renovation must be brought into compliance with the 1997 UBC including the above noted loading requirements. He also indicated that prior to about 1965, the City of Powell was somewhat lenient in its enforcement of the governing building codes.

As part of our evaluation, we have performed a cursory review of the various editions of the UBC that were published around the time that each of the buildings was constructed. This research has indicated the following requirements with respect to wind and seismic loadings (lateral loads) for buildings in Powell, Wyoming:

Wind Design Loads

- Prior to 1961, the UBC required that buildings, less than 60 ft in height, be designed for a wind pressure of 15 pounds per square ft (psf). A pressure of 20 psf was required for any portion of a building greater than 60 ft above the ground.
- Changes to the wind pressure requirements were introduced in the 1961 UBC, and these remained unchanged through the 1967 edition. These editions of the UBC specified wind pressures of 20 psf for heights less than 30 ft, and 25 psf for heights between 30 ft and 50 ft.
- The 1997 edition of the UBC requires wind pressures of 13.2 psf for heights less than 15 ft, 14.3 psf for heights between 15 and 20 ft, 15.4 psf for heights between 20 and 25 ft, 16.2 psf for

heights between 25 and 30 ft, 17.9 psf for heights between 30 and 40 ft, and 20.3 psf for heights between 40 and 60 ft.

Seismic Design Loads

- Prior to the 1961 edition of the UBC, the provisions covering the design of buildings for seismic loading were contained in the code's appendices, indicating that they were not required unless specifically adopted by the governing jurisdiction. These codes placed Powell, WY in Seismic Zone 1 (minor damage) on a seismic probability map of the US. Based on these codes, the specified seismic design forces were approximately equal to about 3 percent of gravity for structures of the type constructed on the Powell High School campus.
- In the 1961 UBC, the seismic provisions were moved from the appendix to the main body of the document, indicating that compliance with these provisions was mandatory. The 1961 through 1967 codes also placed Powell, WY in Seismic Zone 1 on the same seismic probability map of the US that was published in the previous editions of the UBC. And, like the codes prior to them, the 1961 through 1967 UBC codes specified seismic design forces that were approximately equal to about 3 percent of gravity for structures of the type constructed on the Powell High School campus. In addition, these codes prohibited the use of unreinforced masonry for all elements that resisted seismic forces.
- The 1997 UBC provides requirements for the seismic design of structures in Powell, WY that are quite different than those provided in the earlier editions described above. In the current edition of the UBC, Powell, WY is in Seismic Zone 2B on the seismic zone map of the US. This updated map differs from the previous seismic maps in that it considers factors such as a site's proximity to active seismic sources, magnitude of ground motion, frequency of occurrence, and seismic probability. The placement of Powell, WY in an area of higher seismic hazard characteristics

than that required by previous editions of the UBC came about as a result of a greater understanding of the earthquake characteristics and seismicity of this region.

The 1997 UBC also requires a minimum level of both vertical and horizontal reinforcement in all masonry structures located in Seismic Zone 2B. These changes came about as a result of greater understanding of the way various structural systems and materials behave when subjected to dynamic loading, as from a seismic event. The prohibition in this and other codes against the use of unreinforced masonry in regions of significant seismic risk stems from its brittle nature and lack of ductility. Without both vertical and horizontal reinforcement, masonry elements will generally crack at relatively low levels of both stress and deformation. Once cracked, unreinforced masonry is considered unable to continue to carry load.

Using the requirements in the 1997 UBC (and assuming Soil Profile Type S_D), seismic design forces of about 9 percent of gravity would be required for the Homemaking cottage; seismic design forces approximately equal to 11 percent of gravity would be required for the three-story portion of the Classroom building; seimsic design forces approximately equal to 14 percent of gravity would be required for the 1968 Physical Education Addition; and seismic design forces approximately equal to 34 percent of gravity would be required for the unreinforced lateral load resisting masonry elements (if permitted) in the 1948 Gymnasium, the Natatorium/Auditorium building, and the one-story portions of the Classroom building.

This code also provides detailed requirements for many building components, such as diaphragms and their connections, anchorage of masonry walls, and lateral forces on elements of the structures, that were not included in earlier editions of the code.

Based on a comparison of these codes, it is clear that, while the magnitudes of the code-required lateral wind forces in Powell, WY are not much different today than they were when these buildings were constructed, the magnitudes of the lateral forces required by the current building code for the seismic design of buildings in this area are significantly greater than that required by the codes in effect at the time that each of these buildings was constructed.

The detail requirements contained in the 1997 UBC for many building components, such as diaphragms and their connections, anchorage of masonry walls, and lateral forces on elements of the structures, that were not included in earlier editions of the code, were added because of lessons learned from previous earthquakes.

REVIEW OF AVAILABLE CONSTRUCTION DRAWINGS AND OBSERVATIONS OF AS-BUILT AND EXISTING CONDITIONS

Gymnasium Buildings

1948 Gymnasium Building

The following is a summary of some of the most pertinent aspects of the drawings and our observations with regard to this building:

- The gravity-load framing systems primarily consist of unbraced structural steel frames. The lateral load resisting systems appeared to consist of masonry infill between and encasing the structural steel columns.
- No signs of distress were observed indicating inadequate gravity load carrying capacity of structural members or their connections.
- The building appeared to have been constructed in general compliance with the drawings. One exception to this was that the floor joists at the mezzanine level appeared to consist of 2x12's spaced at 16 in. on center rather than the specified 3x12's spaced at 12 in. on center.
- The multi-wythe masonry walls appeared to consist of face brick at the exterior surfaces and a combination of brick and glazed, clay tile at the interior face. With the use of an M-100 metal detector (manufactured by Fisher Research Laboratory), we found that horizontal joint

reinforcement appeared to have been installed at about 12 in. on center. Vertical reinforcement in the walls did not appear to have been specified, and none was detected with the metal detector.

- Along the east and west edges of the upper roof, the 2x12 roof joists appeared to be connected to the continuous 2x plates on the top chords of the trusses with one 16d toe-nail each. The connections of the 2x12 roof joists at the north and south edges of the upper roof to the tops of the masonry walls could not be observed, however no connection appeared to have been specified on the drawings.
- Where the lower roofs are supported along the bottom chords of the upper roof trusses, the 2x12 roof joists appeared to be connected to the continuous 2x plate with one 16d toe-nail each. Where the lower roofs are supported along the east and west exterior walls, the 2x12 roof joists appeared to be connected to the continuous 2x plate on the top of the steel beam with one 16d toe-nail each.
- Where the steel columns are encased by the exterior masonry at all four walls of the building, no mechanical connections between the columns and walls appeared to have been specified on the drawings.
- Mr. Albrecht informed us that asbestos is present in some of the construction materials used in this building.

1968 Physical Education Addition

The following is a summary of some of the most pertinent aspects of the drawings and our observations with regard to this building:

- The gravity-load framing systems for the roofs of both buildings, and the lateral load resisting systems, appeared to consist primarily of unbraced concrete frames at the exterior walls, with masonry infill between the concrete columns.

  The structural floor in the two-level building consists of a steel frame, braced by the concrete frames at the exterior walls.

- No evidence of distress was observed that would indicate inadequate gravity load carrying capacity of the structural members or their connections.
- Design notes on Sheet S-2C indicated that the governing building code was the 1967 UBC. These notes also indicated that the building was designed for a roof live load (snow) of 30 psf, and the building design was based on Seismic Zone 1, "Minor Damage."
- The building appeared to have been constructed in general compliance with the drawings. The following exceptions were noted:
  - Where the steel floor joists bear on the CMU walls, no mechanical connections between the joists and the walls were specified, however incomplete and somewhat ambiguous connection details were shown in the drawings. Our observations indicated that, in general, no mechanical connections were provided at these locations.
  - Where steel roof joists bear on concrete beams, no mechanical connections between the joists and the beams were specified, however incomplete and somewhat ambiguous connection details were shown in the drawings. Our observations indicated that in some locations connections were provided, however in other locations they were omitted.
- Along the edges of the roof framing, where the steel joists are supported on concrete beams, no connections were detailed or provided for the transfer of diaphragm shear forces into the beams and walls.
- All CMU walls were constructed using stack bond. Horizontal joint reinforcement in the CMU walls was specified at 16 in. on center, and use of a metal detector appeared to confirm this installation. Vertical reinforcement and grout in the CMU walls does not appear to have been specified, and no vertical reinforcement was detected in any of the walls with the metal detector.
- Mr. Albrecht informed us that asbestos is present in some of the construction materials used in this building.

Homemaking Cottage

The following is a summary of some of the most pertinent aspects of the drawings and our observations with regard to this building:

- Both the gravity load framing system and the lateral load resisting system consist of conventional light-frame wood construction.
- No evidence of distress was observed indicating inadequate gravity load carrying capacity of the structural members or their connections. However, we did review the above noted repair sketches prepared by Whitten & Borges, P.C. for the Type "T3" trusses at the roof.
- The building appeared to have been constructed in general compliance with the drawings. The following exceptions were noted:
  - The floor framing appeared to consist of 2x12 joists spaced at 16 in. on center rather than 2x10's spaced at 16 in. on center, as specified.
  - In one location, the anchor bolts that connect the 2x sill plate to the top of the foundation wall were found to be spaced at 48 in. on center rather than the 72 in. maximum spacing specified. However, our measurements indicated that the diameters of these anchor bolts was 1/2 in. rather than the 5/8 in. diameter specified on the plans.
- Mr. Albrecht informed us that asbestos is present in some of the construction materials used in this building.

Natatorium/Auditorium Building

The following is a summary of some of the most pertinent aspects of the drawings and our observations with regard to this building:

- The gravity load framing systems are a combination of precast concrete frames and load-bearing masonry walls. The lateral load resisting systems appeared to consist of masonry shear walls.

- No evidence of distress was observed indicating inadequate gravity load carrying capacity of the structural members or their connections.

  Observations of distress included the following

  - A diagonal, stair-step type crack was observed in the east exterior wall of the Auditorium, above the low roof over the bathrooms. The cause of this crack was not apparent.
  - Spalling of the exterior face of the CMU was observed at the north wall of the Natatorium, above the low roof over the lobby. This condition appeared to be the result of the passage of moisture through the wall, combined with freeze-thaw cycles.

- The building appeared to have been constructed in general compliance with the drawings. However, it is important to note that the structural drawings for this building were not available for our review; we reviewed only the architectural drawings.
- In general, the architectural drawings did not indicate any connections between the precast concrete roof planks and the supporting masonry walls. (The one exception to this occurs where the low roofs are supported along the east and west exterior walls, at each side of the auditorium.) Similarly, the architectural drawings did not indicate any connections from the precast roof planks to the precast concrete beams, or from the precast beams to the precast concrete columns. We were unable to observe or otherwise confirm the presence of any connections between these structural elements.
- All CMU walls were constructed using running bond. Horizontal joint reinforcement in the CMU walls was specified at 24 in. on center. However, through the use of a metal detector it appeared that the spacing of this reinforcement varied throughout the building from 24 in. to 48 in. or more. Vertical reinforcement and grout in the CMU walls does not appear to have been specified, and no vertical reinforcement was detected in any of the walls with the metal detector.

- Mr. Albrecht informed us that asbestos is present in some of the construction materials used in this building.

Classroom Building

The following is a summary of some of the most pertinent aspects of the drawings and our observations with regard to this building:

- In the three-story portion of the building, the gravity load framing system appeared to consist of unbraced structural steel frames. However, the beam-to-column connections in these frames were not designed to provide rotational restraint. As a result, the lateral load resisting system appeared to consist of the steel columns that cantilever upward from the tops of the foundations.

  In the one-story portions of the building, the gravity load framing system appeared to be a combination of unbraced steel frames and load-bearing masonry. The lateral load resisting systems in these areas appeared to consist of masonry shear walls.

- No evidence of distress was observed indicating inadequate gravity load carrying capacity of the structural members or their connections.

  Observations of distress included the following:

  - Distress and cracking in the non-load bearing masonry walls in the vicinity of the four external corners of the three-story portion of this building was observed at both the interior and exterior. This condition has existed for some time and is not believed to be related to deficiencies in the structural framing system. Rather, it is our opinion that this distress is the result of expansion of the bricks due to moisture absorption and cycles of temperature changes, combined with a lack of control joints in the masonry.

    In the vicinity of the northwest corner at the interior of the building, the expansion of the bricks along the north wall has resulted in a 1 in. gap between the edge of the concrete slab and the inside face of the west masonry wall. This condition can also be observed at the exterior side of the west wall where the brick masonry has been pushed outward with respect to the steel lintel over the windows.

- A 1 in. wide horizontal crack was observed along the top of the interior brick masonry wall, just below the ceiling, at the south side of the main entrance lobby. This wall is non-load bearing and is supported on the concrete slab-on-grade. Because the wall is not connected to the structural framing, we do not believe that the crack is the result of a structural deficiency. Rather, it is our opinion that this crack is the result of minor differential movement of the concrete slab-on-grade, upon which the wall is supported.

▪ The building appeared to have been constructed in general compliance with the drawings. The following exceptions were noted:

  - The drawings indicated that the 4 in. CMU backup for the exterior brick veneer at the three-story portion of the building was to bear on top of the structural concrete slabs at the second and third floors. However our observations indicated that the concrete slabs at these levels stop short of the walls, and the CMU passes by the edges of the slabs, with no apparent mechanical connections to the slabs.
  - The drawings specified that all steel joists were to be anchored to load bearing masonry walls with 1/2 in. diameter bolts at each bearing plate. However, our observations indicated that, in general, no steel bearing plates were provided at the joist bearings and no anchor bolts were provided between the steel joists and the tops of the masonry walls.

▪ All CMU walls were constructed using stack bond. Horizontal joint reinforcement in the CMU walls was specified at 16 in. on center, and use of a metal detector appeared to confirm this installation. Vertical reinforcement and grout in the CMU walls does not appear to have been specified, and no vertical reinforcement was detected in any of the walls with the metal detector.

- Mr. Albrecht informed us that asbestos is present in some of the construction materials used in this building.

STRUCTURAL ANALYSES, DISCUSSION AND RECOMMENDATIONS

It is our understanding that a complete renovation is planned for the existing buildings that are the subject of this study. As such, we have used the current building code requirements for the City of Powell, WY for our analyses of the existing structural systems in these buildings. Where we found that the existing structural systems do not satisfy the current building code requirements, we have proposed recommendations to up-grade these systems into compliance with this code. (Seismic strengthening recommendations are based upon the 1997 UBC, for buildings located in Seismic Zone 2B and assuming Soil Profile Type $S_{D.}$)

In general, our findings indicated that the vertical load carrying elements of the buildings were properly designed for the appropriate design gravity loads, and should continue to provide adequate and safe load carrying capacity.

However, our review of the drawings, together with our observations and analyses, indicated that the design and detailing of these buildings for resistance to lateral forces, especially those that could result from a seismic event, are substantially inadequate by today's standards. (It is possible that the structural design of some of these buildings may not have included any analytical or empirical analysis for seismic loads. But even if this is not the case, the higher lateral force level required by the current code for buildings in Powell, WY compared to older codes, together with our current understanding of material and building behavior in seismic events, would render these buildings substandard.) It is our opinion that these structures possess significant inadequacies in their ability to resist the code required lateral seismic forces, and if a seismic event on the order of magnitude required by the current building code were to occur, structural failures would likely result, with the potential for loss of life. Strengthening is necessary to bring these buildings into compliance with minimum standards for life-safety and into compliance with current building code requirements. The primary factors that have led us to this opinion are the following:

- Poor design and detailing of component connections, and lack of quality control during construction.

  In general, the construction drawings did not adequately define the connections required between the floor and roof structures and the walls. In addition, it appears that even where these details were specified on the drawings, a lack of quality control during construction led to their omission. Adequate connections of the floor and roof systems with the walls are necessary to transfer diaphragm forces and to prevent walls from collapsing during wind or seismic events.

- Code changes have placed Powell, WY in an area of higher seismic hazard characteristics.

  When these buildings were designed and constructed, the seismic design maps in the UBC placed Powell in an area of much lower seismic hazard characteristics than that indicated by current maps. This change has resulted in a significant increase in the lateral design forces required by the current building code for buildings located in Powell.

- Code changes have incorporated an increased understanding of how unreinforced masonry behaves in seismic events.

  As noted above, the 1997 UBC, and other model codes, require a minimum level of both vertical and horizontal reinforcement in all masonry structures located in regions of significant seismic risk, including Seismic Zone 2B. Where permitted, the force levels required for the design of buildings that utilize unreinforced masonry in their seismic resisting systems are on the order of 3 to 4 times higher than would be required for the same buildings that utilize reinforced masonry. This requirement stems from the brittle nature of unreinforced masonry and its lack of ductility. Without both vertical and horizontal reinforcement, masonry elements will generally crack at relatively low levels of both stress and deformation. Once cracked, unreinforced masonry is considered unable to continue to carry load. Even if the location of Powell, WY had not changed to an area of higher seismic hazard characteristics on the seismic maps, this change in the code would have resulted in design force levels that are about 3 to 4 times higher than the forces required by codes in effect at the time that these buildings were constructed.

- Code changes have significantly increased the requirements for connections of components.

  The 1997 UBC provides detailed requirements for many building components, such as diaphragms and their connections, anchorages of masonry walls, and lateral forces on elements of the structures, that were not included in earlier editions of the code. Experience has shown that the loss of connectivity between elements such as these during a seismic event can result in the loss of support and collapse of all or portions of structures.

Gymnasium Buildings

1948 Gymnasium Building

Gravity Load Analysis

An analytical check of representative framing members within this building indicated that, in general, the roof and floor framing elements appeared to have been properly engineered to satisfy the requirements of the current building code for the appropriate gravity loads, i.e. dead loads, live loads and snow loads. Our visual observations substantiated the findings of this review in that virtually no evidence of distress was observed indicating inadequate gravity load carrying capacity of the structural members or their connections.

1. However, as noted in the Observations section of this report, the floor joists at the mezzanine level appeared to consist of 2x12's spaced at 16 in. on center rather than the specified 3x12's spaced at 12 in. on center. Our analysis has indicated that this existing framing may be nominally overstressed if subjected to the design live load of 100 psf. However, the presence of no

noticeable distress in the finish materials at the ceiling below this floor, or excessive deflections in the framing, suggest that the existing joists are adequate. We recommend that the existing framing be examined in more detail when some of the repairs recommended below are executed. We believe it likely that the existing mezzanine framing will be found to be adequate.

Lateral Load Analysis

Our evaluation of the lateral load resisting elements of this building has revealed the following deficiencies with regard to current building code requirements:

1. The upper roof diaphragm is inadequate to transfer seismic loads in the east-west direction. This diaphragm consists of 1x wood decking perpendicular to the supporting 2x12 roof rafters. Allowable shear for diaphragms such as these is about 100 pounds per ft (plf) and the calculated design shear is approximately 275 plf. Strengthening of the upper roof diaphragm is recommended. We propose the installation of a new, APA rated, structural panel wood diaphragm over the existing wood decking. Installation of this framing will include connections to transfer in-plane shear forces to the perimeter walls. Execution of these repairs will require removal and replacement of the roofing.
2. The two lower roof diaphragms (over the mezzanines) also consist of 1x wood decking perpendicular to the supporting 2x12 rafters and were found to be inadequate to transfer seismic loads in both directions. The calculated design shear in these diaphragms under seismic loading is in excess of 500 plf and, as noted above, the allowable shear for diaphragms such as these is about 100 plf. Strengthening of the lower roof diaphragms is recommended. We propose the installation of horizontal steel x-bracing on the underside of the low roofs between the bottom chords of the upper roof trusses, adjacent to the roof steps, and the steel beams at the tops of the east and west walls. Installation of this framing will include connections to transfer in-plane shear forces to the perimeter walls.

3. The exterior walls of the building do not comply with the UBC requirements that masonry shear walls in Seismic Zone 2B have a minimum level of both vertical and horizontal reinforcement. In the absence of these requirements, and using an "R" value of 1.5 for unreinforced masonry, the shear stresses in the east and west walls under design seismic loading would be well in excess of allowable values. However, there are numerous window openings in these walls below the mezzanine floor level that have been enclosed with non-masonry infill. We recommend removal of the existing infill material and replacement with reinforced and grouted masonry in these openings in order to reduce the shear stresses in these walls, under design loading, to approximately half their current level.
4. As noted, the exterior walls of the building do not provide the minimum level of both vertical and horizontal reinforcement required by the UBC for masonry shear walls in Seismic Zone 2B. Because unreinforced masonry lacks ductility, failure of these walls is probable if a seismic event on the order of magnitude required by the current building code were to occur. Therefore, we recommend that a supplementary system of steel x-bracing be installed against the inside faces of the four primary gymnasium walls to provide some redundancy to the existing system. Because of the significant stiffness of the existing masonry walls, the x-bracing would not absorb the lateral loads until the masonry cracked; however the supplementary system would prevent collapse of the building. These x-bracing systems would be continuous from floor to roof. Vertical elements in these x-bracing systems would coincide with, or utilize, the existing steel columns in each of these walls. At the north and south walls, the x-bracing would be designed to accommodate the existing doorways. At the east and west walls, the x-bracing will pass between the edges of the mezzanine floor framing and the inside faces of the walls.
5. The building drawings do not appear to have specified lateral connections of the roof and floor framing to the walls, and our observations did not indicate the presence of these connections.

Connections such as these are necessary to prevent lateral, out-of-plane movement of the walls with respect to the roof and floor diaphragms, and to transfer wall loads to the diaphragms during a seismic event. We recommend that the connections of the roof and floor diaphragms to the exterior walls be improved.

1968 Physical Education Addition

Gravity Load Analysis

An analytical check of representative framing members within this building indicated that, in general, the roof and floor framing elements appeared to have been properly engineered to satisfy the requirements of the current building code for the appropriate gravity loads, i.e. dead loads, live loads and snow loads. Our visual observations substantiated the findings of this review in that virtually no evidence of distress was observed indicating inadequate gravity load carrying capacity of the structural members or their connections.

1. One exception to this was the concrete frames at the exterior walls. Our analysis has indicated that, by themselves, the concrete beams along the tops of these frames do not possess adequate strength to support the roof loads, and are dependent on the masonry infill below to provide the required gravity load carrying capacity. Consequently, it would be necessary for the existing roof framing members to be temporarily shored if the existing masonry infill walls were ever to be removed.

Lateral Load Analysis

Our evaluation of the lateral load resisting elements of this building has revealed the following deficiencies with regard to current building code requirements:

1. The roof diaphragm over the two-level building is inadequate to transfer seismic loads in the east-west direction. This diaphragm consists of 5/8 in. plywood with 8d common nails spaced at 6 in. on center at panel perimeters and 12 in. on center at intermediate supports. Allowable shear for

diaphragms such as these is about 270 plf and the calculated design shear is approximately 470 plf. Strengthening of this roof diaphragm is recommended. We propose the installation of a new, Structural I, wood panel diaphragm over the existing framing. Installation of this new diaphragm will include connections to transfer in-plane shear forces to the perimeter walls. Execution of these repairs will require removal and replacement of the roofing.

2. Two drag struts (collector elements) are required in the roof diaphragm over the two-level building to transfer loads into the east and south walls of the auxiliary gymnasium building. Drag struts were not installed in the original construction. One of these drag struts will be located at the juncture of the roof over the auxiliary gymnasium with the roof over the two-level building and will transfer diaphragm loads from both roofs into the east wall of the auxiliary gymnasium. The second drag strut will be located in the roof over the two-level building and will be aligned with the south wall of the auxiliary gym to transfer diaphragm loads into this wall. Removal of the roofing and existing plywood sheathing over a portion of the auxiliary gymnasium building will be necessary to facilitate installation of these struts.
3. At the upper-level floor in the two-story building, two drag struts are also required to transfer loads into the east and south walls of the auxiliary gymnasium building. These drag struts did not appear to have been installed in the original construction. These drag struts will be aligned with the two drag struts described above in the roof framing.
4. The concrete frames at the exterior walls of these buildings have insufficient capacity to transfer the lateral loads from the roofs (and upper level floor at the two-story building) into the foundations. The CMU infill between the columns in these buildings does not comply with the UBC requirement that masonry shear walls in Seismic Zone 2B provide a minimum level of both vertical and horizontal reinforcement. Also, this masonry lacks sufficient horizontal reinforcement to span between concrete columns to support its own self-weight, and the weight of

the attached brick veneer, in a seismic event. Therefore, it is our recommendation that new steel x-bracing be provided between the existing concrete columns to carry the lateral forces resulting from the code required wind and seismic events. This x-bracing can be installed immediately adjacent to the inside faces of the existing CMU masonry infill between concrete columns. The existing masonry should be connected to the new horizontal steel members in the x-bracing systems and spaced at about 8 ft on center vertically.

5. The building drawings were vague with respect to required connections of the roof and floor joists to the exterior concrete beams and masonry walls, and our observations indicated that in many cases no connections between these members were provided. Connections such as these are necessary to prevent lateral, out-of-plane movements of the walls with respect to the roof and floor diaphragms, and to transfer wall loads to the diaphragms during a seismic event. Also, connections between the wood roof diaphragms and the concrete columns and masonry walls, to prevent out-of-plane movement of the walls (along the east and west sides of the auxiliary gym, and along the north side of the two-level building), were neither specified nor provided. We recommend that all connections of the roof and floor diaphragms to the exterior walls be improved.

Homemaking Cottage

Gravity Load Analysis

An analytical check of representative framing members within this building indicated that, in general, the roof and floor framing elements appeared to have been properly engineered to satisfy the requirements of the current building code for the appropriate gravity loads, i.e. dead loads, live loads and snow loads. Our visual observations substantiated the findings of this review in that virtually no evidence of distress was observed indicating inadequate gravity load carrying capacity of the structural members or their connections.

1. However, as noted in the above Review of Available Construction Drawings section of this report, Whitten & Borges, P.C. has indicated that the Type "T3" trusses in the roof framing are inadequate for the required snow loads and should be strengthened. Our analysis of these trusses has confirmed the conclusion reached by Whitten & Borges, and it is our recommendation that these trusses be strengthened in conformance with their sketches.

Lateral Load Analysis

Our evaluation of the lateral load resisting elements of this building has revealed the following deficiencies with regard to current building code requirements:

1. The existing 1x wood decking on the roof is inadequate to provide the necessary diaphragm strength. The allowable shear for diaphragms such as this is about 100 plf, and the calculated design shears are as high as about 250 plf. Strengthening of the roof diaphragm is recommended. We propose the installation of new, APA rated structural wood panel sheathing over the existing wood decking. Execution of these repairs will require removal and replacement of the roofing.
2. The existing 1x wood sheathing on the exterior walls is inadequate to satisfy the lateral force demand required of the exterior shear walls in the north-south direction. The calculated design shear in these walls is about 150 plf, and the allowable shear is about 100 plf. Strengthening of the shear walls is recommended. We propose the installation of new, APA rated structural wood panel sheathing on either the inside or outside faces of the east and west exterior walls.
3. Use of the primary interior partitions at the main level is necessary to provide additional shear wall resistance. We recommend that connections be provided from the tops of these walls to the roof trusses that are capable of transferring loads from the roof diaphragm into these walls.

Natatorium/Auditorium Building

As noted above, this building appeared to have been constructed in general compliance with the drawings, however, the structural drawings for this building were not available for our review; we reviewed only the architectural drawings. These architectural drawings provided almost no indication of the connections that were to be made between the precast concrete roof planks and the supporting masonry walls and beams, or between the precast beams and the precast concrete columns. During our investigative work, we were unable to observe or otherwise confirm the presence of any connections between these structural elements. Similarly, the architectural drawings did not indicate the presence of either horizontal reinforcement (other than joint reinforcement) or vertical reinforcement in the CMU walls, and we did not detect any reinforcement such as through the use of a metal detector. (The thickness of some of the masonry walls, between 12 and 18 in., may have precluded our being able to detect reinforcement that may be centered in their thickness.) Based on these findings, we believe it probable that no connections (or nominal connections at best) exist between the structural elements noted above, and that, except for horizontal reinforcing in some bond beams at bearing elevations, no horizontal or vertical reinforcement was provided in the masonry walls.

Gravity Load Analysis

An analytical check of representative framing members within this building indicated that, in general, the roof and floor framing elements appeared to have been properly engineered to satisfy the requirements of the current building code for the appropriate gravity loads, i.e. dead loads, live loads and snow loads. Our visual observations substantiated the findings of this review in that virtually no evidence of distress was observed indicating inadequate gravity load carrying capacity of the structural members or their connections.

Lateral Load Analysis

Our evaluation of the lateral load resisting elements of this building has revealed the following deficiencies with regard to current building code requirements:

1. The precast concrete roof deck over the natatorium, is incapable of acting as a diaphragm if adequate connections between adjacent roof planks were not provided. Removal of the roofing in

selected locations should be performed to determine if connections between planks were installed as part of the original construction. Based on the current information available, that no connections were provided, we recommend that a 2 in. concrete topping be provided over the top of the existing precast planks to enhance the diaphragm. New connections between the roof deck and the perimeter concrete beams will be installed with this new topping.

2. The concrete frames at the north, east, south, and west exterior walls of the natatorium area of this building are inadequate to transfer the lateral loads from the roof into the foundations. Similarly, the masonry infill between the columns along walls is incapable of providing adequate shear resistance. Furthermore, these walls do not satisfy the UBC requirements for masonry shear wall construction in Seismic Zone 2. Therefore, it is our recommendation that the existing masonry between the concrete columns in the east, south, and west walls be removed and replaced with properly designed masonry shear walls. At the north wall, the existing masonry can only be removed above the low roof over the building lobby because the masonry below this roof is load-bearing. In these areas, we propose the installation of fiber reinforced composites on both faces of the existing load-bearing masonry to provide the needed strengthening.
3. The precast concrete roof deck over the auditorium area and stage are incapable of acting as a diaphragm if adequate connections between adjacent roof planks were not provided. Removal of the roofing in selected locations should be performed to determine if connections between planks were installed as part of the original construction. Based on the current information available, that no connections were provided, we recommend that a 2 in. concrete topping be provided over the top of the existing precast planks to enhance the diaphragm. New connections between the roof deck and the perimeter walls will be installed with this new topping.
4. The tall masonry walls at the east, south and west sides of the auditorium and the west, north, and east sides of the stage are incapable of acting as adequate shear walls to transfer the lateral loads

from the roofs into the foundations. Furthermore, these walls do not satisfy the 1997 UBC requirements for masonry shear wall construction in Seismic Zone 2B. Therefore, it is our recommendation that the existing masonry walls be strengthened. Vertical reinforcement can be installed to provide the strength necessary for these walls to span between roof and floor levels. Fiber reinforced composites can be installed on the exterior face of these walls to provide the needed strengthening to resist lateral loads from the roof diaphragms.

5. The two 15 ft long masonry walls that separate the stage from the auditorium are incapable of acting as adequate shear walls to transfer the lateral loads from the auditorium and stage roofs into the foundations. Furthermore, these walls do not satisfy the 1997 UBC requirements for masonry shear wall construction in Seismic Zone 2B. Therefore, it is our recommendation that new concrete shear walls, connected by a concrete beam near the level of the auditorium roof, be constructed next to the existing masonry walls on the auditorium side. The construction of new foundations to support these new walls will be necessary.
6. The concrete frames at the north, east, west exterior walls of the band room at the north end of this building are inadequate to transfer the lateral loads from the roof into the foundations. Similarly, the masonry infill between the columns at the east and west sides of this room are incapable of acting as suitable shear walls. Furthermore, these walls do not satisfy the UBC requirements for masonry shear wall construction in Seismic Zone 2. Therefore, it is our recommendation that the existing masonry between the concrete columns at the east and west walls be removed and replaced with properly designed masonry shear walls. At the north wall, we recommend that steel x-bracing be installed between the existing columns (to maintain use of the windows) to provide the necessary resistance to lateral loads.
7. At the low roof areas, along the east and west sides of the auditorium, there are an inadequate number of shear walls in the east-west direction. Therefore, we have recommended that several

of the existing interior masonry partitions be removed and replaced with properly designed masonry shear walls. Similarly, at the exterior walls in these areas of the building, the existing masonry is incapable of providing the necessary shear resistance. Furthermore, these existing walls do not satisfy the 1997 UBC requirements for masonry shear wall construction in Seismic Zone 2B. Therefore, it is our recommendation that the existing masonry walls in these areas be removed and replaced with properly designed masonry shear walls. Along the east side of the building, no full height exterior masonry walls currently exist, and so it is our recommendation that the east exterior wall along the bathrooms be removed and replaced as full height masonry.

Classroom Building

Gravity Load Analysis

An analytical check of representative framing members within this building indicated that, in general, the roof and floor framing elements appeared to have been properly engineered to satisfy the requirements of the current building code for the appropriate gravity loads, i.e. dead loads, live loads and snow loads. Our visual observations substantiated the findings of this review in that virtually no evidence of distress was observed indicating inadequate gravity load carrying capacity of the structural members or their connections.

1. As noted above, cracking has occurred in the non-load bearing masonry walls in the vicinity of the four corners of the three-story portion of the building. This condition was the subject of an investigation by WJE in 1991, and a copy of the report that documented our findings from that study is included in Appendix B. This condition has apparently existed for some time and is not believed to be related to deficiencies in the structural framing system. Rather, it is our opinion that this distress is the result of expansion of the masonry due to moisture absorption and cycles of temperature changes, combined with a complete lack of control joints.

It is our recommendation that five vertical control joints be provided in the existing masonry on the east and west elevations of the building, and that at least two vertical control joints be provided in the masonry at the north and south elevations. We also recommend that the existing masonry be removed and replaced in the vicinity of the four corners of the building where it has been pushed away from the floor slabs as a result of the expansion of the brick and block. The extent of this work should be sufficient to include the areas where the masonry is distressed and where any outward movement of the brick has occurred.

2. The horizontal crack along the top of the interior brick masonry wall, just below the ceiling, at the south side of the main entrance lobby is not believed to be the result of a structural framing deficiency because the wall does not appear to be connected to the structure. The crack appeared to disappear where the wall meets the glazing adjacent to the west entrance; at this location, the brick wall is supported on a concrete foundation. Therefore, it is our opinion that the crack is the result of minor differential movement of the concrete slab-on-grade, upon which the wall is supported. We recommend that a brick mason remove and reconstruct the top portion of the wall in the vicinity of the crack.

<u>Lateral Load Analysis</u>

Our evaluation of the lateral load resisting elements of this building has revealed the following deficiencies with regard to current building code requirements:

1. In the three-story portion of the building, the resistance to lateral loads appears to rely solely on the strength and stiffness of the steel columns to cantilever from the foundation to the roof. No bracing was indicated on the plans and none was observed. Furthermore, the beam to column connections were not designed to provide moment resistance and are capable of offering only very nominal rotational restraint. Our calculations have indicated that the cantilever columns are moderately overstressed when the building is subjected to the code required wind pressures, but

that they are grossly overstressed when subjected to the code required lateral seismic forces. Therefore, it is our recommendation that steel x-bracing be installed at various locations throughout the three levels of this building to provide an adequate lateral load resisting system in both the north-south and east-west directions of the building. The actual locations of the x-bracing are beyond the scope of this evaluation, but should coincide with the existing column and beam lines for structural economy and efficiency. Installation of the x-bracing along these lines will require removal and replacement of many of the existing masonry partition walls, and in other locations will require removal of existing lockers or windows. It is also likely that in some areas the x-bracing will need to be installed in what is currently an open room, thereby necessitating revisions to the layout of some rooms and door locations.

2. In the one-story portions of the building, resistance to lateral loads appears to rely primarily on the unreinforced masonry walls. These walls are incapable of acting as adequate shear walls to transfer the lateral loads from the roofs into the foundations. Furthermore, these walls do not satisfy the 1997 UBC requirements for masonry shear wall construction in Seismic Zone 2B. Therefore, it is our recommendation that some of the existing masonry walls in these areas be removed and replaced with properly designed masonry shear walls. In other locations, the installation of steel x-bracing between the existing steel columns appeared more appropriate and is, therefore, recommended. Not all of the existing masonry walls will need to be removed and replaced as shear walls. However, where these remaining walls are load-bearing, we recommend that vertical reinforcement be installed to provide the strength necessary for these walls to span between the roofs and floors so as to prevent their collapse. New connections between the roof deck/roof joists and the perimeter walls will also be required.
3. At the three-story portion of the building, the exterior masonry walls are not mechanically connected to floor slabs and roof deck. This condition represents a significant hazard in that

during a seismic event, the masonry could fall off the face of the building. Therefore, it is our recommendation that new connections be provided around the perimeter of this building at the second and third floors and the roof to prevent collapse of this masonry.

APPENDICES

ENGINEERS
ARCHITECTS
MATERIALS SCIENTISTS

Appendix A:

Engineer's Opinion of Cost

ENGINEERS
ARCHITECTS
MATERIALS SCIENTISTS

Engineer's Opinion of Cost

A conceptual estimate of the costs required to bring each of the buildings into compliance with the structural provisions of the 1997 UBC has been prepared. The designs and techniques employed in the structural retrofits, as well as the restoration of interior and exterior finishes required to be removed to accommodate the retrofits, are conceptual in nature. The ultimate costs of the retrofits will be influenced significantly by the actual structural designs and choices made in the quality and style of finishes that will need to be replaced or patched. It must be anticipated that in many instances it will not be feasible to provide an exact match for existing finishes. In some locations, the retrofits will require the relocation of doors, millwork and other fixtures, whereas room sizes, layouts, and usage may need to be altered in other locations. While some assumptions have been made in the estimate, they remain conceptual in nature and may not reflect the final designs.

The estimated costs provided should only be used for budgetary purposes. Contractor overhead and profit has been included. Architectural and structural engineering design fees, plus costs for testing, inspections and structural observations, have been included as a percentage of the estimated construction costs. Contingencies have been applied to cover unforeseen conditions typically discovered in retrofit projects of this size and nature. To allow for the conceptual status of the project at this time, a design contingency has also been applied.

Costs have not been included for the moving, transportation, or storage of furnishings and equipment, the removal or abatement of hazardous materials such as asbestos, modifications to existing foundations that could be necessary depending upon the findings of an updated subsurface soil investigation, or seismic strengthening of non-structural building components and equipment. We have also assumed that there will be no remodeling or refinishing of building areas outside of the immediate locations where the recommended repairs will be made. Finally, we have assumed that the buildings will not be occupied when the work is being performed, that all the work will be performed under one contract, and that temporary heat, water, electricity and toilet facilities needed during construction will be provided by the owner.

Based upon the above described scope, we anticipate the costs to retrofit the five structures to be between $6,700,000 and $7,700,000. A breakdown of these costs follows.

Powell Wyoming School District
Seismic Retrofit
WJE # 2002.2667

CONCEPTUAL COST ESTIMATE

		Classroom Building	Homemaking Cottage	Old Gym	Physical Education Addition	Natatorium - Auditorium	TOTAL
		103,000 SF	4,800 SF	17,000 SF	27,000 SF	35,000 SF	186,800 SF
Cost		$ 1,900,000	$ 70,000	$ 390,000	$ 540,000	$ 1,235,000	$ 4,135,000
G.C. O&P	20%	$ 380,000	$ 14,000	$ 78,000	$ 108,000	$ 247,000	$ 827,000
Construction Cost		$ 2,280,000	$ 84,000	$ 468,000	$ 648,000	$ 1,482,000	$ 4,962,000
Cost per SF		$22.00	$17.50	$27.50	$24.00	$42.25	$26.50
Engineering Fees	10%	$ 228,000	$ 8,400	$ 46,800	$ 64,800	$ 148,200	$ 496,200
Testing & Inspection	3%	$ 68,400	$ 2,520	$ 14,040	$ 19,440	$ 44,460	$ 148,860
Architect Fees	5%	$ 114,000	$ 4,200	$ 23,400	$ 32,400	$ 74,100	$ 248,100
Subtotal		$ 2,690,422	$ 99,138	$ 552,268	$ 764,664	$ 1,748,802	$ 5,855,293
Contingency -Unforeseen Conditions	15%	$ 403,563	$ 14,871	$ 82,840	$ 114,700	$ 262,320	$ 878,294
Subtotal		$ 3,093,985	$ 114,008	$ 635,108	$ 879,364	$ 2,011,123	$ 6,733,587
Design Contingency Conceptual - Schematic	15%	$ 464,098	$ 17,101	$ 95,266	$ 131,905	$ 301,668	$ 1,010,038
TOTAL		$ 3,558,083	$ 131,109	$ 730,374	$ 1,011,268	$ 2,312,791	$ 7,743,625
USE		$ 3,560,000	$ 130,000	$ 730,000	$ 1,010,000	$ 2,310,000	$ 7,740,000
Cost per SF		$34.50	$27.00	$43.00	$37.50	$66.00	$41.00