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Wells High School Analysis and Reconstruction Following the February 21, 2008 Earthquake

by

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ABSTRACT

On February 21, 2008 a 6.0 seismic event occurred 6 miles north of the City of Wells, Nevada. The Wells High School experienced significant damage as a result of the earthquake and aftershocks. The school was designed and constructed in 1951.The lateral load-resisting system for the longitudinal direction (N-S) consists of reinforced concrete frames with unreinforced concrete and brick masonry (URM) in-fill walls. For the transverse direction, 13- and 17-inch-thick URM masonry shear walls were used. Initial inspection revealed some damage of the URM in the high roof areas of the gymnasium and auditorium. Demolition for the repair of these areas revealed major internal vertical fracturing and separation of the multi-wythe masonry creating extremely high and unacceptable height-to-width ratios and potential instability of the walls. The low-roof classroom areas constructed during the same period experienced minor cracking along mortar lines that could be repaired by simple repointing of the mortar joints. Additions to the school, built in 1961, 1989, and 2000 and designed under later versions of the UBC were constructed using reinforced masonry. These newer additions performed well and sustained only minor cracking.

INTRODUCTION

On the morning of February 21, 2008, the area of Wells, Nevada was subjected to a magnitude 6 seismic event that was located approximately 6 miles to the north of the Wells High School. Following the main event, the area experienced several aftershocks that continued to increase the damage to the Wells High School building that had been weakened by the original earthquake.

HISTORY OF CONSTRUCTION AND ADDITIONS TO THE WELLS HIGH SCHOOL

The Wells High School was designed and constructed in 1951. The new high school included a gymnasium, an auditorium with a stage facility, eight classrooms, and administrative offices. Heating facilities were located in a basement boiler room below a portion of the auditorium. See figure 1 for plan view of the school building and additions to the school. See figure 2, Wells High School from northeast side.

In 1961 a library and three classrooms were designed and constructed as a wing that protruded to the west of the original building. A third addition was designed and constructed in 2000 that included a new library, additional boys and girls restrooms, and the conversion of the original library into a classroom. This addition was an extension of the 1961 west wing. Other minor additions were constructed in 1989 that included the expansion of the original boys and girls locker rooms on the east and west sides of the gymnasium.

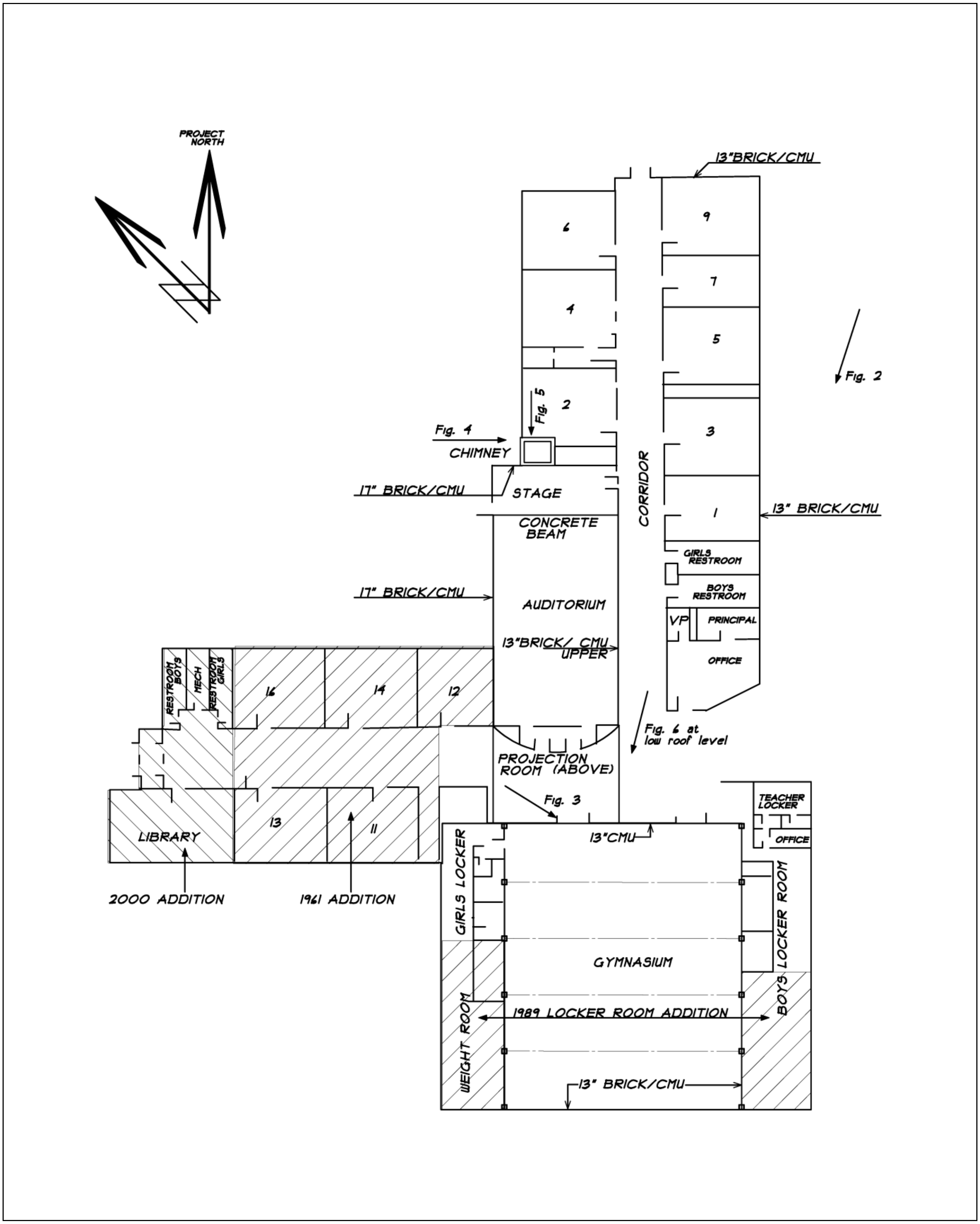

Figure 1. Wells High School floor plan (not to scale).

Figure 2. Wells High School, February 21, 2008.

TYPE OF CONSTRUCTION

The original building, designed in 1951, consisted of three similar building types each utilizing reinforced concrete frames along the east and west walls that carried the majority of the gravity loads and provided the lateral resistance for seismic and wind loads in the north-south direction. These concrete building frames were in-filled with unreinforced masonry (URM) walls. The north and south end walls were constructed of URM that served as bearing walls carrying minor gravity loads from the roof structure and also provided the lateral rigidity to resist the lateral wind and seismic forces in the east-west direction (see figure 1–Floor Plan).

While similar, each of the three buildings did have the following differences:

The gymnasium

This portion of the building, with an eave height of about 22-feet high, has a barrel-shaped roof with a maximum height at the center of 33 feet above the floor. The roof structure consists of steel “bow-string” trusses spanning 93 feet that are spaced at 20-foot intervals on center. The roof sheathing consists of 1” nominal lumber on 2”x12” rafters that are supported on the trusses and north and south end-walls.

The concrete frames located in the east and west walls consisted of 16- by 16-inch square columns spaced at 20-feet on center. At the top of these columns, 22 feet above the floor, a concrete beam along each wall ties these columns together. A second beam is located 12 feet above the floor at the low-roof level of the locker rooms. This beam exists only in the bays of the frames adjacent to the original locker rooms or in two bays on the west side and three bays on the east side.

The locker rooms, located along the east and west walls of the gymnasium were of similar construction, except they have a low (12-foot high) roof system that consists of a 2- to 2.5-inch- thick concrete deck on steel bar joists. These bar joists span east-west between reinforced concrete frames at the exterior wall of the locker room and on the lower beam of the concrete frame of the high walls in the gymnasium

A concrete tie beam 13- by 18-inches deep, located in both the north and south end walls of the gymnasium at the 22-foot level, ties the corner columns of the gymnasium. The weight of this 93-foot-long concrete beam is supported by the URM wall located below the beam.

The auditorium

The auditorium, located immediately north of the gymnasium, shares a common wall with the gymnasium. The roof system over the audience area of the auditorium is a 2- to 2.5-inch-thick concrete deck on steel bar joists. Steel trusses spaced at about 20 feet span the width of the room. The steel bar joists are supported by the steel trusses, by the gymnasium/auditorium common wall, and by a concrete beam at the stage roof. The roof over the stage area at the north end of the auditorium was constructed using a reinforced concrete slab-and-beam system.

The floor of a projection room, about 12-feet high and located at the south end of the auditorium, is also a reinforced concrete slab-and-beam system.

The roof of the auditorium, for acoustical purposes, slopes upward 4 feet toward the stage, or the north wall, which is approximately 28 feet high and is constructed of unreinforced masonry. This wall is common with the south wall of the large chimney structure that serves the boiler room. The outside dimensions of the chimney are 85 inches by 90 inches and the height was approximately 64 feet above finish grade or about 80 feet above the floor of the basement boiler room. Three walls of the chimney are 14 inches thick and the fourth wall, the wall common to the auditorium, is 17 inches thick.

Classroom and administrative offices are located along the east side of the auditorium. For structural review and analysis this low roof portion of the school was considered a part of the auditorium. A hallway that serves the classrooms is located along the east wall of the auditorium. This low, 12-foot-high portion of the building was also constructed with a thin concrete slab roof on steel bar joists. These bar joists are supported on concrete frames located at the east wall of the auditorium, the east wall of the hallway, and the east exterior wall of the classroom and offices.

North classroom wing

A low classroom building is located to the north of the auditorium and is separated from the auditorium by an expansion joint that isolates the roof, walls, and concrete frames from the auditorium and classroom section described above. The construction of this section containing the remaining 7 classrooms also utilizes a thin concrete slab roof on steel bar joists that span between the concrete frames located in the east and west exterior walls of the classrooms and along each side of the hallway that serves these classrooms.

Unreinforced masonry construction

The unreinforced masonry walls were typically 13 or 17 inches thick and were constructed of either concrete masonry units (CMU) or clay brick. The CMU walls were, in general, not grouted with the exception of the cells that contained anchor bolts or an occasional 3/8” diameter vertical steel bar. These walls, being multi-wythe walls, were constructed as follows:

A “13-inch” wall consisting of 8-inch wide CMU and 4-inch wide clay brick was constructed with the CMU on the inside face and the clay brick veneer on the exterior face. At every 6 or 7th brick course the clay brick was turned 90-degrees to act as a “header-course” to tie the two wythes of the wall together. A 1-inch vertical mortar or collar joint separated the two wythes (see figure 7 – 13-inch brick/CMU wall).

A “13-inch” wall consisting of 8-inch and 4-inch wide CMU was used where the aesthetic qualities of the clay brick were not required. As the courses were stacked vertically up the wall the 4-inch wide unit was laid alternately between the inside face and the outside face of the wall, thus creating a 4-inch overlap of the CMU block at each course. Again a 1-inch mortar or collar joint separated the two wythes (see figure 8 –13-inch CMU wall).

The only “17-inch” walls are located at the west and the north end walls of the auditorium. These 28-foot-high walls were constructed using 8-inch and 4-inch concrete masonry units and an exterior course of clay brick veneer. The masonry units were staggered similar to that described above for the 13-inch CMU wall, however, at every 6th or 7th brick course, two clay bricks were turned 90 degrees and laid end–to-end to provide a 17-inch interlocking course for the exterior and interior units (see figure 9 – 17-inch brick/CMU wall).

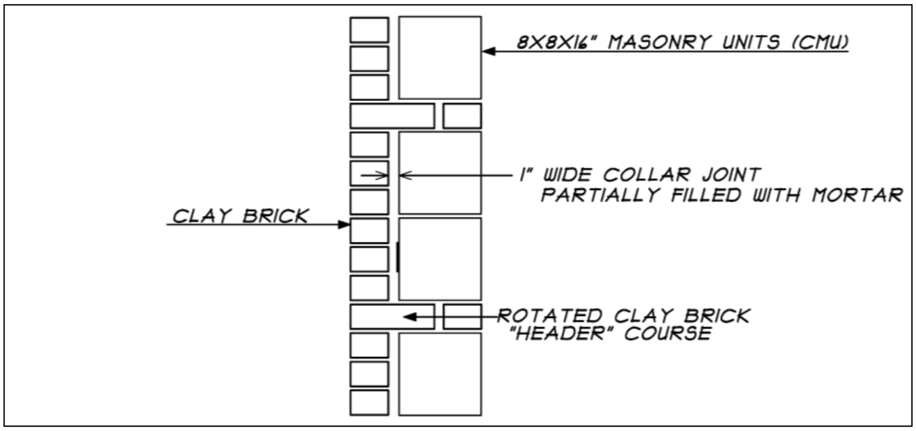

Figure 7. Thirteen-inch-wide brick/CMU wall (not to scale).

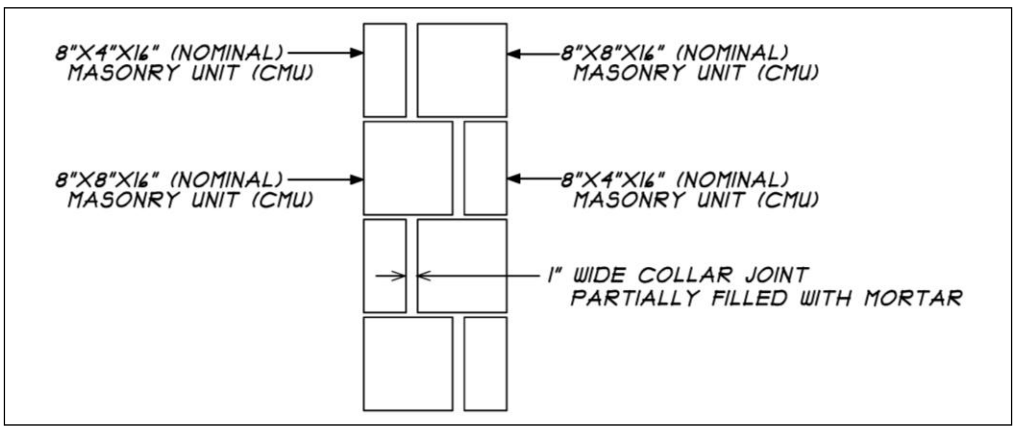

Figure 8. Thirteen-inch-wide CMU wall (not to scale).

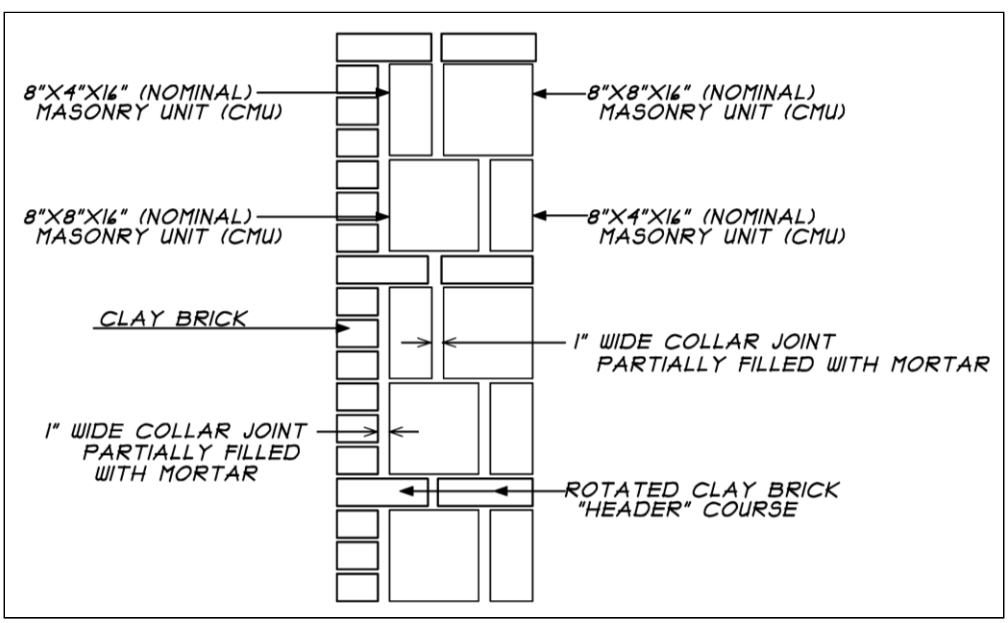

Figure 9. Seventeen-inch-wide brick/CMU wall (not to scale).

INSPECTION OF DAMAGE

Our first inspection of the school building was conducted about 7 hours following the seismic event of February 21, 2008. At this initial inspection we observed the following:

Gymnasium

- Two or three clay bricks that had fallen from a high window lintel in the south exterior wall of the gymnasium were lying on the gymnasium floor.
- Two horizontal steel braces, between the bottom chord of the northern roof truss and the north wall of the gymnasium, had pulled loose from their connection to the wall. These steel angle braces were anchored with two ½”-diameter anchor bolts to the top of a concrete tie beam at 22 feet above the floor.
- There were some diagonal cracks that followed mortar lines in the four masonry walls of the gymnasium. Similar cracking was also found in the walls of the adjoining locker rooms.
- A bracket supporting the basketball backboard at the north end of the room had been displaced several inches from its original location causing the backboard to jam when an attempt was made to raise or lower it.
- There were no signs of any cracking of any of the concrete frame members. An inspection of the main truss anchor bolts at the tops of the columns indicated that the bolts were sound and the concrete was not cracked around the bolts.

Auditorium

- The one exception to the 13- and 17-inch wide masonry walls described above existed in the common wall between the auditorium and the hallway to the east. This in-fill wall, built between the columns of the concrete frames, was constructed with ungrouted 4-inch thick concrete masonry. This 4-inch wall began at the concrete floor and extended to a height of about 10 feet above the floor. At 10 feet above the floor the thickness was increased to a 13-inch wall consisting of an 8-inch CMU and a 4-inch clay brick veneer, as detailed in figure 7. The lower 4-inch portion of the wall had significant cracks that followed the mortar lines of the block in a diagonal direction.
- The south end wall of the auditorium is common with the north wall of the gymnasium. At this wall the auditorium roof bar joists bear on the masonry wall at a location about 24 inches above the top of the concrete tie beam or 24 feet above the floor. Tension in the bar joists created by the lateral movement of the wall had pulled them from their seats in the pockets in the CMU wall. This caused severe shattering of the masonry units below these joists (see figure 3). The photograph shows that a very unstable condition was created in the roof and wall system of the auditorium as well as in the gymnasium because this wall supports a roof on each side. It will be noted later in the section “Repair and Remediation” that this wall, following the earthquake was left in a very unstable condition. A failure of this wall would have allowed a total collapse of at least 20 feet of the gymnasium roof and 15 feet of the auditorium roof.
- As in the gymnasium, there were no signs of any cracking of the concrete frame members.
- Other than minor hairline mortar cracks, we did not observe any cracking in the 17-inch-thick walls at the north and west sides of the auditorium.
- The chimney is integral with the north wall of the auditorium, which is 64 feet tall, receives lateral support from cast-in-place concrete roof above the stage area of the auditorium. This roof is located approximately 28 feet above the finish floor of the school. The seismic forces caused a shear failure in the chimney just above this auditorium roof line. The failure resulted in a horizontal displacement of the upper portion of the chimney of 4 to 6 inches with the upper portion moving to the north of the lower section (see figures 4 and 5).

Figure 3. Common wall between gymnasium and auditorium showing fractured masonry units below auditorium roof bar joists.

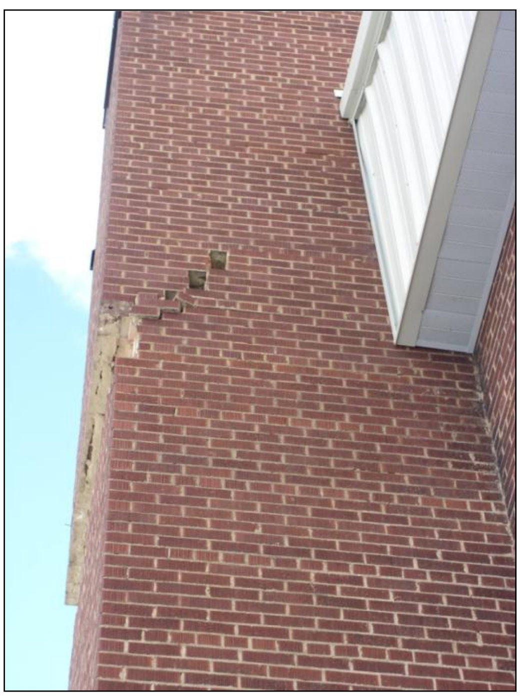

Figure 4. West wall of chimney. Note shear fracture and horizontal displacement.

Figure 5. North wall of chimney.

Classrooms and administration offices

The classrooms of the 1951 building experienced some cracking of the mortar joints, but they were all considered superficial and not a significant structural concern. There was no evidence of any cracking of any of the concrete frames located along the exterior wall or along either side of the hallway.

The new classroom and locker room additions of later years were constructed of reinforced concrete masonry and, while there was some minor cracking, there were no signs of any significant structural damage.

BUILDING CODES, ASSUMPTIONS, AND TESTING

At the time of the earthquake and during the time of the repair procedures, the City of Wells, Nevada had adopted and was currently operating under the 1997 Edition of the Uniform Building Code "UBC (1997)" along with the 1997 Edition of the Uniform Code for Building Conservation "UCBC (1997)." The "UCBC (1997)," with its appendix "Seismic Strengthening Provisions for Unreinforced Masonry Bearing Wall Buildings" was used as a guideline for the evaluation and design of the repairs of the existing building. Where feasible, the "UBC (1997)" was used for the design and construction of the replacement elements.

Testing was conducted to evaluate the condition of the mortar in the URM walls. These tests were conducted by an independent testing laboratory in accordance with the "UCBC (1997)" and the UBC Standard 21-6. The results of these tests indicated that the mortar shear strength V_{to} did meet the minimum value of 30 psi and for the purpose of our analysis a value for V_{to} equal to 30 psi was adopted for the entire 1951 portion of the school. Similar testing conducted in a limited portion of the newer additions to the school indicated the shear strength V_{to} to be in excess of 150 psi.

The City of Wells is located in the "UBC (1997)" Seismic Zone 2-b. Within this seismic zone, the code allows URM walls to exist provided their height-to-thickness ratio does not exceed 20. The code also provides for URM parapet walls provided their height-to-thickness ratio does not exceed 2.5.

Since the school was built in 1951, the following strength values were assumed for the existing building materials:

- Concrete: The plans specified a compressive strength of 2500 psi in 28 days; however a lower value of 2000 psi was used in the analysis.
- Concrete Reinforcing Steel: Reinforcing steel used during the period of construction typically had a yield strength of 40.0 ksi and this value was used in the analysis.
- Structural Steel: During this construction period ASTM A-7 steel was common and had a yield strength of 33.0 ksi. Therefore, the 33.0 ksi yield strength was used to analyze the existing steel components.

REPAIR AND REMEDIATION

The additions to the Wells High School that were constructed after the 1951 original building were constructed of reinforced masonry and concrete. These additions experienced only minor superficial damage from the earthquake and were, in general, excluded from further review. The mortar joints that did experience some superficial cracking were repointed. Repointing consisted of grinding out the damaged mortar to a depth of about two inches and then filling them with fresh mortar.

In order to eliminate the potentially hazardous conditions that existed and prevent the possibility of collapse and possible injury the following emergency measures were taken:

Chimney

It was learned from the Elko County School District that the chimney was no longer used and approval was given to remove the chimney. Structurally, however, the lower portions of the chimney were believed to provide stability to the 17-inch north wall of the auditorium and total removal could weaken the integrity of that wall. Therefore, the contractor was directed to dismantle the chimney to the level of the auditorium roof or to a level lower as required to eliminate any loose masonry that may exist. Following the removal of the upper portion of the chimney, a steel frame was designed to provide stability to the top courses of brick and a steel-sheathed roof structure was added to provide a storm-proof enclosure for the open end of the chimney.

North and south gymnasium walls

Shoring was designed to provide support to the auditorium roof bar joists that had caused the fracturing of the masonry in this wall. This shoring consisted of square tube steel columns and high-strength grout and was designed to provide permanent support and allow the contractor to remove and replace the fractured masonry.

However, during the process of installing these supports it was discovered that the portion of the 13-inch-thick double-wythe masonry wall (figure 8) above the concrete tie beam had delaminated. A vertical fracture line occurred along the vertical collar joint and continued through the masonry units causing the two wythes to act independently. Therefore, as two independent wythes or walls, this upper section of wall no longer met the 20-to-1 height-to-width ratio requirement. Now acting as two walls moving independently away from each other a P-Δ moment was created in each wall. The combination of a significant increase in the height-to-width ratio and the P-Δ moment created an extremely unstable condition for this bearing wall. This is the same wall previously noted and shown in figure 3.

During the period that these procedures were taking place, several aftershocks occurred. These aftershocks caused the delamination of other multiple wythe walls to become evident. Cracks began to appear in the upper level of the south wall of the gymnasium. This wall had several large windows that reduce the strength of the wall and displacement of the clay brick veneer became more apparent with each after shock. Removal of selected clay bricks from the wall provided verification that the separation of the two masonry wythes had occurred and again resulting in an unstable condition.

The tie beam located in each of these end walls is 22 feet above the finish floor. The wall section above this beam is arch-shaped to match the arch of the barrel-shaped roof trusses. The 2” x 12” wood roof rafters that span 20 feet bear on the top of this upper arched section of wall. It appears that the original contractor built the masonry portion of the arched wall to the level of the bottom of the joists, then set the joists on the masonry units and poured a concrete cap beam 13” wide by 7 to 12 inches high to encapsulate the ends of the joists. Steel anchors had been installed to tie the ends of the joists horizontally to the cap beam and wall. The delamination of the masonry wall was limited to this arched section above the tie beam.

The repair of both the north and south end walls of the gymnasium was performed in basically the same manner, but the north wall was complicated by the existence of the auditorium roof. The basic repair procedure was as follows:

- Vertical steel tube supports were installed at about 9-foot centers between the tie beam and the thin concrete arched cap beam. Twelve-inch wide sections were saw-cut and removed from the damaged masonry wall. Anchor bolts were set in adhesive grout in the top of the tie beam to anchor the vertical steel tube supports at the 9-foot-spacing.
- After the steel tube supports were installed and fastened to the concrete beam and the arched cap beam, the damaged masonry wall was removed with the exception of about 10-feet at each end. These end sections were left to insure stability of the arched cap beam and lateral stability of the roof during the removal and replacement of the center portion of the wall.
- An 8-inch reinforced concrete masonry wall was constructed to replace the original wall. The steel support tubes were allowed to remain within the new CMU wall. Because of geometry requirements of the common wall between the gymnasium and auditorium, 12-inch reinforced masonry was used.
- After the new wall had been grouted, the 10-foot end sections were removed and replaced with reinforced masonry walls.

East and west side walls of gymnasium

The east and west walls of the gymnasium also contained very large windows. These windows, approximately 7- to 10-feet high by 18-feet wide, had been permanently covered with sheet metal to eliminate undesirable lighting of the gymnasium floor. During the lateral load analysis of the gymnasium it was revealed that there were two areas where the gymnasium did not meet the lateral load requirements of the “UCBC (1997).” These problem areas were the connection of the high roof diaphragm to the east and west walls and the lack of shear strength in the plane of the walls themselves. The large window openings that existed above the roof of the locker rooms reduced the shear strength of the wall such that the required strength exceeded the tested mortar strength of the wall as well as the strength of the concrete columns.

To correct these conditions, the following modifications were designed and constructed:

- A revised roof diaphragm-to-wall connection was designed and constructed to provide a stronger connection that met the code requirements.
- Selected windows in the east and west walls were filled with 8-inch-thick reinforced and grouted masonry. The reinforcing steel was embedded in adhesive grout into the beams and columns of the concrete frames.

Repair of the lateral bracing of the gymnasium roof trusses

At some time during the life of the school, the basketball backboards had been upgraded to a heavier retractable style of standard. These backboards were supported by the lateral truss bracing that existed between the end walls and the bottom chord of the adjacent truss. The higher backboard weight increased the stresses in the bracing members. As a part of the lateral load analysis of the gymnasium structure the lateral bracing between the roof trusses and the north and south walls was reviewed. Additional bracing was designed and installed to improve the load transfer between the wall and the roof diaphragm and the support the weight of the backboards.

During the construction of these bracing modifications, additional bracing connection failures were discovered. These connections were also repaired and strengthened.

Auditorium walls

The lower section of the east auditorium wall is common with the west wall of the main school corridor while the upper portion of this wall, above the low classroom level, is an exterior wall. The lower portion was constructed of 4-inch masonry and the upper portion was constructed as a 13-inch wall having wythes of 8-inch CMU and clay brick veneer. As the area experienced several heavy aftershocks, it became apparent that the multiple wythes of the upper portion of this wall had also delaminated or separated (see figure 6).

The three sides of the auditorium roof not adjacent to the gymnasium were surrounded by an 8-inch-wide by 24-inch-high clay brick parapet. This unreinforced brick parapet did not meet the height-to-thickness ratio required by the "UCBC (1997)" and required either bracing or removal. An evaluation of the fire code requirements, considering recent improvements to the school's fire suppression system, indicated that the parapet was not required. A final determination by the City of Wells Building Department confirmed this conclusion and they allowed for the elimination of the parapet wall.

The process of removing the brick parapet from the perimeter of the auditorium roof allowed a thorough examination of the multi-wythe masonry to verify the extent of the delamination. The 17-inch thick masonry walls on the west and north sides of the auditorium appeared sound and undamaged. The area of delaminated wall was limited to the east wall. This delamination extended from the north wall of the gymnasium to the cast-in-place concrete roof above the stage area. The damaged wall was removed and a new 8-inch-thick reinforced masonry wall was constructed from the floor level to the roof diaphragm along this section of the auditorium.

Figure 6. Delaminated brick and masonry wall at southeast corner of auditorium (roof level).

Along the east and west auditorium walls, the parapet wall had to be removed to the top of the upper beam of the concrete frame. The top of this beam is lower than the roof slab by about 16 inches. This created a gap or opening in the top of the wall (see figure 6). This opening could not remain or be covered with a light-weight siding as it violated the fire-resisting envelope of the auditorium. The gap was filled with 8-inch-thick reinforced concrete masonry that was grouted solid. The construction of this wall provided a means to improve the connection between the concrete roof diaphragm and the concrete frames.

North Classroom Wing

The classroom portion of the school to the north of the auditorium was reviewed and analyzed as an independent structure, as it was separated from the main building by an expansion joint in the roof and walls. The expansion joint consisted of a 1-inch separation at the roof slab and the independent concrete frames on each side of the expansion joint had independent columns separated by the 1-inch joint.

The lateral-load-resisting system of this classroom wing consists of four concrete rigid frames that are located in the east and west exterior walls and in the two walls on each side of the central corridor. A 12-inch by 14-inch beam is supported by 12-inch by 15-inch columns spaced at 12 feet along the east and west exterior walls. A 12- by 20-inch beam exists in each of the two corridor walls, and they are supported by 12-inch square columns spaced at 18 feet on center.

In the lateral-load analysis of this classroom area, it was determined that the four concrete rigid frames had adequate strength to resist the "UCBC (1997)" required forces in the north south direction.

A crawl space about 5 feet high exists below the floor of these classrooms. Therefore, the footings that support the columns are located four to five feet below the cast-in-place concrete slab-and-beam floor system of the school. This separation between the footings and the rigid elevated slab-and-joist system created a fixed restraint for the columns at the top of the floor slab. The columns were reviewed as inverted pendulums to resist the lateral forces in the east-west direction (see schematic diagram of column in figure 10). The analysis was conducted twice: once on a symmetrical building ignoring the URM wall at the north end and again with the horizontal discontinuity of the URM wall in place. This analysis indicated that the columns have adequate strength to resist the required code forces with or without the unreinforced masonry walls.

The fact that only minor cracking occurred in the walls during the recent seismic events served to reinforce this conclusion. Therefore, other than the repointing of the superficial cracking of the mortar joints, no structural modifications were made in this classroom section of the original building.

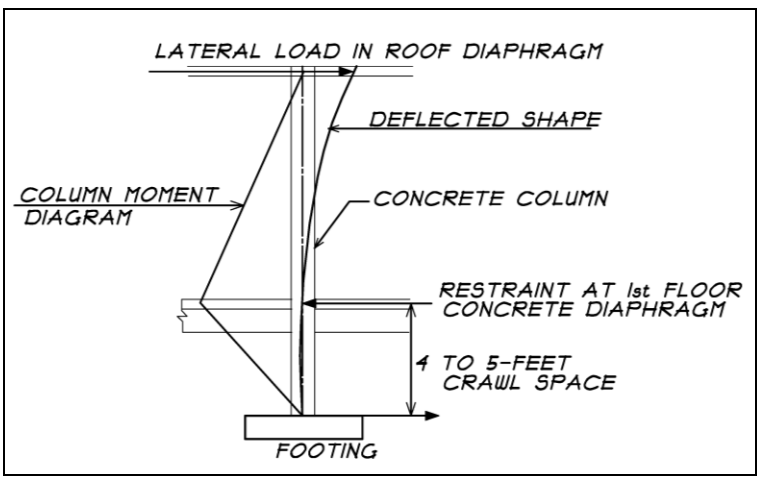

Figure 10. Low roof column restraint (not to scale).

SCHEDULING AND PHASING OF REPAIRS

The contractor began demolition of the brick chimney and the cleanup of debris the day after the February 21st earthquake. After an intense search, the 1951 drawings for the construction of the original building were located on April 21, 2008. These plans proved to be invaluable and contributed significantly to the designer's knowledge of the building and to the final analysis and reconstruction of the original structure.

Under the direction of the Elko County School District, the goal of the contractor was to complete the repairs of the gymnasium first, followed by the auditorium and classrooms. The ultimate goal was to allow occupancy of the school building by the students by August 18, 2008, the scheduled opening day of school. In addition, it was desired to allow limited occupancy by the teachers and administration by August 13, 2008. To meet this requirement, the contractors had to not only complete the major structural components, but also address other life safety issues such as testing and certification of fire alarms and fire suppressions systems and completion of hazardous material cleanup.

With the approval of the City of Wells the project was divided into three phases: the gymnasium, the auditorium, and the north wing classrooms. Structural plans for the first phase, the gymnasium, were reviewed and approved for construction by the City of Wells on April 24, 2008 and the contractor began the reconstruction work for the gymnasium immediately. This phase also included the north gymnasium wall common to the auditorium.

In order to accomplish these imposed deadlines, weekly coordination meeting were conducted by the Elko County School District. These meetings were attended by all interested and involved parties, including the members of the school district, the school board, the school principal, the contractor (BELFOR Property Restoration), the engineers, the city manager and representatives of the City of Wells, and the insurance company. Through the efforts and coordination of all involved parties, the unsafe conditions created by the earthquake and many aftershocks were identified, corrected, and reconstructed in an efficient manner, meeting the occupancy dates established by the school district. A total of 110 days were required to construct the structural repairs and to complete the other life safety issues required for occupancy.

ACKNOWLEDGMENTS

The author would like to acknowledge John Bland of the Elko County School District for his efforts in organizing and chairing the weekly meetings and coordinating the many disciplines involved, including the contractor, engineer, city and state officials, and the school district. He would like to acknowledge George Ostan of BELFOR Property Restoration, the contractor, for his professional approach and efforts in coordination of the many sub-contractors required to accomplish this project in the allotted time constraints.

The author would also like the thank Dr. David Sanders, Professor of Civil and Environmental Engineering at the University of Nevada, Reno, for his review and comments that greatly improved the contents of this paper.

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