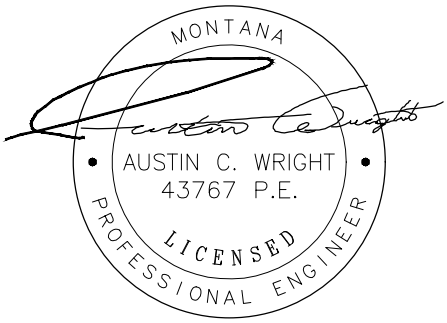
Tier 1 Assessment
Linderman Gymnasium

Polson, MT



June 15, 2018

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June 15, 2018

Mr. Paul Bishop
Paradigm v.2.0
125 ½ W. Main Street
Missoula, MT 59802

RE: Linderman Gymnasium Polson, Montana

Dear Mr. Bishop

We are serving as structural consultants for design of the Linderman Gymnasium. In this role, we provided structural analysis, design, and review of the existing structure as well as a site visit for structural observation. All documents and calculations were prepared in keeping with the local standards of structural engineering practice and principles. Complete work considered self-weight, code live load, code wind, code seismic and all applicable combinations of these loads.

Structure

The Linderman Gymnasium is constructed from a concrete column and beam system with unreinforced brick masonry infill. The roof framing is constructed of wood framed construction with wood bowstring trusses over the gymnasium and wood rafters over the lower roofs of the structure. Also found in the structure are bleachers and gymnasium floor that are built from wood framing. Interior walls of the structure were constructed both from unreinforced brick masonry and 2x wood framing with gypsum wall board. During the initial site visit the following damages were found; visible cracking was found in the unreinforced brick masonry walls, checking was found in many of the structures wood framing elements, cracking was found in many of the concrete bond beams, these appeared to be primarily shrinkage cracking on the North side of the building and a combination of shrinkage cracking and flexural cracking on the South side of the building. Along with the bond beam cracking it was found at many of the gypsum wall board seams were cracked as well. Cracking in the gypsum wall board was found to be predominately on the South side of the structure. It was also visible where the parapet was missing along the South wall of the gymnasium and where the South locker room had been removed. Due to cleanup that had taken place, failed elements associated with the locker rooms were not available for observation. Number convention started from the North West of the building, columns are numbered starting at the West and proceed East with a total of 5 concrete columns, the truss numbering is in reference to the column numbers. Windows were numbered starting in the North and numbered sequentially around the gym as shown in Figure 1.

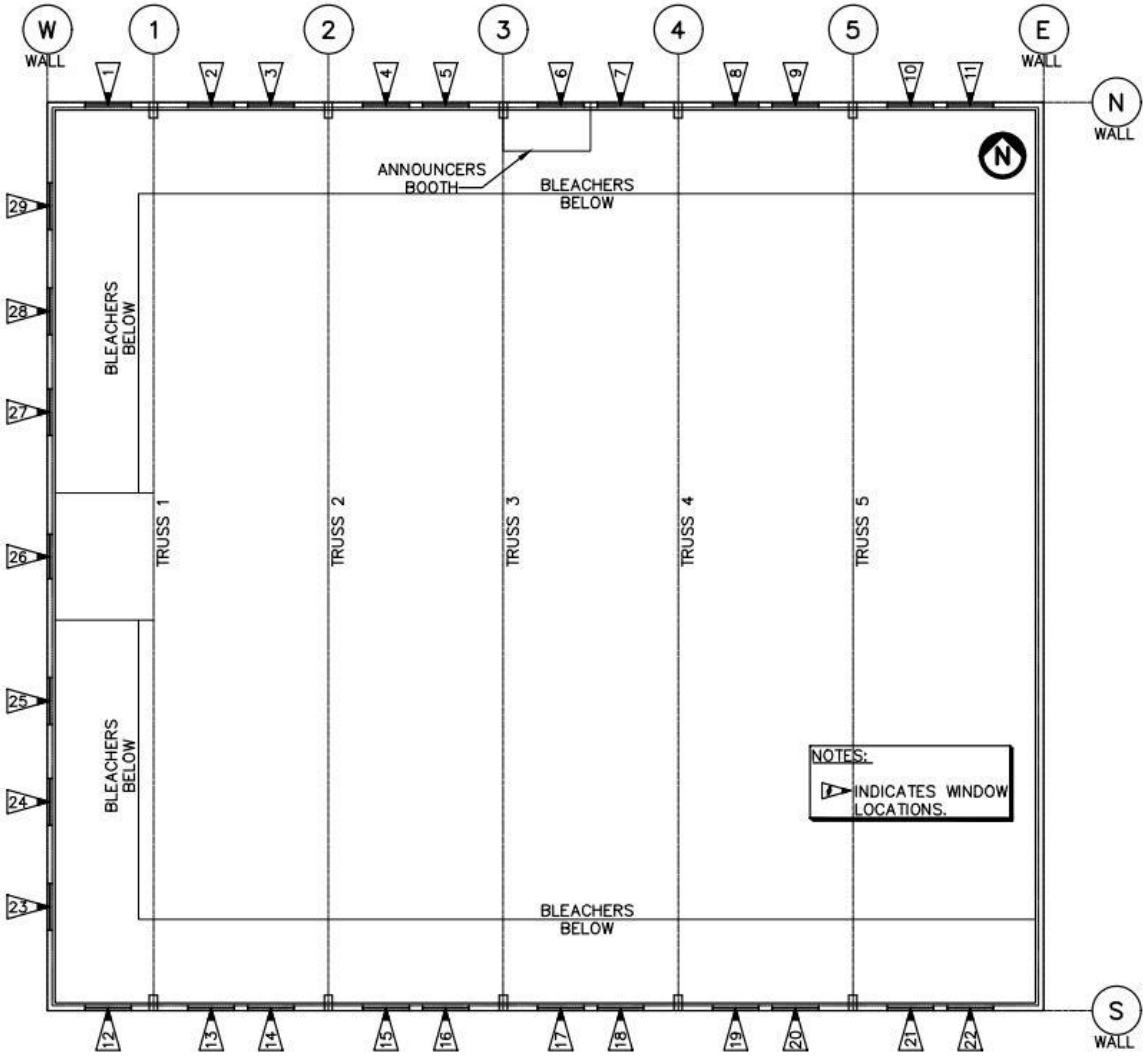

Figure 1 Gym plan view indicating column lines, truss numbers, and window locations.

Roof framing

The Roof framing consists of wood bowstring trusses that span the width of the gymnasium. Roof trusses consist of 2 - 5-1/2”x9-1/2” solid bottom chord members spliced in 3 locations using 3/4” diameter bolts with 3” washers and 4” diameter split ring connections, using 12 at each splice location. The top chord of the truss consists of 2 - 5-1/2”x14-1/8” Glulam members at their deepest that taper down at the ends to 7-5/8” deep. Splices occur at member ends and center line using 6 – ¾” diameter bolts with 2-1/2” diameter split ring connectors. Truss web members consist of both 3-1/2x5-1/2” and 5-1/2x5-1/2” members that are connected at each end with 1 – ¾” bolt with 3” diameter washer with 2-3/4” diameter split ring at each end. Rafters spanning between the trusses consist of 2x12’s at 16” on center. The rafters support an acoustical panel tile ceiling

using 1x4" members at 12" on center and the exterior of the roof is sheathed with 1x8" shiplap and supports the roof membrane.

It was observed that the truss members showed signs of aging and checking through many of the truss top chord, bottom chord, web members, and wood splice members. At truss lines 3 and 4 the bottom chord of the truss has 7/16" steel plates added to the bottom and sides. At the bottom chord splice points the bolts washers were welded to the steel plates to reinforce the bottom chord member of the trusses and aid in supporting the roof top mechanical unit that was added during the remodel.

Figure 2 Bowstring roof truss number 1

Figure 3 Bowstring truss South end.

Figure 4 Bowstring bottom chord splice

Figure 5 Bowstring truss bearing end splice.

Figure 6 Bowstring top chord splice.

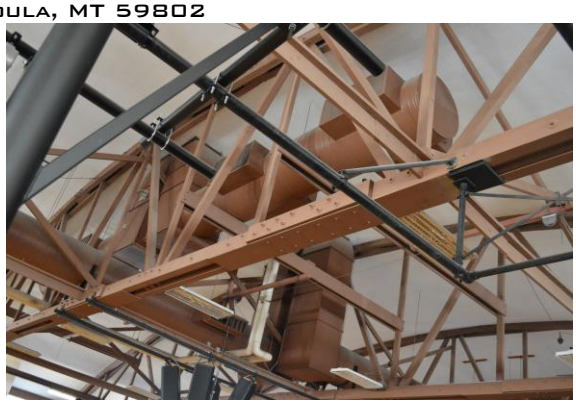

Figure 7 Duct work and bottom chord reinforcing.

Walls

The masonry walls in the structure consist of 3 wythe unreinforced brick masonry infill between reinforced 16”x 24” concrete columns that support the bowstring trusses. Located at 12’-10” above grade there are 16”x32” deep concrete bond beams between the columns. The walls include 2 wythe unreinforced brick masonry parapets that extend above the roof deck framing. Also, on the interior of the building are 3 wythe unreinforced brick masonry walls and 2x6 wood framed walls that support the structure. The 2x framed walls are non-structural partition walls within the space.

Cracking was found during the onsite observation. At the West wall in the South West corner cracking along mortar joints and through brick units was found propagating from the upper portion of the corner and traveling diagonally down and to the North. At the base of window 23 the separation was found to be large enough that the mortar had separated from the brick above and below and could freely move in the joint. Also found was at the location where the cracking extended to below the bleachers level the varnish was cracked and separating indicating the cracking was recent. Similar cracking was found along the South wall propagating up at a diagonal from below the bleachers to the West most window through the varnish layer. This cracking continued along the wall at the base of the window elevation and up at the columns along the mortar joint tot the East wall. At the South East corner wall cracking was found to propagate from the upper corner diagonally down and to the North along grout lines and through brick units to below the stairs. At the North West corner of the building diagonal cracking was found on the West wall around the openings and down to below the bleachers, this crack also travels through the varnish and appears to be recent. Starting at the West wall along the length of the North wall at the base of window elevation and up the columns cracking is visible in the mortar joint to the East wall. At the North East corner cracking is also visible from the upper corner diagonally down to the stair case. Also, on the South wall where the stage was infilled vertical cracking has occurred between the brick masonry and the cement masonry units. From below the bleachers the diagonal cracking on the

West wall in the North West corner what appears to be the crack from above travels down to where it terminates at the bond beam.

Figure 8 Loose mortar at window 23 crack.

Figure 9 West wall cracking at varnish.

Figure 10 Cracked mortar below window 12.

Figure 11 Cracked varnish South below window 12.

Figure 12 Cracking at East wall South corner.

Figure 13 Cracking at East wall South corner.

Figure 14 Cracking above window 27.

Figure 15 Cracked varnish North West corner.

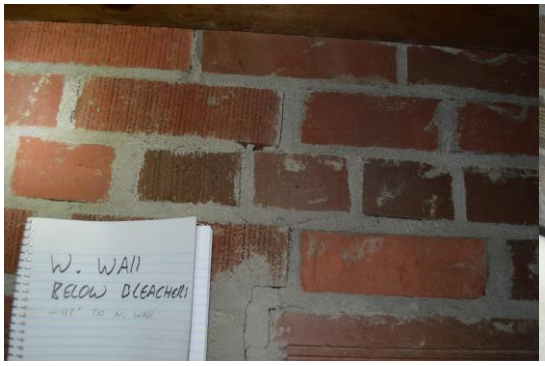

Figure 16 West wall crack below bleachers.

Figure 17 Wall crack West wall at bond beam.

Along the bond beams at the North wall cracking was found to be typically located within the middle 1/3 of the beam span with 1 or 2 cracks per bond beam. The cracks were vertical in nature and were typically 1/8 of an inch at the top and a tight hairline at the bottom indicating that they are shrinkage based and that there may be reinforcing located at the bottom of the beams. This is supported by the finding of a void in the concrete along the bond beam between column 4 and 5 at 22-3/4" from column 5 in the upper portion of the beam. This void shows a vertical #3 shear tie and a #8 horizontal bar bent at 45° toward the bottom of the beam. At the South wall of the building the bond beams were also visible from the mechanical space below the bleachers.

Figure 18 Typ crack in North bond beams.

Figure 19 Visible rebar in bond beam between 3&4

At the South wall there was an increase in cracking regularity typically finding 5 or 6 cracks per bond beam. The South bond beams had cracking that trended both vertically and diagonally some of these cracks were similar to the North bond beams being wider at the top and narrow at the bottom and some of the cracks were consistent top to bottom. The cracking found at the South wall indicates similar reinforcing to the North wall bond beams shrinkage cracks. The increase in cracking reflects the failure of the parapet on this side of the building and the forces induced on the system.

Figure 20 Typ. vertical crack in South bond beam.

Figure 21 Typ. diagonal crack in South Bond Beam.

The failure at the parapet was also observed with the failure occurring where the unreinforced masonry units transitioned from a 2 whyte at the parapet to the 3 whyte of the wall. From the description given the parapets appeared to have initiated between trusses 2 and 3, the wall deflected away from the building with snow curling behind it as is slid off the roof until the wall became unstable and fell landing on the roof of the locker rooms below. It was visible where the roof membrane had torn at each of the locations where it was attached to the unreinforced brick masonry. Also found at the exterior of the building was cracking from the parapet failure around the corners of the building.

Figure 22 Failed South wall parapet.

Figure 23 Failed South wall parapet.

Figure 24 Failed parapet unreinforced masonry.

Figure 25 South wall crack at parapet, West end.

Figure 26 South wall crack at parapet, East end.

Figure 27 East wall crack at South parapet failure.

Cracking was also found in the occupiable space of the ground level. Cracking was found in the gypsum wall board seams and was located at the East, West, and South sides of the building. There was an increase in occurrence along the South hallway of the building where cracking was found at nearly all gypsum seams along the hallway ceiling and in the band room walls and ceiling all of which appeared to be new. Along with cracking at the gypsum wall seams it was described to me that one of the ceiling panels fell from the ceiling of the band room and that one of the light fixtures in the South end of the foyer had fallen from its bracketing shortly after the parapet failure. The fallen panel was visibly broken and moved to a wall while the light fixture still was hanging from the ceiling. At the East wing of the building cracking was found in the library at the corners of the room and in the upper portion of the interior wall and appeared to be new and at gypsum wall board seams. Cracking at the West side of the building was found in the mechanical room and along the hallway and above the doorways all of which appear to be new and at gypsum wall board seams.

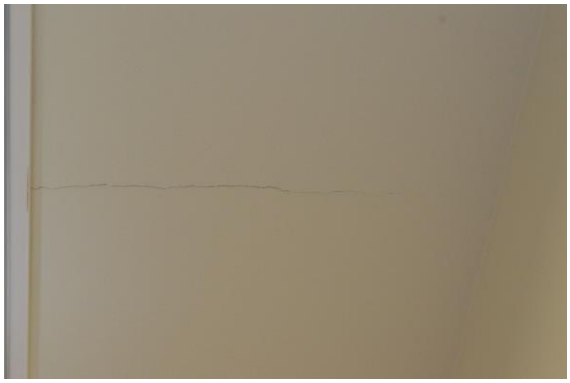

Figure 28 Typ. cracking in South hallway.

Figure 29 Typ. cracking in South hallway.

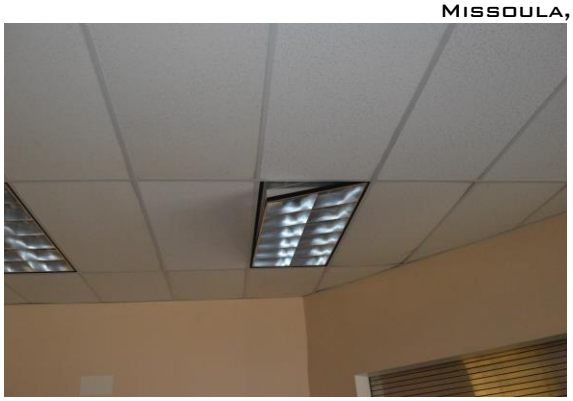

Figure 30 Falling light fixture in S.W. lobby.

Figure 31 Missing ceiling tile in band room.

Figure 32 Broken ceiling tile in band room.

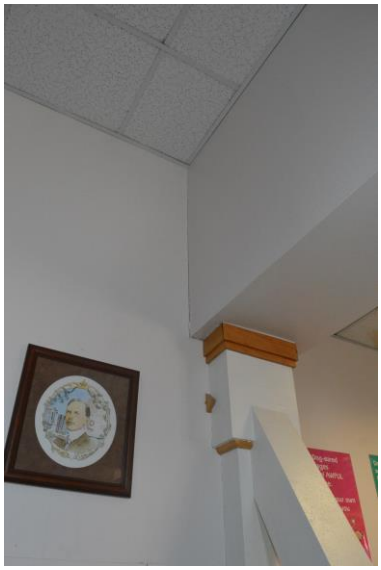

Figure 33 Cracking in Library at ceiling.

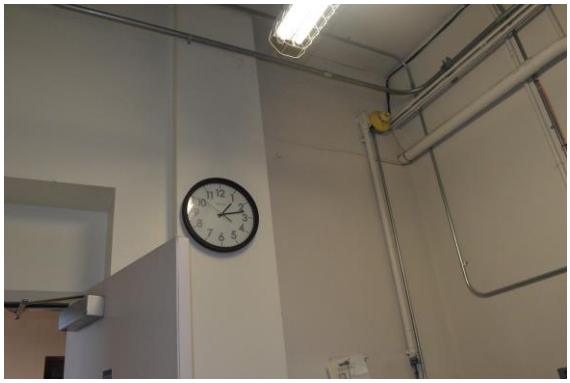

Figure 34 Crack in West mechanical room.

Figure 35 Crack above doors in West hallway.

<u>Bleachers</u>

Access to the North side bleachers was accessible and allowed for observation of the bleacher structure. Direct observations were made at the North side of the building the South side of the building had been retrofitted as a mechanical space and the underside of the bleachers was sheathed with gypsum board as a part of the retrofit limiting visible access to them. The bleachers in the structure are constructed from 2x12" wood members at 16" on center and supported at the exterior unreinforced brick masonry walls with a 3-1/2x7-1/4" ledger anchored to the wall with 5/8" diameter anchor bolts at 3'-0" on center. At the interior of the structure the walls are supported by a masonry wall with a 2x sill plate. At both ends the 2x12" members are birds mouthed and attached using what appears to be 16d toe nails. Attached to and built from the 2x12's are the bleacher seats constructed from 2x4 framing and sheathed with solid 1x material.

Figure 36 Typ. bleacher framing.

Figure 37 Typ. bleacher bearing at wall.

Figure 38 North bleacher bearing at wall.

Figure 39 West bleacher bearing at brick wall.

Under the South bleachers was found that the gypsum sheathing tape seam joining the gypsum wall board installed to the underside of the bleachers framing to the rim board at the wall had separated. This separation occurred most heavily between columns 2 and 3. It is visible where the tape seam has separated and there was loose gypsum flaking found below the separation indicating it happened recently.

Figure 40 Separated gyp. tape at S. wall col. 2-3

Figure 41 Separated gyp. tape at S. wall col. 2-3

Figure 42 Separated gyp. tape at S. wall col. 2-3

Figure 43 Loose gyp mud found below ledger.

Foundation walls and Footings

The foundation walls were observed from the mechanical chase that runs along the perimeter walls of the structure. This mechanical chase allowed for examination of the foundation wall at multiple locations. At various locations around the perimeter of the building penetrations have been made for mechanical system plumbing. Examination showed that the foundation walls were reinforced with rebar and did not show signs of large settlement or heave. Direct observation of the footings was not possible from the mechanical chase.

Within the chase it was found that the North foundation walls had only occasional hairline cracking. At the South side of the building the foundation walls had more cracking, hairline and wider cracks were found along the foundation wall.

Figure 44 North foundation wall reinforcing.

Figure 45 South foundation wall reinforcing.

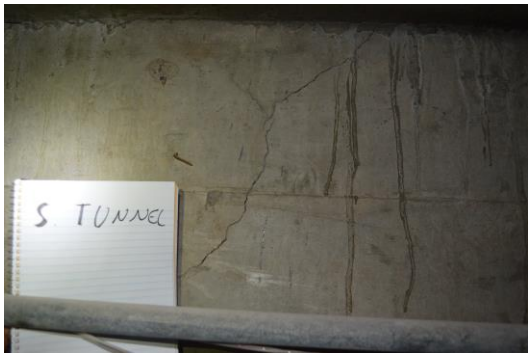

Figure 46 Cracking in South foundation wall.

Figure 47 Cracking in South foundation wall.

Figure 48 Cracking in South foundation wall.

Figure 49 Broken pipes below locker rooms.

Structural Tier 1 Assessment

The initial analysis has been preformed utilizing the ASCE 41-17 standards for Tier 1 assessment. Tier 1 assessment is a screening of the building to identify the elements that do not meet the basic performance objectives for the existing structure and what will need to be evaluated using the Tier 2 level assessment to determine the best method of repair or retrofit. For structures used as a public building with an occupant load of greater than 300 or an educational building with an occupancy load of 250 or greater assigns the structure with a Risk Category III designation. For the Tier 1 assessment a basic performance objective for existing buildings (BPOE) was selected based on the Risk Category and assigns a performance objective and seismic hazard of the building. Per the ASCE 41-17 a Risk Category III structure is to be assessed for seismic loads for Collapse Prevention Performance Level (S-5) loads based on Basic Safety Earthquate-2 level forces (BSE-2E). This ensures that the structure provides adequate resistance to BSE-2E forces, BSE-2E forces are based on a seismic hazard with a 5% probability of exceedance in a 50-year period and set forth by the ASCE 41-17.

Along with the Tier 1 analysis the structure was assessed to determine what effects the failed parapet wall at the South wall of the building and associated damage/removal of the locker rooms at the South side of the building on the remainder of the structure. The existing bowstring roof structure is also under investigation and concern to determine capacity and possible need of repair from the forces induced by the parapet failure.

During the site visit to look at the existing structure it was found that along with the missing parapet wall along the South wall of the building there is cracking of the masonry bond between the brick masonry at the window base elevation. It was also found that the structure had increased occurrence of cracking in the South wall bond beams and that the gypsum wall board joints located within the mechanical space, under the South bleachers has separated from the wood ledger that is attached to the brick masonry wall. Along with the separation of the joint there was material found below indicating that the separation was recent. Cracking was also found in the gypcrete wall board seams at the East, West, and South wings of the structure, these cracks appear to be recent and is our opinion that they are a direct result of the parapet failure.

From the Tier 1 analysis the existing structure does not meet the current code design demands required by the S-5 designation. Based on the analysis both the columns and walls will exceed the design capacity allowed by the code and will require further investigation and testing to determine the capacity of these members to develop a lateral system to resist the current code level forces. Under the tier 1 analysis when BSE-2E loads are applied the concrete columns are stressed to 132% of their capacity and the unreinforced brick masonry walls are stressed to 311% of their ultimate capacity. Both elements will require further evaluation using the Tier 2 analysis from the ASCE 41-17 in order to more accurately develop the level of stress and develop a retrofit solution to meet

the ASCE 41-17 design loads. The force induced onto the structure from the failing parapet wall and associated cracking throughout the building has left the structure at lower performance level than what the ASCE41-17 tier 1 analysis calculation shows. In order to determine the best method of repair for the existing structure from the parapet failure and to design a retrofit for the existing structure further analysis will be performed using the Tier 2 assessment from the ASCE 41-17. This will determine the extent of the repairs and methods for retrofit/repair will be developed based on the Tier 2 analysis to bring the structure to a strength level that can meet the current loading requirements.

Parapet Failure

In addition to the Tier 1 assessment an analysis was preformed to determine possible causes of the parapet failure and the extent of the damage to the structure. First an analysis of the trusses was preformed to determine the effects of the parapet failure and any structural integrity issues of the trusses. To accurately determine the effects of the failure on the trusses a model was developed based on measurements and information taken on site. Loads were developed from onsite information, historical data and current code requirements. These loads were then applied to the truss model and used to determine the effects on the individual members within the trusses. Load combinations were developed based on both the historical snow loads and current code snow loading to determine the effects. Historical loading information requires a minimum snow load of 20.4psf be applied. When the roof is analyzed under current code the minimum required snow load to be applied to the roof is 33.6psf at the eaves and increases to 50.3psf at the crown and is required to be applied in conjunction with drift loading, unbalanced snow loading, and the lateral load from the failing parapet wall on the top chord of the truss.

The truss model various loading results make apparent that the truss can support the design loads from the initial design before the building was constructed. When the snow loads are adjusted to the current load requirements and the additional loads from the mechanical piping and duct work are added the truss bottom chord members become stressed to 99% of their ultimate capacity. Next the load from the falling parapet was added to the top chord of the truss these members become stressed to 102% of the members ultimate capacity. This increase causes the truss members to exceed their ultimate design capacity set by the ASCE 41-17 and may have created internal lateen damages that are not readily visible. The possible lateen damages within the truss members or at connections may have left the truss members in a state of reduced performance that will limit the trusses ability to resist the design loads and reduce the truss members ultimate capacity by an unknown amount. It is our opinion that the trusses will need to be further examined to determine if there are any retrofit/repaired measures that can be implemented to ensure the trusses will meet the loads and standards set by the current code.

Figure 50 Bowstring truss model isometric view.

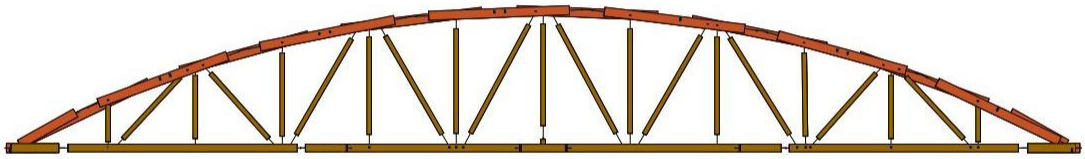

Figure 51 Bowstring truss model elevation view.

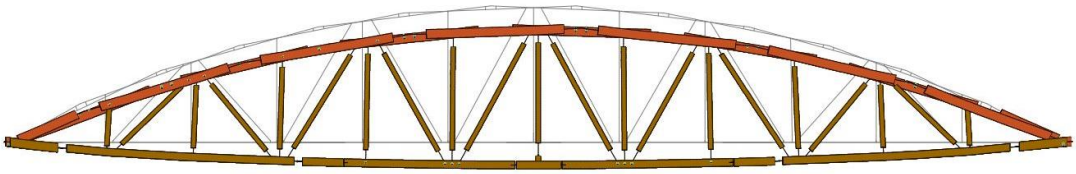

Figure 52 Bowstring truss model deflected elevation view.

To determine the loads applied to the roof and to determine a possible cause of the parapet collapse an analysis based on historical snow loads and current design snow loads was performed. Under the historical design roof snow loading for a roof with a slippery unobstructed surface the parapet walls would have adequate capacity to resist both the drift and sliding forces. The combination of the sliding snow load and drift loading would stress the parapet wall to 88% of the unreinforced brick masonry out of plane loading capacity. This would have left the parapet without damage however with the advancement of the code many of the snow loads in our region have increased to more accurately represent actual conditions this includes the Polson region.

When the current code design snow loads are applied to a roof with slippery unobstructed surfaces combined with the current code design drift loading against parapet walls. The unreinforced brick masonry parapet wall would have enough capacity to resist the out of plan force applied to it from a combined drifting event. This combined event would cause the out of plane unreinforced brick masonry bond stress to be 76% of its capacity. The parapet wall would be able to resist this amount of force and support the snow load combined with drift.

In addition to the out of plane force from a snow load combined with drift snow load a sliding snow load event must be considered. A sliding event is caused if the roof has a smooth enough roof material, this allows snow to slide off the roof where it collects behind the parapet wall. When the roof is fully loaded with snow and the snow then slides off the roof into the parapet wall there is an out of plane force that is applied to the parapet wall. This sliding snow load from ¼ of the roof span would stress the unreinforced masonry bond to 111% of its capacity. The assumption of ¼ of the span is an assumption and the load may be anything from ¼ the roof span to ½ the roof span. At ¼ of the roof span this force would be enough to break the masonry bond and cause the wall to begin to show deformation. Once the unreinforced brick masonry bond has been broken the parapet wall strength is reduced to where a repeated sliding event the parapet would be stressed to 133% of the parapet walls overturning capacity. At this stress level the parapet wall would deflect to a point where the center of mass was pushed outside of the walls kern envelope. Once the walls center of mass is outside of the walls kern envelope the walls ability to reach overturning has been exceeded causing the parapet wall to fall from the upper level and impacting the lower roof over the locker rooms below. This sliding snow event is what caused the curling of the snow that was documented prior to the parapet failure.

The parapet wall failure may not have been solely from a sliding snow event as sited previously. A combination of wind drifting and a sliding snow load could also create an event that could have caused the parapet wall failure, both of which are highly likely. The eventual failure of the parapet wall would most likely be to a combined event from a wind/snow storm followed by a warming event would cause the snow to slide on the roof membrane causing the snow to slide from the roof and into the parapet wall.

In conjunction with the parapet wall failure at the base of the windows above the bleachers cracking in the wall is visible and can be contributed to this snow event. The combined drift and sliding forces stressed the wall elements between the windows to 131% of their capacity causing cracking. This force is not enough to cause failure and overturning of the wall. It is enough load that the mortar bond between the bricks capacity would be exceeded to a point of cracking.

Conclusion

From the parapet wall failing, additional loads are applied to the building. This load is distributed to the roof diaphragm and was approximately 41.5kips of force that had to be resisted by the buildings lateral force resisting system. To be conservative this load was not increased by an impact factor and was rather applied in the form of a smooth tension load that was applied to the top chord of the truss. The combination of force induced on the gym roof and the load applied from the falling parapet onto the lower locker room roof in our opinion is what resulted in the visible damages within the interior of the structure. This is specifically noticeable, at the isolated location on the South side of the building between truss and column lines 2 and 3where visible cracking is found to be more prevalent than the North side of the building. The failure of the parapet forces on the structure was carried throughout the lateral system of the structure causing damages to many of the gypsum wall board seams and masonry grout lines through the structure. leaving the structure at a reduced level of performance from where it was prior to parapet failure. The out of plane force from the parapet wall would have caused a force to be applied at the roof diaphragm toward the South and is also substantial enough to expect out of plane movement of the walls as shown by the survey report.

To develop a plan to retrofit/repair the structure further testing and design will be required based on the Tier 2 assessment from the ASCE 41-17. This will aid in developing a system to repair/retrofit the existing structure to a point that will bring the Linderman gymnasium structural supporting system up to the current design standards.

Sincerely,

Austin Wright, PE
Aegis Engineering Incorporated