Detailed Structural Assessment
Southern Nevada Health District Main Building
Las Vegas, NV

Prepared for
PGAL, Inc.

Prepared by
Walter P. Moore and Associates, Inc.
3883 Howard Hughes Parkway, Suite 190
Las Vegas, NV 89169

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EXECUTIVE SUMMARY

Walter P Moore was engaged to perform a detailed structural engineering assessment of the Southern Nevada Health District (SNHD) Public Health Center (PHC) Main Building. This detailed assessment is supplemental to our limited report titled “Visual Assessment of Southern Nevada Health District Public Health Center” dated August 11, 2011. Our detailed assessment identified serious deficiencies which we recommend be addressed. In our visual assessment, we believed the PHC building had a compromised lateral system due to observable deterioration of the building elements. Upon further consideration as a result of our detailed assessment, we have concluded the building does not have a complete lateral system. The structural deterioration which has occurred is a direct result of the lack of the building being able to transfer the lateral loads appropriately and this deterioration of the structure will continue to progress.

The building does not have an engineered diaphragm. A lateral diaphragm is an essential element in providing support for structural elements subject to wind or seismic elements and distributing those loads to the lateral force resisting elements. A diaphragm is typically created by floor or roof decks, floor slabs or engineered bracing. In the case of the PHC building, it has relied on a gypsum panel system as its lateral diaphragm. This system was apparently not engineered as, nor was it intended to be constructed as, a suitable diaphragm. It was never adequate for and is increasingly failing in that role for the structure.

The attachments of roof joists to the bearing and non-bearing masonry walls are also inadequate. The current building code, as well as the original 1961 UBC under which the building was designed, requires concrete or masonry walls to be anchored at all floors and roofs which provide lateral support using connections capable of resisting a minimum force of 200 pounds per linear foot. Such anchorage is critical to the stability of these walls.

As a result of these two deficiencies, the building does not have an adequate lateral load resisting system. The PHC building exhibits numerous signs of structural distress and in our view it appears to be unsafe and should not be occupied further until these inadequacies are addressed.

These structural deficiencies could can be repaired. The repairs required to correct these existing conditions are extensive and it may not be financially feasible to implement. There is also strong evidence of sulfate deterioration of the masonry walls and foundations. This has been reported by the third party testing described in our August 2011 report and in the retrofit design by R2H engineers in 2008. Any major new reconstruction may also require replacement of these walls and foundations.

To understand how the PHC building has been operating without a diaphragm, we performed assessment calculations of the latent stability and lateral capacity of the structure considering non-traditional and non-engineered load paths. This analysis of what level of lateral resistance could be provided by these elements found:

1. The masonry walls could only resist a wind load of approximately 35 MPH to 50 MPH without support from a roof diaphragm. This is considerably less than the code requirement for Clark County of 90 MPH as specified in American Society of Civil Engineers (ASCE) 7-10. The original 1961 Uniform Building Code (UBC) required the structure to be designed for 15 PSF which equates to slightly more than 90 MPH.
2. The foundations were not designed to support the masonry walls for the behavior listed in item 1, but they could be providing a degree of latent strength that has helped the building survive the loads subjected to the building during its life. This latent strength could be provided through stressing the soils beyond the original design capacities and through assistance of resisting elements such as the slab on grade.
3. The building’s non-structural components such as interior walls and ceilings may have also likely contributed to the stiffness and stability of the structure, but these items are not recognized to provide support for lateral forces by the building

code. However, these secondary elements are only unintentional load paths that have not been designed nor detailed for resisting the building lateral loads and likely lack the required capacity to provide reliable structural performance. As these elements are not intended to act as structural elements, their capacity to do so can deteriorate over time and they cannot be relied upon.

4. The ongoing deterioration throughout the building makes it clear that this building is not performing under its current service loads. The ongoing deterioration is also likely further compromising the structural strength and stability. We conclude that it is unsafe and unwise to base future decisions about the PHC building on the fact that it has survived for nearly 50 years.

Finally, we recommend that the occupancy of the building should cease until the structural deficiencies can be addressed. The City of Las Vegas Department of Building and Safety should be advised and consulted in this matter. If desired, we or another structural engineering firm can be consulted to develop conceptual solutions to repair these deficiencies. It may be, however, that these repairs are not economically viable.

INTRODUCTION

Walter P Moore was retained to provide a detailed lateral assessment of the Southern Nevada Health District (SNHD) Public Health Center (PHC) main building located at 625 Shadow Lane in Las Vegas, Nevada. This detailed assessment was supplemental to our earlier visual assessment performed in the summer of 2011. Our previous observations and recommendations are contained in the Walter P Moore report "Visual Assessment of Southern Nevada Health District Public Health Center" dated August 10, 2011.

Scope of Work

Walter P Moore's scope of services was to provide detailed structural assessment of the PHC main building to identify deficiencies with the structure and better understand the deficiencies outlined in our report on the visual assessment. Our scope of work was as follows:

1. Survey the existing diaphragm to identify the extent, location and level of deterioration.
2. Perform a comparative analysis of the calculated existing diaphragm performance against the original design and original code requirements.
3. Define an updated risk assessment of the wind and seismic resistance of the building diaphragm limited to the diaphragm capacity only.

Walter P Moore fulfilled these scope items by performing the following:

1. We outlined limited destructive investigations to be performed by a third party contractor. These investigations locations were targeted to obtain information that was not provided on the original construction documents provided to us by the owner.
2. We performed a thorough review of the provided construction document. This detailed review was limited to the lateral structural systems and load paths.
3. We performed a Tier 1 and Tier 2 evaluation of the PHC main building was performed per the ASCE "Seismic Evaluation of Existing Buildings" standard 31-03.

4. We performed structural modeling and calculations of the lateral load carrying elements to determine their as constructed load carrying capacity.
5. We contacted the manufacturer of the 2” gypsum board roof panels, USG, USG field personnel and other sources to obtain load capacity information regarding the roof panels.

BACKGROUND

The information below was obtained from visual observations, discussions with SNHD personnel, partial construction documents, limited destructive investigations and discussions with sources familiar with the gypsum roof system. Some of this information is duplicated from our August 10, 2011 report.

The PHC main building is located at 625 Shadow Lane in Las Vegas, Nevada. The building property, per the Clark County Seismic Maps, is located in Seismic Site Class C. This results in a Seismic Design Category of C per the ASCE Standard 7-05. The basic wind speed required per ASCE 7-05 is 90 MPH and the Exposure Category is B.

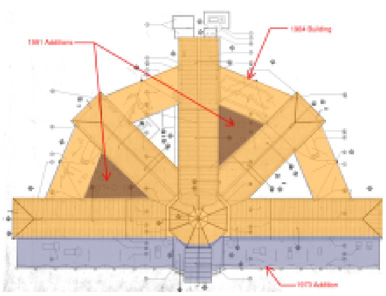
Figure 1 - Overall PHC Main Building Roof Plan

The original PHC building was constructed in 1964 (Figure 1 and Photo 1). It is mostly one story with a small mezzanine currently used as a maintenance workshop. The total building area was approximately 46,000 SF at the time of construction. The structural system was concrete masonry unit (CMU) walls and shear walls, which are noted to be solid grouted "typical" on the partial set of plans provided to us. Roof members consisted of steel joists at 4 feet on center supported directly on the CMU walls. The roof deck consists of a "2" USG metal edge roof deck" (Photo 2 and Photo 3. USG was determined to refer to United States Gypsum Corporation, a major manufacturer of gypsum board products which is still in existence. Foundations were indicated as concrete spread foundations although none of these were visible. The slab on grade is indicated to be 4" of reinforced concrete on a base course.

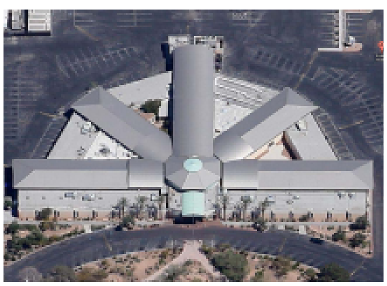
Photo 1 – PHC Main Building Aerial View


Photo 2 – Joint between Roof Panels

In 1973 the building was expanded to the west with the addition of approximately 12,500 SF of new office space and a new vestibule at the main entrance. This addition was constructed from precast concrete panels for the exterior walls. The construction documents note the roof framing to be steel decking on steel joists supported by wide flange girders. However, visual observations clearly identify the roof system to be TJL type wood joists with plywood roof sheathing supported on glulam girders. It is unknown why the as-built condition does not match the existing documents. It may be that the construction documents were revised and the revised sheets no longer exist or it may be that the


Photo 3 – Gypsum Panel Roof

construction documents were never revised. It is also not known which systems were part of the final permit. The slab on grade is noted to be 4" reinforced concrete. Neither the 1964 documents or the 1973 addition were sealed by a registered engineer of record. The 1964 documents, which bear a City of Las Vegas Building Department stamp, are not sealed. The 1973 documents are sealed, but by a registrered architect. It is assumed that this conformed to the state regulations for sealing of documents at that time.

Photo 4 – Underneath over-framed roof

In 1991 two of the triangular courtyards were enclosed, one for an administration area and one for a nurses area (Figure 1). The framing of the roof enclosure was steel deck on steel joists supported on steel angle ledgers. The supporting masonry walls were all original walls from the 1964 construction. The slab on grade construction is 4" reinforced concrete. These drawings are also not sealed but are labeled "as-built". No reference was found to a structural engineer.

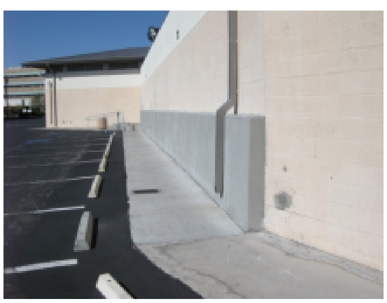

Photo 5 – Concrete buttress wall

The final addition was constructed in 1997. The title of these documents was "Clark County Health District Remodel". No major structural changes to the building foot print appeared to have been made in this remodel. There was a major addition of an over-framed roof and mechanical, electrical and plumbing upgrades (Photo 4). The extent of the over-framed roof cover the "spokes" of the building. The over-framed roof was constructed of standing seam steel deck over structural steel deck on steel trusses. The steel trusses were directly supported on the existing masonry walls. The documents for this addition did include separate sheets for each consultant and the structural documents do list the name of a designated structural engineer. There were no improvements to the existing foundations or lateral system made to accommodate the over-framed roof. Additionally, the over-framed roof was not detailed to allow it to supplement the capacity of the original building's lateral system, including the roof diaphragm.

It is understood by us that a structural engineer was consulted within the past four years to perform an assessment of the cracking observed on the exterior masonry walls. That engineer, R2H, found the masonry deterioration was likely due to sulfate deterioration at the base of the masonry walls and recommended the addition of concrete buttress wall along certain areas of the exterior of the building (Photo 5). Their recommendations provided a short term and long term option. Only the short term option was constructed. The short term option included only partial installation of the concrete buttress walls along the worst of the damaged walls. The constructed option was predicted to extend the life of the damaged walls three years by the engineer. It is our understanding the existing foundations were utilized where the buttress walls were added and that the foundations were not widened.

SITE VISITS

Walter P Moore conducted site visits on March 20, March 22 and March 31, 2011. We were met on all dates by representatives of the SNHD who provided access to both the public and non-public areas. Access was provided to the exposed roof and the interior of the over-framed roof. On March 22 and March 31 we were also met by representatives of a construction company, which performed the destructive investigations.

Walter P Moore made several visits to the PHC site to investigate the components of the assumed lateral system. On March 20 we performed a general investigation of the roof panels and masonry wall attachments. The roof connection to the masonry walls could only be observed from below as the membrane roofing covered the joint from above. Our investigation found no visible connection between the masonry non-bearing walls and the roof system. In some areas, a steel bar through the seat of the steel joist appeared to be connecting the steel joists to the bearing walls, but it was determined closer examination would be necessary for the nature of this connection. No other strap ties, screws, anchors, ledgers, etc could be observed between the steel joist seat bearings, which are spaced at 4 foot on center, and the masonry walls. Voids were observed beneath some joist seats where grout had pulled away from the seat bearings. No other seat support was visible other than the portion supported on the masonry face shells. It is assumed this condition has existing since the original construction.

As a result of our observations on March 20, we recommended some limited destructive investigation be performed to find out more regarding the joist seat connection, masonry wall attachment and gypsum panel attachment. A general contractor was engaged to perform the limited destructive investigation and Walter P Moore provided a plan of the areas to be investigated. As we only possessed incomplete construction documents, and since access and obstructions could not be well determined prior to the site work, a member of Walter P Moore’s staff was on hand during the destructive investigation work to provide guidance. The destructive investigation took place in the early morning hours of March 22 and on the morning of March 31.

Our field investigation and the destructive investigations have determined the following:

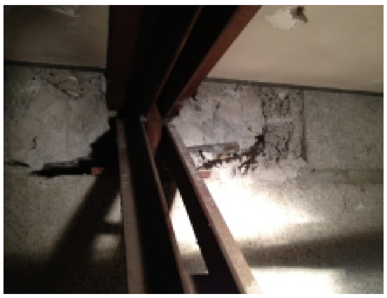
Photo 6 – Steel joist seat.


Photo 7 – Close up of the joist seat.

1. No connection was provided between the top of the non-bearing masonry exterior walls and the roof system. This was consistent with the 1964 construction documents, which do not provide a connection detail. The only mechanical connections observed at all were anchor points between the joist bridging and the masonry. These appear to be failing and are not intended to brace the masonry walls.
2. The only connection between the roof framing and the masonry bearing walls was via 1 foot long steel bars inserted through holes drilled in the joist seats (Photo 6 and Photo 7). The pockets for the joist seats were measured to be 8 inches wide, leaving approximately 1.5 to 2 inches of bar embedment into the grouted masonry on either side of the joist pocket. Additionally, no joist bearing plate or attachment bolts were found.
3. We directed the contractor to remove a section of gypsum roof panels. No screw or bolted attachment was observed between the panels and the joists or from the panels to each other. This finding made surveying and testing of the gypsum panels unnecessary as transfer of lateral loads between these elements is not possible. Our research, as described in the Discussion section below, indicated that this was representative of the whole gypsum panel system.
4. There is no connection between the top of the non-bearing masonry interior walls and the other elements of the structure sufficient to brace the top of the wall. These walls were observed to be bracing other elements such as joist bridging and hallway ceilings.

DISCUSSION


Photo 8 – Gap between the roof panels and the masonry wall.


Photo 9 – Roof panels viewed from above with membrane removed.

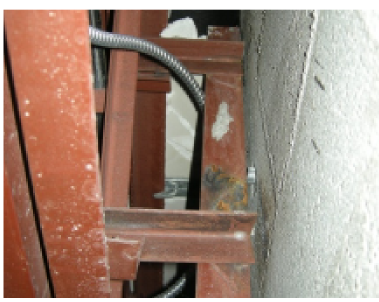
Photo 10 – Joist bridging anchor has separated from the masonry wall.


Photo 11 – Metal edged USG gypsum roof panel.

The PHC main building is being used as a public health clinic. A thorough discussion of our visual assessment and of notable features of the PHC main building can be found in the Walter P Moore report dated August 10, 2011. This detailed structural assessment was limited to analysis of the lateral system and the lateral load carrying members pursuant to our previous recommendations. A discussion of our findings and conclusions follows below.

We reviewed the partial original construction documents provided last summer to Walter P Moore by the SNHD. We determined these drawings are a partial set as the sheets are listed as XX of 50. Only 36 sheets of the 50 have apparently survived to date. However, some sheets have no title block and may be duplicates or parts of other sets. Based on the sheets which do have title blocks it appears that approximately half of the 50 sheet set can be accounted for. The last 13 pages in particular are missing, but these are often mechanical, electrical and plumbing drawings which have no relevance to the current analysis.

Our review of the surviving drawings found no mechanical connection was detailed between the masonry bearing walls and non-bearing walls and the roof system. Our field investigations performed during March confirmed that this was, in fact, the case (Photo 8). From below a clear gap could be seen between the edge of the gypsum panels and the masonry walls. We confirmed this in several areas by removing a section of the roofing membrane to observe the condition from the topside as well (Photo 9). It appears that the gap between the masonry walls and the gypsum panels is recent as the joist bridging anchors also shows signs of having separated from the wall (Photo 10).

We found the only roof system used on the original 1964 building was comprised of gypsum metal edged gypsum wall planks. The existing drawings do not show information regarding the attachment of the gypsum planks to the steel joists, the attachment of the gypsum planks to each other or of the lateral or gravity load carrying capacity of the gypsum planks. A section of the roof panels was removed by the contractor during the destructive investigation at our direction (Photo 11).


Photo 12 – A section of metal edged roof panel with attachment clip.

Photo 13 – Panel tabs located intermittently connecting the roof panels to the steel joists.


Photo 14 – Panel tab connection seen from below the joist top chord.


Photo 15 – Roof level with a panel

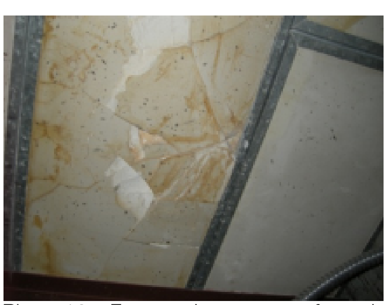
Photo 16 – Fractured gypsum roof panel.

The roof panels consist of poured gypsum enclosed by a light gauge steel channel around the perimeter (Photo 12). The metal channel appeared to be connected to the steel joists using small clips, but these were only intermittent (Photo 13). These panels were easily removed as there was not much holding them to the other panels or to the joists (Photo 15). The mechanical attachment of the panels to the joists was minimal or absent (Photo 14).

As we already documented in our August 10 report, many of the gypsum panels were observed to be cracked or broken. The degree to which these panels were broken varied from minor to major fractures (Photo 16 and Photo 17) to complete failure (Photo 18, Photo 19, Photo 20 and Photo 21). Our survey found more than half of the accessible panels were fractured. Some panels were not accessible due to solid gypboard ceilings or other access restrictions. Our August 10 report identified these fractures as a hazard to maintenance personnel needing to access the numerous and heavy rooftop mechanical units (Photo 22), piping or multiple other items located on the roof. Plywood has since been placed over many of the most frequently accessed areas in order to provide a degree of safety for the maintenance personnel.

We contacted USG last year to see if more technical information could be found regarding the roof panels. At that time we were told by their technical support that USG had never manufactured such a product. No information could be found either on the USG website or in a limited search of technical journals or of the various standard writing organizations. We suspected no one with direct technical knowledge of the product was still employed by USG if indeed they did manufacture the panels.

In view of our discoveries during the field investigations, we made another effort with USG. Our findings were beginning to indicate this may be a system without any diaphragm capacity at all, instead of only a reduced capacity, due to the numerous field deficiencies found and loose attachment to the joists. Once again a technical support representative stated that USG had never manufactured a product similar to what was described. However, they were able to refer us to a field representative, who referred us to a second representative, Jennifer Link-Raschko. Once


Photo 17 – Gypsum roof panels showing major fractures.


Photo 18 – Failed roof panel.


Photo 19 – Failed roof panel.


Photo 20 – Failed roof panel.


Photo 21 – Failed roof panel.

contacted, Ms Link-Raschko provided some information on a poured gypsum system. When we discussed with the field representative that this was not the system on the PHC, she stated it was the only gypsum system USG manufactured. She did provide the contact of an architect, Bruce Poteet, located in Charlotte, SC who she said may be able to provide more assistance. We contacted Mr. Poteet and he was very familiar with the gypsum plank system. He is retired from USG but has an arrangement with USG to continue to answer technical questions for them.

Mr. Poteet said USG has not manufactured these panels for over thirty years. However, they are still produced by another manufacturer, mainly as replacement panels for systems such as the one on the PHC structure. This outside manufacturer had bought the rights to the system from USG many years ago. Mr. Poteet also stated that the panels were often only placed on top of the supporting members or connected by small gauge metal clips nailed attached to the panels via nails. He said these clips were easily knocked off during construction and in many cases never installed. Most importantly, Mr. Poteet, stated that the gypsum panels have no diaphragm capacity and cannot be considered to be part of a lateral system. They are also only minimally connected to the supporting members and should not be considered to provide compression flange bracing or other support. We requested Mr. Poteet provide a letter regarding this information. His letter is attached in Appendix B below.

The discussion with Mr. Poteet confirmed many of the items we suspected based on observations made during our March site visits and the results of the destructive investigations. The panels were only minimally attached to the steel joists at best and certainly not sufficiently to transfer diaphragm loads or brace the joist top chord. The panel clips were often not regularly installed and many times left out entirely. There was no connection between the panels or to the exterior walls. In short the panels were laid over the joists and held in place through friction or through confinement of the perimeter walls.

In our August 2011 Visual Observation Report we were suspect of the gypsum panel system due to the numerous fractures, age and apparent poor performance of the system. We recommended at that time the replacement of the diaphragm system due to these defects but we still expected to find the panels were minimally connected together sufficient to provide some level of diaphragm capacity and support for the walls. In our opinion it is alarming that this building has no functioning diaphragm system and it apparently has never had one.

A diaphragm is an essential part of the lateral system of most buildings. In a typical one story shear wall structure such as this one, lateral loads acting perpendicular to the structure are transferred to both the foundations and the diaphragm by the exterior walls facing the lateral load (Diagram 1).

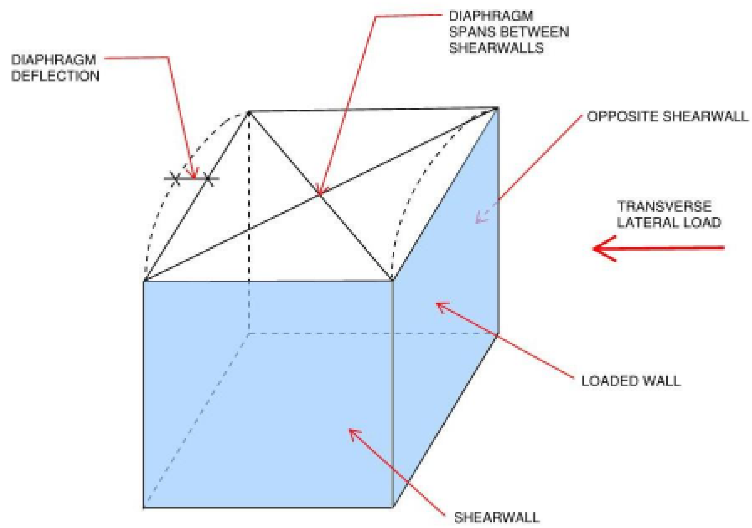

Diagram 1 – Elements of a lateral force resisting system

The diaphragm resists the lateral load and provides support to the exterior wall by acting as a horizontal "deep beam". The diaphragm then transfers the lateral load to the resisting shear walls. Diaphragms can consist of many materials including plywood, steel deck, concrete, concrete on steel deck and cross bracing. The absence of a roof diaphragm means that the exterior walls have no support at the top and must cantileverl from their foundations. The lack of attachment between the walls and the roof are particularly troubling at the PHC as the original design seems to assume the presence of a roof diaphragm. With the lack of a diaphragm system a substantial collapse of portions of the building could occur.


Photo 22 – Typical rooftop mechanical unit.


Photo 24 – Interior wall with no attachment at the top of the wall.


Photo 23 – Interior wall without attachment at the top of the wall.

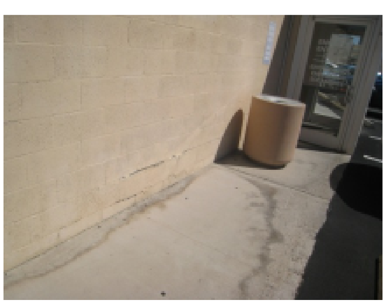
Photo 25 – Wall fracture.


Photo 26 – Fracture at wall base.


Photo 27 – Fracture at wall base.


Photo 28 – Fracture at wall base.


Photo 29 – Fracture on the interior side of an exterior wall.

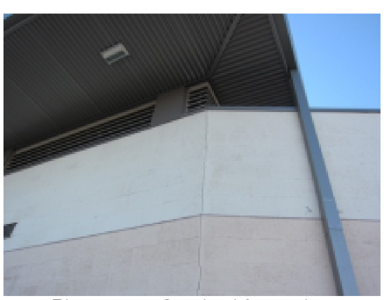
Photo 30 – Cracked formed at intersecting walls.

We observed other items which could contribute to such a collapse. Interior walls were not supported at the top of wall (Photo 23 and Photo 24). This is a function normally performed by a roof diaphragm. Interior walls often extend to the deck level or are braced to roof or floor levels. We observed the interior walls at the PHC were serving as anchor points for the roof joist bridging, a function they would not be able to perform well without being braced themselves. The current building code, as well as the original 1961 UBC under which the building was designed, requires a positive mechanical attachment of masonry walls at floor and roof levels. Additionally, as previously mentioned, the gypsum panel system does not provide adequate bracing of the top flange of the joists. Joists ability to carry gravity loads is dependent on the top flange being braced. Further deterioration of the minimal bracing the gypsum panel system is providing could lead to failure of joists under gravity loads.

As we noted in our August report, the exterior walls show evidence of deterioration, particularly at the base. These cracks are particularly evident at approximately 8” above the exterior sidewalk slab (Photo 25 to Photo 28), but fractures were observed throughout the exterior walls (Photo 29). We observed the exterior walls to have separated at intersecting corners (Photo 30 and Photo 31).

Based on our findings, we performed a structural analysis to determine the capacity of the existing to carry wind loads. Our findings are as follows:

For exterior non-bearing, structural walls, based on the design information in the original construction drawings, without being braced at the top of the wall by an adequate connection to a diaphragm, we have calculated the masonry wall to have a capacity of 4 PSF acting as a cantilever. Based on ASCE 7, this would be equivalent to an approximately 50 MPH. This is considerably less than the 90 MPH wind loads required by the current building code or the 15 PSF required by the original building code (1961 UBC). The 2' wide strip foundations under these walls, however, are only adequate for approximately 2 PSF wind pressure (35 MPH) against overturning. The soil bearing pressures produced are over 3500 PSF which approach the likely ultimate capacity of the soils, but likely exceed their


Photo 31 – Cracked at joint of intersecting walls.


Photo 32 – Corroded lateral attachment.


Photo 33 – Corroded lateral attachment.

allowable bearing capacity. Soils stressed to their ultimate capacity could result in sudden, extreme failure.

For exterior load-bearing structural walls, without being braced at the top by a connection to a roof diaphragm or even an adequate joist connection, the capacity of the masonry wall is also approximately 4 PSF acting as a cantilever. The strip foundations under these walls, however, are only adequate for approximately 3 PSF wind pressure (45 MPH) against overturning. The soil bearing pressures produced, however, are over 5000 PSF which likely exceeds the ultimate capacity of the soils.

The steel joists of the original 1964 building are attached to the supporting masonry walls by means of a 12" bar which extends only a couple of inches beyond the seat pocket. Some of these connections are spalled or damaged and some joists had no positive attachment to the walls at all. We observed the steel joists of the original 1964 building in several locations to have grout pockets or inadequately grouted cells beneath the joist bearing condition. These joists are supported only by the face shells of the masonry bearing walls. These wall attachments are inadequate and should be repaired. This is especially important due to the lack of a lateral system.

Our calculations found the building foundations are not capable of carrying more than minimal lateral load. The typical spread foundation for the masonry walls is two foot wide and ten inches thick. Load bearing wall foundations are loaded to full capacity with gravity loads alone. Several additions have been constructed without any increase in foundation size. We also have found evidence of sulfate deterioration in the masonry walls and possibly in the foundations. This was also noted by the consultant engineer, R2H, hired four years ago. Little was known about sulfate deterioration of concrete, a common issue in the valley, when the building was constructed.

Finally, we noted in our previous report corrosion was observed in the expansion bolts and plates of the lateral anchorage for the over framed roof constructed in 1997 (Photo 32 and Photo 33). Given the exposure of these elements to the effects of weather, stainless steel should have been used. As we also noted, several of the bolts were observed to have failed. Some of the worst of these conditions were repaired in December 2011.

That the structure has been able to stand fifty years in this condition is likely due to several factors. The building is only one story and the story height is not extremely high and the masonry walls are reinforced and fully grouted giving them some limited cantilevered wall capacity as previously noted. The building has an unusual shape in which intersecting exterior walls may prevent others from overturning. The building has a large number of non-structural interior walls, which in addition to the ceiling, provide some level of bracing to the structural walls. However, having not been specifically designed or detailed to function as structural elements, these elements are not reliable as structural elements and are not permitted by code to be considered in the structural capacity of the building. In addition, while it does not appear that it was intended to do so, the over-framed may be acting to nominally tie some of the walls together. Even considering these factors, this does not represent a complete structural system and the performance of such elements for lateral load resistance is not predictable. Finally, and perhaps most importantly, these elements were not designed to carry loads of the necessary magnitude and loading them in this manner, the capacity of these secondary members will deteriorate over time, reducing the real capacity of the structural elements to carry loads.

The masonry walls show deterioration at the base, exactly as would be expected for walls performing as cantilevers. The opening of the joints observed at intersecting corners is also consistent with this behavior as the opening would be expected to be wider at the top than at the base. However, the deterioration increases the likelihood of a critical failure and our concern is that a small localized failure will quickly progress. We recommend the building be provided with a functioning lateral system and the masonry walls be provided with positive structural connection to the lateral diaphragm. Until such time as these repairs are made we recommend that occupancy of the facility cease. Without correction the structure does not even meet the requirements of Tier 1 minimum requirements per ASCE 31-03. ASCE 31-03 is the standard for reviewing existing structures for seismic loads.

The design of the three additions to the building did not address the issue of the missing lateral system. We recommend discussions with the building department in order to determine under what conditions the building could or should be occupied. It is our opinion the building department would not likely allow the building to be occupied in its current state. Building repairs, including those to the lateral system will likely require the entire building to be compliant with current code.

While the structural deficiencies observed can be repaired, the repairs will be extensive and it may not be financially feasible to do so. In addition to addition of an adequate structural diaphragm and repair of cracked structural walls, there is also strong evidence of sulfate deterioration of the masonry walls and foundations. This has been reported by the third party testing described in our August 2011 report and in the retrofit design by R2H engineers in 2008. Any major new reconstruction may require replacement of these walls and foundations. These extensive repairs are likely to impact other non-structural items such as roof waterproofing and roof top mechanical equipment which could further increase repair costs. The development of these repairs is beyond the scope of this report.

LIMITATIONS

This report has been prepared at the request of PGAL, LLC to perform a detailed structural assessment of the SNHD PHC.

Walter P Moore offers no warranty regarding the condition of concealed construction or subsurface conditions beyond what was revealed in our review. Any comments regarding concealed construction or subsurface conditions are our professional opinion, based on engineering experience and judgment, and derived in accordance with current standard of care and professional practice.

Various other non-structural, cosmetic and structural damage unrelated to this assessment may have been observed throughout the structure, some of which are discussed in general in this report. However, a detailed inventory of all cosmetic, nonstructural and structural damage was beyond the scope of our assessment. Comments in this report are not intended to be comprehensive but are representative of observed conditions. A peer review or administrative review for code conformance was beyond the scope of this report. Repair recommendations discussed herein are conceptual and will require additional engineering design for implementation.

We have made every effort to reasonably present the various areas of concern identified during our site visits. If there are perceived omissions or misstatements in this report regarding the observations made, we ask that they be brought to our attention as soon as possible so that we have the opportunity to fully address them in a timely manner.

This report has been prepared on behalf of and for the exclusive use of PGAL, LLC and the Southern Nevada Health District. This report and the discussion contained herein shall not, in whole or in part, be disseminated or conveyed to any other party or used or relied upon by any other party, in whole or in part, without prior written consent.

APPENDIX A – FIGURES AND PHOTOGRAPHS

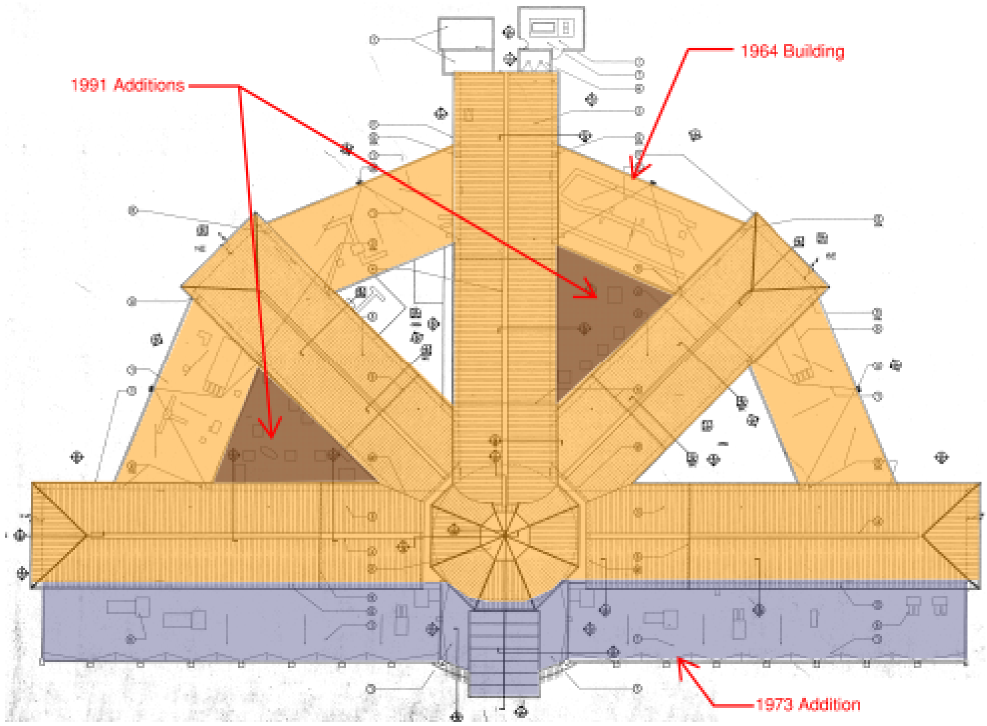

Figure 1 - Overall PHC Main Building Roof Plan

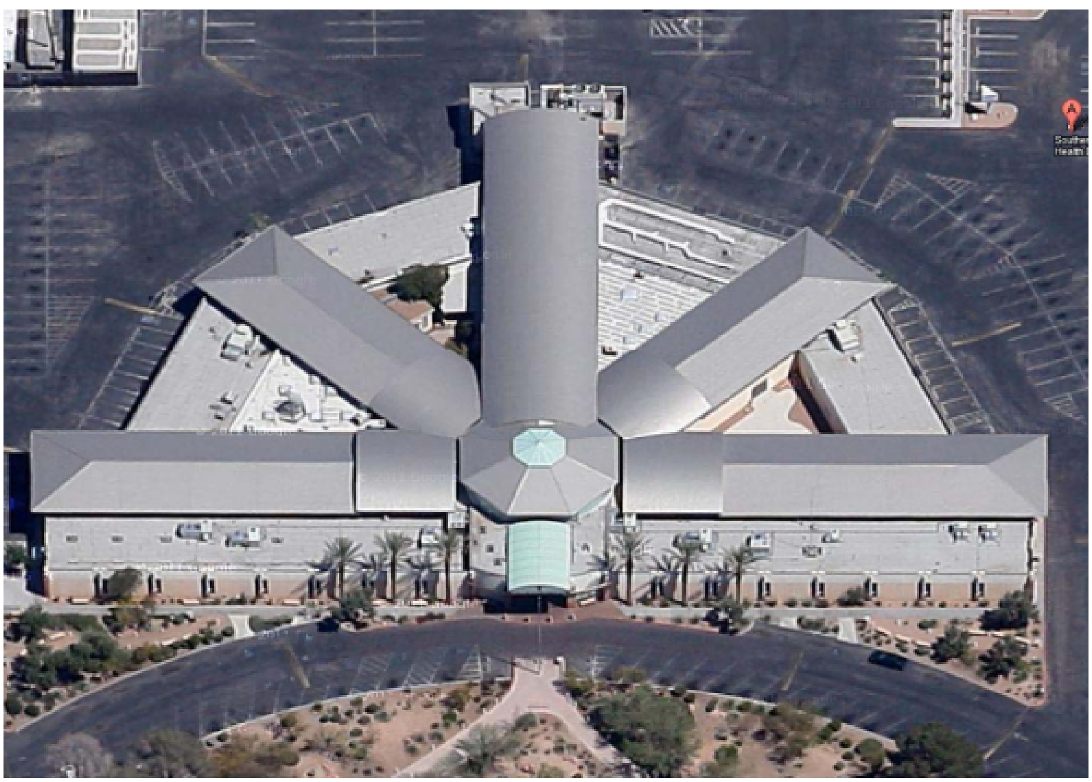

Figure 2 – Aerial View

Photo 2 – Joint between Roof Panels

Photo 3 – Gypsum Panel Roof

Photo 4 – Underneath over-framed roof

Photo 5 – Concrete buttress wall

Photo 6 – Steel joist seat.

Photo 7 – Close up of the joist seat.

Photo 8 – Gap between the roof panels and the masonry wall.

Photo 9 – Roof panels viewed from above with membrane removed.

Photo 10 – Joist bridging anchor has separated from the masonry wall.

Photo 11 – Metal edged USG gypsum roof panel.

Photo 12 – A section of metal edged roof panel with attachment clip.

Photo 13 – Panel tabs located intermittently connecting the roof panels to the steel joists.

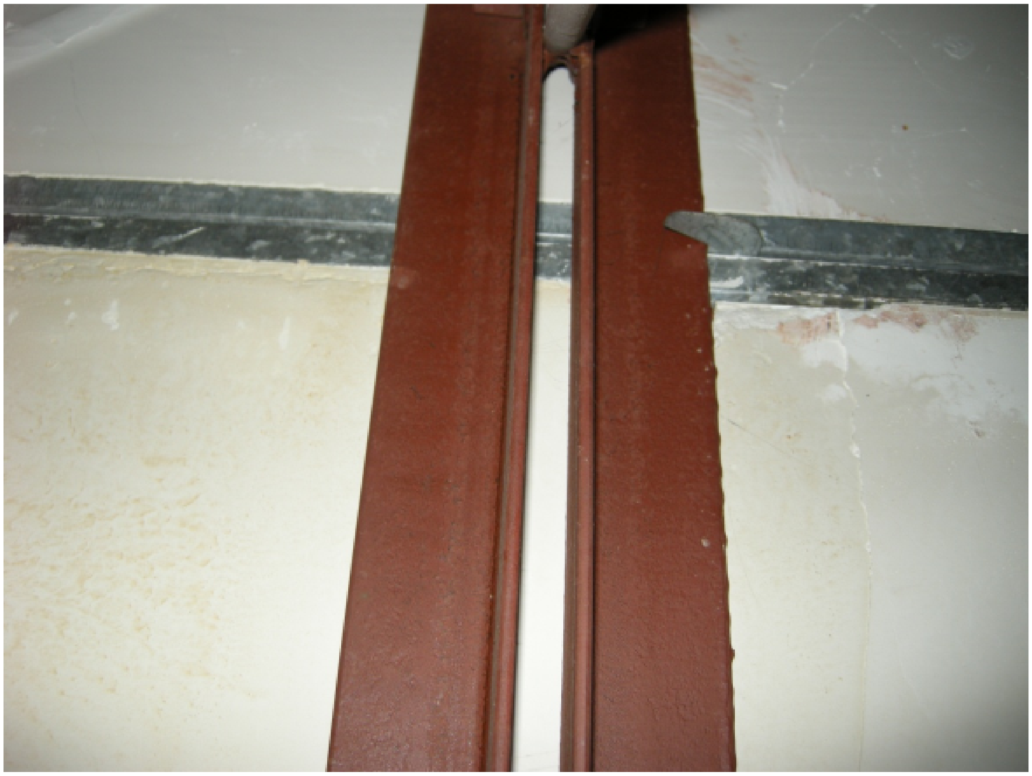

Photo 14 – Panel tab connection seen from below the joist top chord.

Photo 15 – Roof level with a panel section removed.

Photo 16 – Fractured gypsum roof panel.

Photo 17 – Gypsum roof panels showing major fractures.

Photo 18 – Failed roof panel.

Photo 19 – Failed roof panel.

Photo 20 – Failed roof panel.

Photo 21 – Failed roof panel.

Photo 22 – Typical rooftop mechanical unit.

Photo 23 – Interior wall with no attachment at the top of the wall.

Photo 24 – Interior wall without attachment at the top of the wall.

Photo 25 – Wall fracture.


Photo 26 – Fracture at wall base.

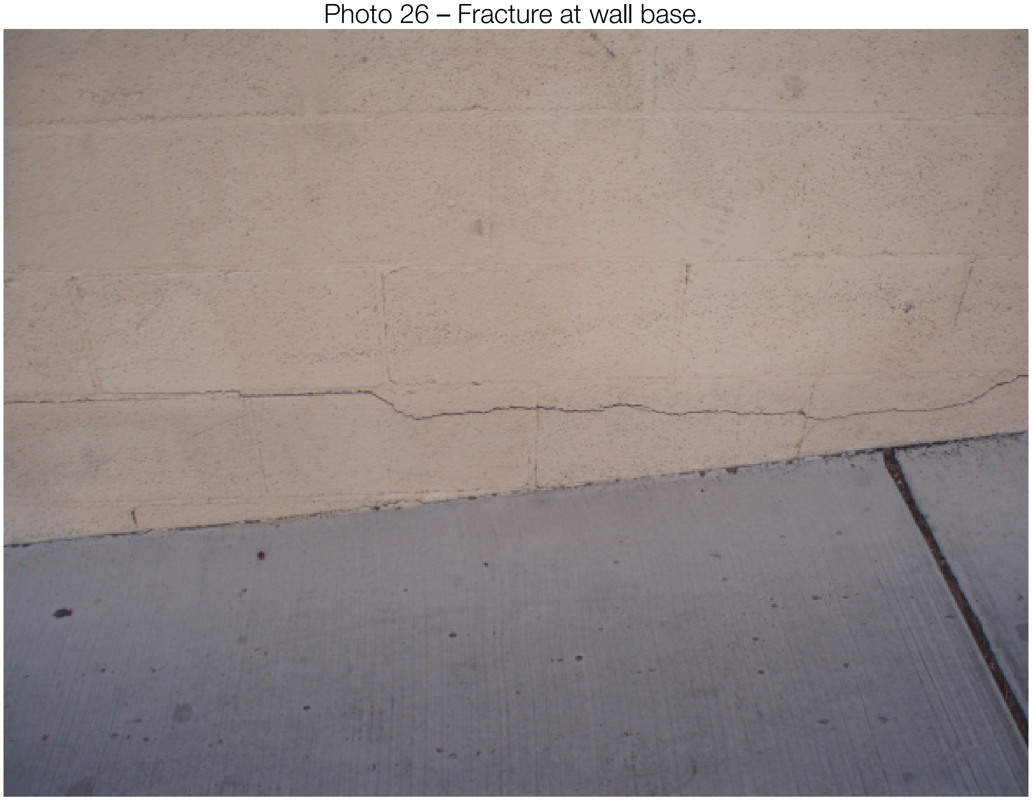
Photo 27 – Fracture at wall base.

Photo 28 – Fracture at wall base.

Photo 29 – Fracture on the interior side of an exterior wall.

Photo 30 – Cracked formed at intersecting walls.

Photo 31 – Cracked at joint of intersecting walls.

Photo 32 – Corroded lateral attachment.

Photo 33 – Corroded lateral attachment.

APPENDIX B

THE POTEET GROUP

2300 SARDIS ROAD NORTH - SUITE O
CHARLOTTE , NORTH CAROLINA 28227
PHONE 704.321.0266 FAX 704.321.0286

thepoteetgroup.com

March 27, 2012

Kenneth Zarembski, S.E.
Walter P. Moore Engineers
3883 Howard Hughes Parkway, Suite 190
Las Vegas, NV 89169

Re: USG Metal Edged Gypsum Plank

Dear Kenneth,

This letter is to advise you that we are the representatives for the gypsum roof deck products and systems manufactured by the United States Gypsum Company. The production of Metal Edge Gypsum Plank, as manufactured by USG, was discontinued over 30 years ago. The rights to the material were conveyed to another company, and a similar product, with a Portland cement core, is now produced by MidCon Products Company. This material may be used in all UL Rated Designs in lieu of the gypsum planks.

The Gypsum Planks were secured to the structural steel framing by using metal clips which provided for lateral uplift resistance when used as recommended. They did not however, provide for horizontal shear load transfer.

Please contact me if you have any questions or require additional information.

Very truly yours,

THE POTEET GROUP

Bruce C. Poteet, AIA, CSI