INTEGRATED ENVIRONMENTAL MANAGEMENT

SEISMIC AND STRUCTURAL REVIEW OF ADVANCED MEDICAL SYSTEMS LABORATORY FACILITIFS

by
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AN ASSESSMENT

There were five areas addressed in the March 12, 1996, Nuclear Regulatory Commission letter (Wright 1996) concerning structurally distressed areas of the Advanced Medical Systems Laboratory Facility (AMSLF) shown in Figure 1. In the letter's attachment, a list of detailed concerns was provided for each of the five areas. Attachment I provides a response to those concerns. The five major areas of concern and this reviewer's response to them are provided below. Attachment II addresses seismic and tornado issues.

Figure 1. Advanced Medical Systems Laboratory Facility (looking west).

THE FIVE MAJOR NRC CONCERNS AND REVIEWER'S RESPONSE

1. *The depth and extent of cracking, structural impact, and any measures identified as necessary to repair the cracking identified in the load-bearing masonry wall in the 1958 building's southeast corner. Associated distress that could limit the facility's ability to continue to provide protective confinement of the radioactive materials should also be addressed and corrective actions identified as necessary.*

Response: As described in the NRC report, the cracking does exist. From the outside, the cracking appears to stop just short of the area where the second floor slab ties into the common wall brick. From inside the building below the second floor, it is evident that cracking does extend through the wall; however, it is not continuous, i.e., the cracking on the inside of the wall is almost an opposite pattern. From inside the building above the second floor, the east and south walls in the corner show no cracking; however, there are four cracks in the second floor slab that are visible to the human eye. One very large crack is nearest the corner, is at an approximate 45° angle to each wall, has a width of one-half inch, and has a length from wall to wall of about six inches. Based on the width of the crack nearest the corner, it appears that the southeast corner has moved southeast a distance of as much as one-half to three-quarters of an inch. These floor cracks seem to indicate excessive bending moment in the floor slab at this corner, which would be indicative of significant settlement at the corner. Underneath the second floor slab, matching crack patterns were found.

Unfortunately, it is difficult to tell what actually caused cracking at the southeast corner of the building. As noted in the NRC report, the structural support of the 1958 building is a mix of load and non-load bearing masonry and concrete block, reinforced concrete, and steel framing. These materials are not compatible from an aging and expansion point of view. In addition, the stiffness properties of the structure vary from extremely stiff (the test cell and radiography room) to very flexible (the lobby area). If a significant lateral or vertical load were to be applied to this location of the building, a localized corner failure of the building would occur between the first and second floors, while due to the purlin bearing on the east wall at the corner, a much broader area of the roof in the west direction would collapse. However, due to the construction of the building, this reviewer does not believe that such a loading would lead to overall collapse of the building. Such a failure would not cause loss of containment in the radioactive storage area of the garden room, WHUT room, or radiography room. In fact, based on the massive concrete walls, general building collapse would not cause loss of containment. See Attachment I for more detailed discussion.

2. *The depth and extent of cracking, structural impact, and any measures identified as necessary to repair the cracking identified in the 1958 building's north bay of the east masonry filler/ curtain wall. Associated distress, caused by the introduction of moisture and other waterborne contaminants, that could limit the facility's ability to continue to provide protective confinement should also be assessed and corrective actions identified as necessary.*

Response: Upon inspection of this wall's cracking and movement, it appears evident what is occurring. Basically, it is a problem of the second floor in-filled wall moving outward over the lobby area between the steel columns at Column Lines D and F. This movement can clearly be seen from the roof and is causing rotational stresses to be placed on the original corner of the 1958 building as the east-west wall tries to keep the east wall form moving outward. The fact that the 1958 building in-filled wall was placed tightly against the inner side of the outside column flange, and the fact that ties attached the two wythes of facing brick to the 1958 building concrete block, the 1958 building in-filled wall is being pulled outward next to Column F-1, failing the in-filled wall in shear at the edge of the flange. This is not a structural issue, but it is a life-safety issue. See Attachment I for more information on this issue.

The introduction of moisture and other waterborne contaminants has been minimized by the new roofs that were placed over the 1934 and 1958 buildings within the last five years. Minor migration of moisture may occur through small cracks through the building, but such moisture would not have an impact on the containment ability of the facility. A conceivable way for moisture and water contaminants to limit the facility's ability to provide protective containment or confinement could not be postulated. Obviously, if the in-filled wall of the second floor fell out into the street and was left open, serious penetration of moisture and waterborne contaminants could occur. Leaving the area of the failed wall open is an extremely unlikely event, unless the building were to be abandoned.

3. *The precast concrete roof panels that in several areas exhibit corrosion products on the visible surface.*

Response: While evidence of corrosion exists in some areas of visible roof decking, no structural degradation was noted. Thus, while past leakage has caused appearance problems, there appears to have been no structural degradation of the roof decking. See Attachment I for more details.

4. *The second floor concrete slab in the area where it forms the ceiling of the hallway in front of the hot cell and the radiography room, and exhibits the effects of previous fluid penetration through the slab from above.*

Response: Two chases, one for electrical service conduit and one for ventilation, penetrate the second floor from the first floor into the equipment room on the second floor. The equipment room has a 10-inch riser around its perimeter, including the two entrance doors; however, at both the conduit and ventilation chases, the risers are only two inches. In addition, at the ventilation chase, the riser has a 1½-inch deep notch in it. Thus, the maximum fluid that can be contained within the equipment room is about 40 gallons. Therefore, the fluid runs over the ½-inch riser, down the chase onto the false ceiling. The first time a leak of significance occurred, the fluid collected at the false ceiling and held there until the plaster of the false ceiling gave way. This leakage caused no deterioration of the second floor slab's structural strength, and the fluids did not penetrate the concrete floor slab. There is no visible degradation of the floor slab, and it is highly unlikely there has been any degradation. In addition, the main part of the equipment room is over the radiography room where the floor slab is 2-feet thick.

5. *The need to periodically inspect and evaluate the building's ability to perform its defined functions over the utilization period. If a program is deemed appropriate, it should include inspection frequencies and evaluation activities.*

Response: This concern is a management issue and is out of this reviewer's scope of responsibility.

REFERENCES

Wright, G.C. 1996. Letter Report to Mr. Robert Meschter, Advanced Medical Systems, Inc., Cleveland, Ohio, March 12.

ATTACHMENT I

NUCLEAR REGULATORY FACILITY INSPECTION REPORT ON THE ADVANCED MEDICAL SYSTEMS LABORATORY FACILITY

A RESPONSE

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The following provides responses to an itemized list of concerns, as identified by this reviewer, developed from the DETAILS of NRC Report No. 030-16055/95006 (DNMS) for the Advanced Medical Systems Laboratory Facility (AMSLF) (Wright 1996). Each concern is numbered first by section of the report and then by concern. To identify the location of each concern in the NRC Report, the concern is identified by page number, paragraph, and sentence. Thus, the identifier *2.1 CONCERN-3/1/2* is identifying a concern in Section 2, on Page 3 of the report, in Paragraph one of Page 3, and starting with the second sentence of the paragraph. This attachment addresses those concerns having to due with the basic structural integrity of the AMSLF. Seismic and tornado integrity of the AMSLF are discussed in Attachment II of the main report.

1. Persons Contacted

   N/A

2. Purpose and Scope of Inspection

2.1 CONCERN-3/1/2: *The structural integrity of the building facility with areas of contamination, waste storage or source material storage needs to be assured for the expected future time period over which the radioactivity should be controlled...this time period may extend as much as an additional 25 to 30 years or more beyond the current time.*

2.1 RESPONSE: A sound maintenance program can result in such facilities' lifetimes being easily extended an additional 25 to 30 years.

2.2 CONCERN-3/2/1: *On-site inspections ...information related to the facility, the life of which to date has spanned nearly a forty-year period, with portions spanning over 60 years.*

2.2 RESPONSE: Based on two site inspections, the portion of the building built in 1934 as a stand- alone building appears to be in sound structural condition. Although no thorough inspection of the 1935 building was conducted, one inspection also included the attic and no noticeable cracking was found.

3. Background on the Development of the Facility

3.1 CONCERN-4/2/1: *The 1958 design, development and construction...encompassed the integration of a then existing warehouse/industrial building, with masonry load bearing walls and steel trusses as the roof framing steel...* This item was not a concern but a response is needed for clarification.

3.1 RESPONSE: The warehouse/industrial building referred to is not a complete load bearing masonry wall building with steel trusses, although at first glance it may appear to be. The building was constructed in 1934 as rectangular in shape. The east-west walls, the walls in the longer dimension (~100 ft), are non-load bearing masonry in-filled walls between steel-riveted columns that support the steel trusses. Some small amount of roof load between the trusses was originally carried by the east-west masonry wall, but this load was one-half of a purlin load, thus the wall typically would not be considered a load bearing wall. The tops of the east-west walls are basically unsupported in the out-of-plane direction except at the columns. At the columns, the masonry is placed flush with the column webs and a pilaster is built around the column flanges facing the original exterior of the building, with about six inches of brick covering the flanges. The sketch in Figure A-1 demonstrates the construction. Based on research done in the last 10 years (Henderson, et al., 1995), unreinforced masonry in-filled buildings provide excellent seismic resistance in low to moderate seismic zones. Although the 1935 building is not a true in-fill since the upper masonry walls are not confined by beams, the fact that they are in-filled within the floor and columns should provide lateral capacity of the building superior to a typical unreinforced load bearing wall. This is especially true where the old windows were filled in with masonry.

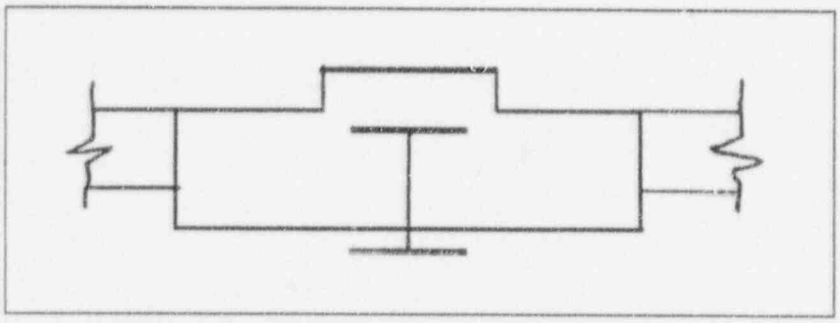

Figure A-1. Masonry pilasters and steel columns.

The north-south walls, the short dimension (~60 ft) of the building, are also constructed in the same fashion as the east-west walls. The only difference is the vertical columns at pilasters do not support any vertical load and the wall itself, between columns, supports one-half of the roof load that the trusses and columns support.

4. Design Basis of Building Facilities

4.1 CONCERN-6/1/1: *All of the loadings identified on the drawings, as noted above, include only vertical gravity loadings...*

4.1 RESPONSE: It would be extremely unlikely that seismic loads would have been included in the design. The BOCA Code of the Building Officials Code Administrators International introduced seismic design as an option in the late 1960s and made it mandatory in the late 1970s. However, in the 1950s and 1960s, the BOCA Code had requirements for wind design, although the city of Cleveland may have not adopted such code provisions. There is no evidence in any of the three buildings that special design features were made for lateral loads. However, the combined steel column and unreinforced masonry load and non-load bearing walls of the 1934 building, the unreinforced load bearing and in-filled steel column and beam walls of the 1958 building, and the 1963 building with Type II AISC column-to-beam connections all have inherent lateral strength for the typical wind and seismic loads of the region. For example, studies and tests on Type II connections used extensively in the 1940s and 1950s (Frye and Morris 1975) have shown they can provide lateral resistance through inherent moment capacity. Moment capacities for Type II connections using six-row fasteners, as used in the 1963 building, can generate moments up to 40,000 ft-lbs.

4.2 CONCERN-6/2/1: *The specifications for materials used in the structural system were, in general, not available...*

4.2 RESPONSE: There is a good chance that the concrete has a compressive strength in excess of 3000 psi since the compressive strength increases with age. If it was poured at 3000 psi, the compressive strength would now be in the neighborhood of 3700 to 4000 psi. Typical tests of such concrete shows the average strength to be 3800 psi. To determine the in situ concrete strength, core samples would have to be taken and then tested in the laboratory. However, based on the structural appearance of the concrete, i.e., there is no evidence of degradation, and the fact that long-term loads, even the seismic load, are not critical to the structural stability of the concrete structure, taking core samples and testing them does not seem warranted at this time.

4.3 CONCERN-6/2/2: *Dwg. F-1 indicated that the concrete for the foundation was to be 3000 psi concrete at 28-days, but no other information was provided on the properties of the reinforcing steel.*

4.3 RESPONSE: Based on the fact that intermediate grade reinforcing steel was commonly used at the time, the reinforcing steel is most likely intermediate grade having a specified yield strength of 40,000 psi. The extensive use of intermediate grade reinforcing steel began in the 1940s. For example, intermediate grade steel was used extensively in the 1940s, 1950s, and 1960s for the construction of high-level radioactive waste storage tanks at the Department of Energy's Hanford site in nearby Richland, Washington. Like concrete, the strength of the steel is typically higher than specified. Intermediate grade steel's mean yield strength is 49,000 psi. Its specified ultimate strength is 70,000 psi while its mean ultimate strength is 78,000 psi. Sample bars could be obtained for testing; however, as with the concrete strength, determining the in situ rebar yield and ultimate strengths does not seem warranted at this time.

5. Field Observations and Structural Evaluation

5.1 CONCERN-6/4/2: *In this area there is evidence of considerable amounts of water or other fluid apparently having penetrated on the second floor of the facility...*

5.1 RESPONSE: Two chases, one for electrical service conduit and one for ventilation, penetrate the second floor from the first floor into the equipment room on the second floor. The equipment room has a 10-inch riser around its perimeter, including the two doors, giving the impression that approximately 850 gallons of fluid would have to be spilled for a leak

outside the area to occur. This may have been the original intent; however, at both the conduit and ventilation chases, the risers are only two inches. In addition, at the ventilation chase, the riser has a 1½-inch notch in it. Thus, the maximum fluid that can be contained within the equipment is about 40 gallons. Therefore, the fluid runs over the ½-inch riser, down the chase onto the false ceiling. The first time a leak of significance occurred, the fluid collected in the false ceiling and was held there until the plaster of the false ceiling gave way as shown in Figure A-2.

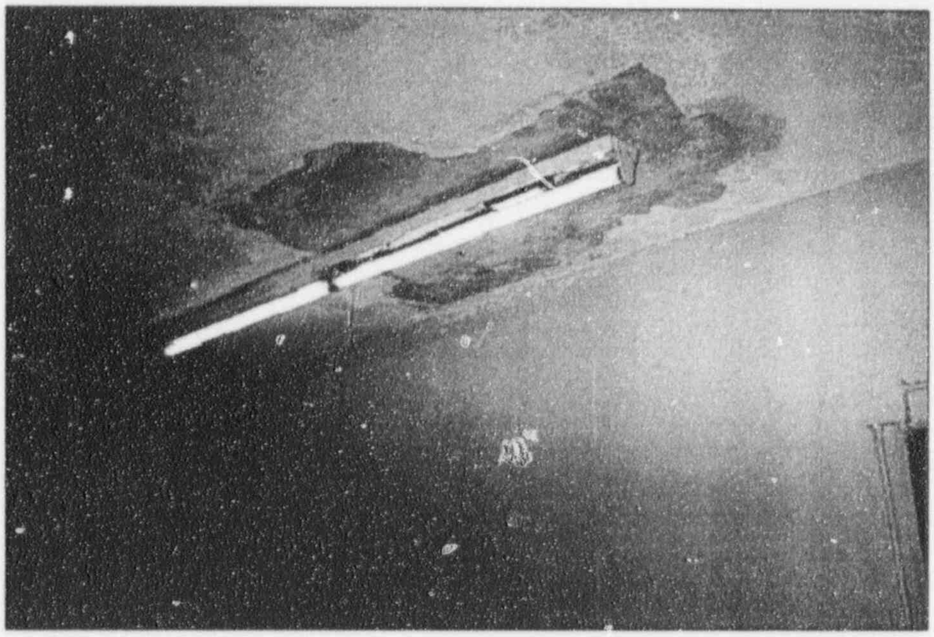

Figure A-2. Ceiling water damage.

This leakage caused no deterioration of the second floor slab's structural strength, and the fluids did not penetrate the concrete floor slab. There is no visible degradation of the floor slab, and it is highly unlikely there has been any degradation. In addition, the main part of the equipment room is over the radiography room where the floor slab is 2-feet thick.

5.2 CONCERN-6/4/4: *Evidence of the fluid that penetrated exists on the ceiling adjacent to the hot cell and in front of the radiography room and around the corner of the radiography room into a hallway at the north side of the radiography room.* (This was not a concern but a response is needed for clarification.)

5.2 RESPONSE: The first inspection of this AMSLF was conducted by this reviewer on April 30 through May 1, 1996. At that time, no ceiling water damage was noticed around the corner of the radiography room into a hallway at the north side of the radiography room.

A second inspection was conducted on June 3 to determine the discrepancy. While no water damage was noted in the false ceiling, it was quite apparent that fluid had run down the wall as shown in Figure A-3.

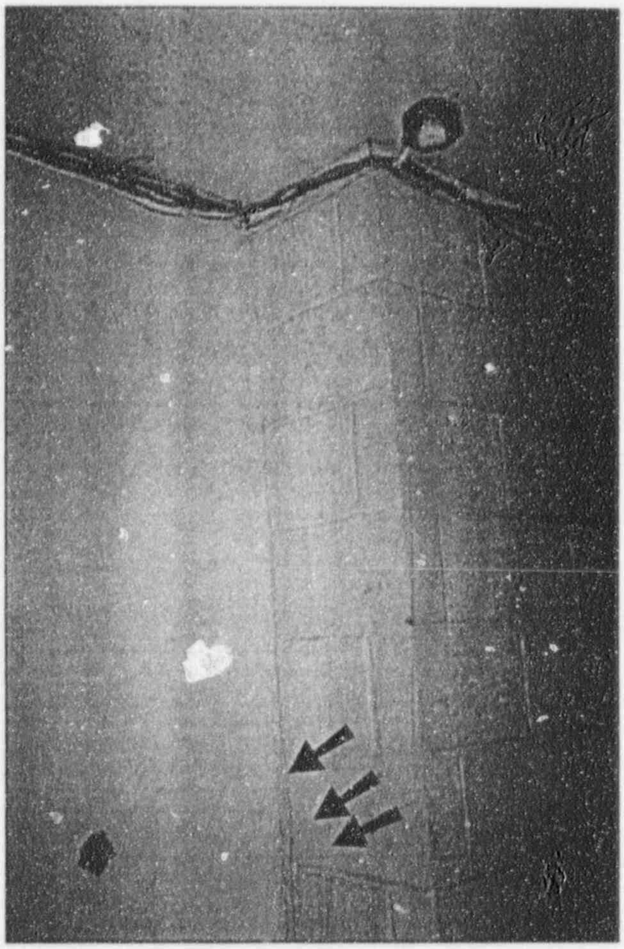

Figure A-3. No ceiling tile water damage but evidence of fluid on the wall

5.3 CONCERN-6/4/5: *It is not known what the source of this fluid was, but it could have been a source such as a ruptured pipe from freezing conditions or from the failure and leakage of exterior roof surfaces...*

5.3 RESPONSE: After discussing the potential source of the fluid with the operations manager and other staff, it became clear that most of the leaks had occurred as a result of equipment failures. For example, in the fall of 1995, a massive heat exchanger failed resulting in approximately 50 gallons of water being spilled into the equipment room which leaked

through to the first floor, as had previous leaks. In January 1995, a boiler leak occurred that caused leakage at the same location. Previous leaks caused the ~2 ft-by-2 ft ceiling plaster to fall from the false ceiling in this area. No one at the AMSLF knows the origin of the original leaks; however, most likely the leaks were caused by similar mechanical failures in the equipment room and possible roof leakage, especially since two major ventilation penetrations through the roof exist. However, it would take a serious roof leak to accumulate over 40 gallons of water.

5.4 CONCERN-6/4/7: *Evidence of significant roof leakage can be seen on the suspended ceiling of the second floor in several areas of the building...several areas such as in the southeast corner of the building and along the east front wall, there is evidence of water penetration of the roof deck structure.*

5.4 RESPONSE: Occasionally during the life of the building, leaks of the roof deck structure have occurred. In most all cases, the leaks have occurred where the roofing plies are tied into the older 1934 building or the parapet of the 1958 building. In October 1994, a new roof was placed over all of the 1958 building and the east half of the 1963 building. Thus, all of the current operating areas are protected by the new roof. In 1991, the roof over the 1934 building was replaced. It was not determined when the west portion of the 1963 building was last roofed. Minor leakage has occurred once or twice during a recent winter. No leakage has been observed since. While past leakage has caused appearance problems, there has been no apparent structural degradation of the building as a result (see also Response 5.5).

5.5 CONCERN-6/4/9: *This structure is made up of haydite (lightweight) precast concrete roof panels, that exhibit corrosion products from the embedded reinforcing steel.*

5.5 RESPONSE: While evidence of corrosion exists in some areas of visible roof decking, no structural degradation was noted. For structural degradation of the roof deck to occur, the reinforcing must corrode enough to significantly reduce its tensile strength. This much corrosion would result in significant expansion of the steel, thus causing spalling of the concrete away from the steel. Typically, failure of a concrete structure by corrosion occurs over a long period of time and shows ample evidence of distress long before failure occurs. Thus, while past leakage has caused appearance problems, there appears to have been no structural degradation of the roof decking.

5.6 CONCERN-7/1/2: *No information was available ... so it is unknown whether or not under freezing conditions there would be expansive forces created that would rupture the waterproof roof envelope again.*

5.6 RESPONSE: Over time it is inevitable that the waterproof roofing will fail causing penetration of the envelope. This could be caused by expansion due to freezing, expansion caused by high temperatures, damage by hail, or aging. Again, as with the precast concrete roof decking, early signs of distress, typically small leaks in the roofing, will be evident. Generally, the useful life of commercial roofing is 20 years before distress occurs. Since the replacement roof over the 1934 building is now five years old and the roof over the 1958 building and the east half of the 1963 building is two years old, one would not expect evidence of distress to appear prior to the year 2005. For the older roofing, distress could begin to show at any time. As such leakage occurs, operations management should have areas of leakage repaired until it is deemed necessary, from an operational or building degradation position, to replace that section of roof.

5.7 CONCERN-7/2/1: *The distress at the southeast corner of the building associated with the east 3-wythe load-bearing brick masonry wall...extending over approximately 4 feet vertically. The open crack, representing ... in the once continuous load bearing masonry wall that is 12 and ½-inches thick (Dwgs A-10 and P-2).*

5.7 RESPONSE: As described in the NRC report, cracking does exist and is shown in Figure A-4. From the outside, the cracking appears to stop just short of the area where the second floor slab ties into the common wall brick. From inside the building below the second floor, it is evident that cracking does extend through the wall; however, it is not continuous, i.e., the cracking on the inside of the wall is almost an opposite pattern, as shown in Figure A-5.

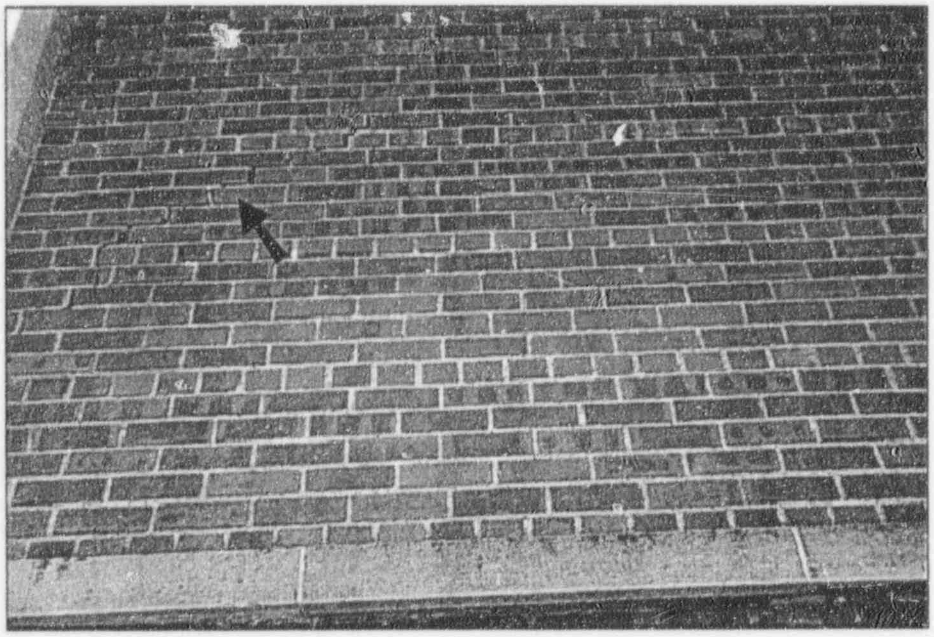

Figure A-4. Cracked southeast corner wall.

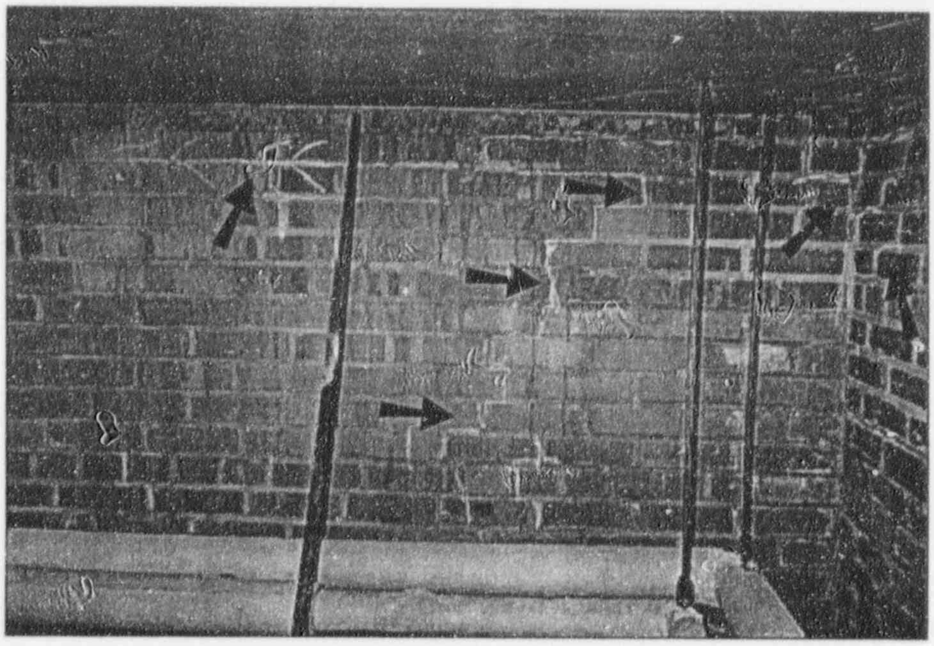

Figure A-5. Cracking on the inside wall.

From inside the building above the second floor, the east and south walls in the corner show no cracking; however, as shown in Figure A-6, there are four cracks in the second floor slab that are visible to the human eye. One very large crack is nearest the corner, is at an approximate 45° angle to each wall, has a width of one-half inch, and has a length from wall to wall of about six inches. Based on the width of this crack, it must be assumed that the crack continues through to the slab edges underneath both the east and south walls. No reinforcing steel was found in this crack. A second crack runs almost parallel to the south wall, appears to cross the third crack, and then merges into the fourth crack. The third and fourth cracks are also at about a 45° angle to each wall. Based on the width of the crack nearest the corner, it appears that the southeast corner has moved southeast a distance of as much as ½ to ¾-inch. These floor cracks seem to indicate excessive bending moment in the floor slab at this corner, which would be indicative of significant settlement at the corner. Underneath the second floor slab, matching crack patterns were found. It was determined that the as-built dimension of the floor slab was six inches.

Unfortunately, it is difficult to tell what actually caused cracking at the southeast corner of the building. As noted in the NRC report, the structural support of the 1958 building is a mix of load and non-load bearing masonry and concrete block, reinforced concrete, and steel framing. These materials are not compatible from an aging and expansion standpoint. In addition, the stiffness properties of the structure vary from extremely stiff (the test cell and radiography room) to very flexible (the lobby area). Thus, in a building over 30 years old with these types of similarities, one should expect to see cracking of this type; however, this cracking appears to have a unique cause. As noted in the NRC report,

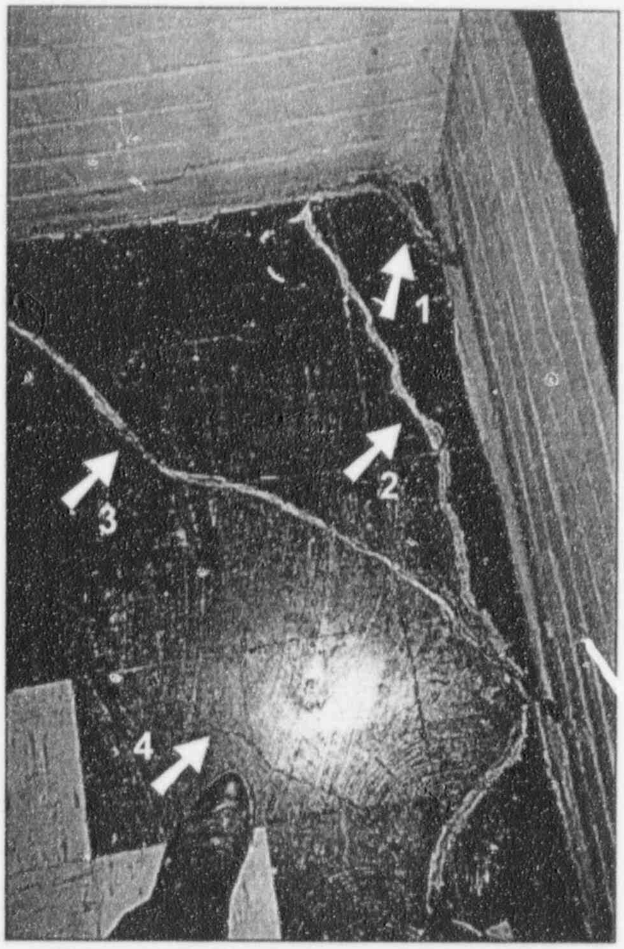

Figure A-6. Cracked second floor slab at southeast corner wall.

the cracking may have been caused by the 1986 earthquake; however, the highest Modified Mercalli Intensity rating for the East Cleveland area was an MMI V (Nicholson, et al., 1988), and the facility was located 25 miles southwest of the epicenter. An MMI is described as:

> Felt indoors by practically all, outdoors by many or most; outdoors direction estimated. Awakened many or most. Frightened few, slight excitement, a few ran outdoors. Buildings trembled throughout. Broke dishes, glassware, to some extent. Cracked windows, in some cases, but not generally. Overturned vases, small or unstable objects, in many instances, with occasional falls. Hanging objects, doors, swing generally or considerably. Knocked pictures against walls or swung them out of place. Opened or closed doors, shutters, abruptly. Pendulum clocks stopped, started, or ran fast or slow. Moved small objects, furnishings, the latter to slight extent. Spilled liquids in small amounts from well-filled open containers. Trees, bushes, shaken slightly.

It is not until an MMI VI has been reached that descriptions of building damage are included, as follows:

> Damage slight in poorly built buildings. Fall of plaster, in some amounts. Cracked plaster somewhat, especially fine cracks chimneys in some instances.

MMI VI was only recorded in a radius of 10 miles of the epicenter. In addition, the directional motion of the seismic wave should have been in a southwest direction. The southeast corner of the building has moved perpendicular to that motion. Thus, this reviewer believes this damage is the result of a different loading mechanism.

If a significant lateral or vertical load were to be applied to this location of the building, a localized corner failure of the building would occur between the first and second floors, while, due to the purlin bearing on the east wall at the corner, a much broader area of the roof in the west direction would collapse. However, due to the construction of this building, this reviewer does not believe that such a loading would lead to overall collapse of the building. The cracking has reduced the total vertical load carrying capacity of the wall. However, this corner and its associated purlin is carrying only half the load of the next northern purlin. See Response 5.12 for more discussion of the roof loads. In the long run, this is a life-safety issue, since failure of the building would not result in breach of the concrete core structure where the cobalt and other radioactive waste are located.

5.8 CONCERN-7/4/3: *The depth of the cracking into the 3-wythe wall is not known...*

5.8 RESPONSE: See Response 5.7.

5.9 CONCERN-7/4/5: *Whether or not the wall was constructed with a mortared collar joint is unknown, but it is assumed the wall was constructed as a solid masonry bearing wall.* This is not a concern, but a response is supportive.

5.9 RESPONSE: This is a good assumption.

5.10 CONCERN-7/4/6: *The crack then appears to trace downward at the vertical joint between the corner stone return on the southeast corner and the east wall.*

5.10 RESPONSE: As stated in the NRC report the fracture does traverse down the wall as noted. Also see Response 5.7.

5.11 CONCERN-7/4/7: *The crack then shows as a fracture in the stone ledge of the east wall at the corner.*

5.11 RESPONSE: The crack does show as a fracture in the stone ledge of the east wall at the corner. Based on the extensive movement that has occurred at this corner, this cracking is most likely a direct result.

5.12 CONCERN-7/4/8: *Originally, the sections of stone were pinned together with brass dowels and the joints were mortared. At some of the joints there has been rotation and translation with the rupture...Above the distressed region...wall supports the southern most roof structural steel purlin...*

5.12 RESPONSE: As noted above, each stone is independently hung off of the common brick wall, thus, unless significant gross wall movement occurs, no further distress should occur to the stone. The cracking in the masonry wall is a stepping crack, typically following mortar joints as shown in Figure A-4, since the mortar is much weaker in shear than the masonry. The load being applied to that section of wall by purlin (P2) is half of the load being placed on the wall by the other purlin (P2) loads. Per Dwg. S-1, the roof dead load is 30 psf and the live load is 30 psf for a total roof load of 60 psf. Thus, the load applied to the wall by the purlin is approximately 3000 lbs, which, based on the base plate area, equals a compression load on the wall of 50 psi. The allowable compressive stress on a non-cracked masonry wall is 1500 psi. At the second floor slab area of the southeast corner of the building, the load applied to the load bearing masonry wall from the floor slab is approximately 1500 lbs/ft based on floor live and dead loads plus the 2100 lbs/ft dead load of the wall. This places an additional compression load of approximately 40 psi on the common wall below the second floor slab. A compression force on the wall of 40 psi is a trivial load compared to the allowable load of 1500 psi. Again, it appears that this cracking must be from localized settlement at the corner.

5.13 CONCERN-7/4/12: *On the inside of the 1958 building at this purlin bearing there is evidence of movement between the bearing wall and the purlin in the longitudinal direction...is not known...which structural element remains with the permanent movement...*

5.13 RESPONSE: This movement appears to be approximately ¼-inch as shown in Figure A-7. Although such movement may occur for a building having a mix of materials involving steel, concrete, and masonry as main structural elements, this movement must be attributed to the corner moving to the southeast at the second floor. According to Dwg. S-1, all beams bearing on the masonry wall have wall anchors. These anchors are two angles, 6 inches by 4 inches by 3/8 inch. Thus, the wall anchor should be holding the common masonry wall from moving further eastward at the top. No sign of distress in the veneer

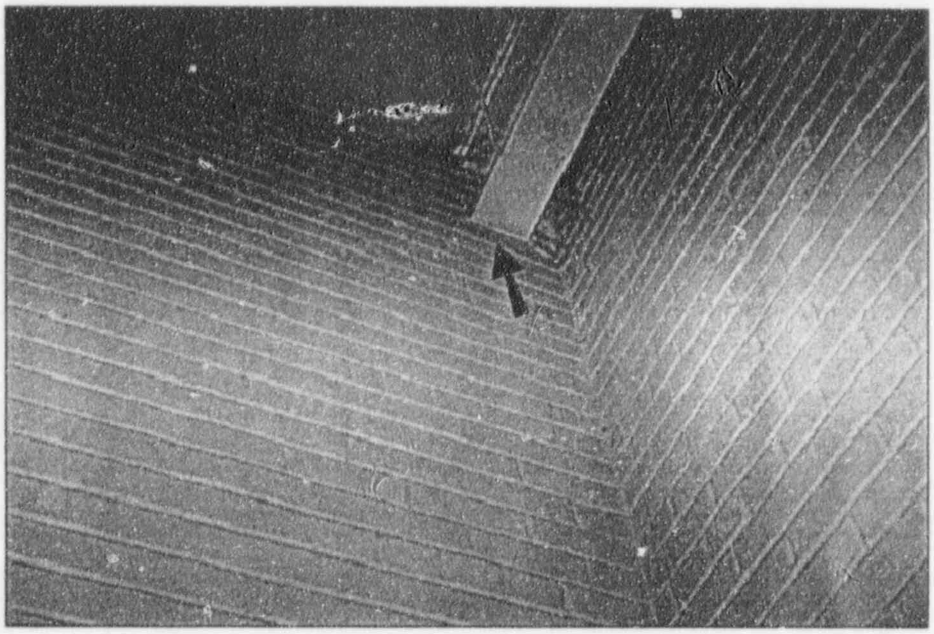

Figure A-7. Wall movement at purlin in corner.

wall was noted from the outside, however, some cracking was noted in the load bearing wall beneath the purlin.

5.14 CONCERN-7/4/14: *In addition to the cracking of the east wall, evidence of lateral loading was found at a point about 17-feet...Rupture of the joints of this wall where the masonry was fit around the purlin as occurred...*

5.14 RESPONSE: It is the opinion of the reviewer that this movement has nothing to do with what has happened at the southeast corner of the building. The biggest problem with the construction of this building is that none of the interior non-load bearing walls were interlocked or tied in with any of the structural members, and most of the non-load bearing walls have separated leaving visible cracks from adjoining walls. The location of the cracking in question is shown in Figure A-8 and, as indicated in the NRC report, the crack opening is approximately one inch. This wall is the west side of a right angle interior wall forming a small room in the southeast corner. If the north side of the interior wall is compared to the paint lines on the roof decking, it is evident that the entire top of the non-load bearing wall has moved northward about one inch. There is no evidence that the purlin has moved southward at this location.

5.15 CONCERN-8/2/1: *The distress of the east wall near the northeast corner of the 1958 building is associated with a rupture type failure...rupture line is most pronounced in a verti-*

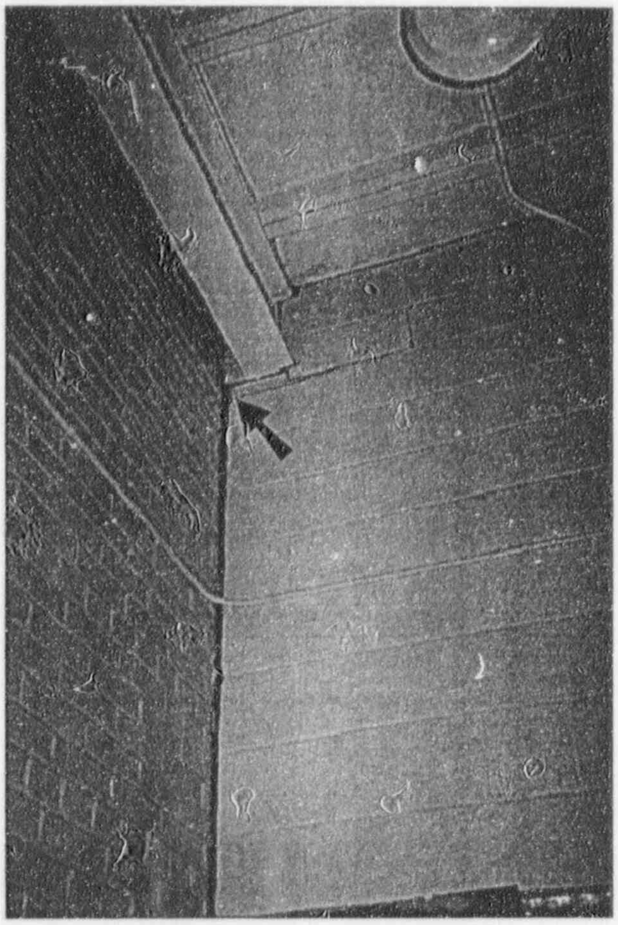

Figure A-8. Purlin and Wall Movement.

cal direction just adjacent to the northeast steel column... rupture surface is generally through every other course of concrete block and does not follow a saw tooth pattern...

5.15 RESPONSE: Paragraph two on Page 8 of the NRC report is mostly about the cracking of the in-filled wall. Upon inspection of this wall's cracking and movement, it seems quite evident what is occurring. Basically, it is a problem of the second floor in-filled wall moving outward over the lobby area between the steel columns at Column Lines D and F. This movement can clearly be seen from the roof and is causing rotational stresses to be placed on the original corner of the 1958 building as the east-west wall tries to keep the east wall from moving outward. The fact that the 1958 building in-filled wall was placed tightly against the inner side of the outside column flange, and the fact that ties attached the two wythes of facing brick to the 1958 building concrete block, the 1958 building in-filled wall is being pulled outward next to Column F-1, failing the in-filled wall in shear at the edge of the flange. Without further inspection, and possibly some destructive inspection, it is difficult to determine the specific cause. Fricke, et al. 1978 have addressed such problems and found that temperature and moisture effects combined with constrained

expansion can result in this type of behavior in such construction. This is not a structural issue, but it is a life-safety issue.

5.16 CONCERN-8/2/17: *In addition, the stone corner and stone return at the northeast corner of the 1958 building show displacement and rotation at the corner with failed joints.*

5.16 RESPONSE: It is believed that the failure of the stone corner at the bottom of the wall has resulted from a totally different cause, but may have something to do with the in-filled wall movement at the second floor. This stone corner is located next to the lobby entrance of the 1958 building. Upon inspection of the site, it is very apparent that the joint failure of the mortar 's been caused by salting the lobby entrance to remove snow and ice. The stone facing ' liscolored where the salt was thrown as shown in Figure A-9, and where discoloration has occurred, mortar in the joint was attacked. At a distance of approximately one foot north of the south corner of the stone, there is no stone discoloration and the joint is intact. As a result of joint failure, the stone has fractured. Again this is a life-safety issue and has no bearing on the structural stability of the 1958 building.

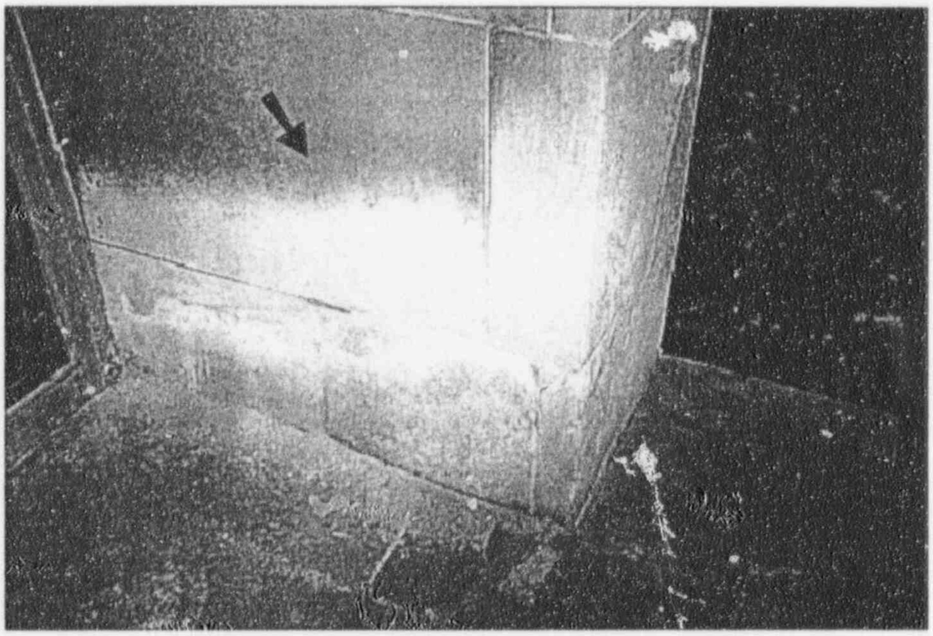

Figure A-9. Lobby stone showing salting.

5.17 CONCERN-8/2/18: *The distress was also reflected in the displacement of the stone coping at the top of the walls as they intersect at the northeast corner of the building...*

5.17 RESPONSE: While there is significant distortion of the stone coping on the parapet of the 1963 building as shown in Figure A-10, with the 1958 building parapet being in the background, this distortion is not believed to be associated with Concerns 5.15 or 5.16 discussed above. Rather, it is believed to have been caused possibly by a crane or crane-like piece of equipment placing workers or equipment on the roof. The movements of the stone coping of the 1963 building do not relate to the movements that have occurred at the northeast corner of the 1958 building.

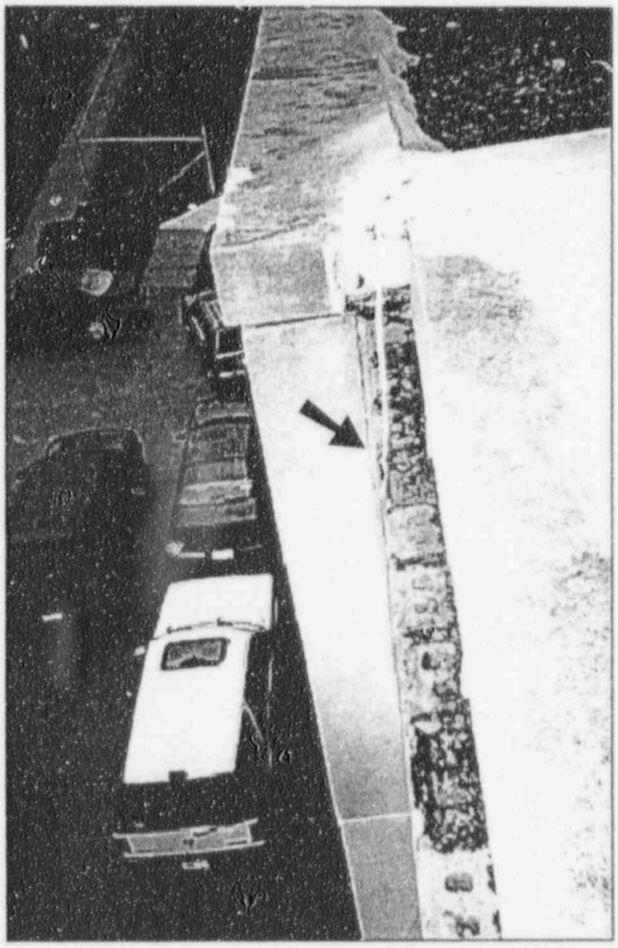

Figure A-10. 1963 building parapet.

5.18 CONCERN-9/2/1: *Based on the observations of the areas of distress of the two ends of the east front wall of the 1958 building, the interface..*

5.18 RESPONSE: The remaining structural integrity concerns of the NRC report have to do with the 1958 building's resistance to seismic load and the impact of the 1963 building's response on the 1958 building. See Attachment II of the main report for a simplified seismic assessment of the 1958 building response.

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Fricke, K.E., W.D. Jones, and J.E. Beavers 1978. *Problems in Masonry Walls - A Case History, Proceedings North American Masonry Conference*, University of Colorado, August 14-16.

Frye, M.J., and G.A. Morris 1975. *Analysis of Flexibly Connected Steel Frames*, Canadian Journal of Civil Engineering, 2, 280.

Henderson, R.C., J.E. Beavers, and W.D. Jones 1995. *Hollow Clay Tile Wall Program Summary Report*, Report No. Y/EN-5347, Center for Natural Phenomena Engineering, Martin Marietta Energy Systems, Inc., July 30.

Nicholson, C., E. Roeloffs, and R.L. Wesson 1988. *The Northeastern Ohio Earthquake of 31 January 1986: Was It Induced?*, Bulletin of the Seismological Society of America, Vol. 78, No. 1, pp. 188-217, February 1988.

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ATTACHMENT II

SEISMIC AND TORNADO STRUCTURAL INTEGRITY OF THE ADVANCED MEDICAL SYSTEMS LABORATORY FACILITY

by

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SEISMIC ISSUES

Some seismic issues were addressed in Attachment I of the full report concerning the likely cause of damage to the south-east corner of the Advanced Medical Systems Laboratory Facility (AMSLF). This attachment looks at the vulnerability of the AMSLF to an earthquake.

The location of the AMSLF places it between the 0.05 and 0.10 g contours of peak velocity-related acceleration coefficient, A_v based on the NEHRP Provisions. Based on the shape of the map's contours, a new facility at this location would be designed for an A_v of about 0.07 g. Typically, well designed unreinforced masonry structures can perform as expected up to an A_v ranging between 0.10 and 0.15 g. The AMSLF is in Seismic Hazard Exposure III because it contains some level of radioactive material. Since it is located between A_v contours 0.05 and 0.10 g it is in a Seismic Performance Category C.

Some simplified calculations were done to determine the response of the AMSLF in an earthquake and are attached. The AMSLF foundation rests on shale, thus, there would be no amplification of the ground motions from that specified on the map. From the seismic map, the design input for such a facility would be about 0.07 g (the calculations in the attachment use 0.05 g as the basic input load).

The structure representing the test cell basement, test cell, and radiography room is a massive reinforced concrete structure with wall thickness varying form three to five and half feet. This is an extremely rigid structure, probably having a fundamental frequency in the 25 to 35 hz range. In

addition, over the radiography room a two foot thick second floor slab is tied into the 3 foot walls. The remaining six inch second floor slab is tied into the test cell and the two foot thick slab over the radiography room. In addition, the first floor is also tied into the test cell structure and foundation of the radiography room. Both the first and second floors can be considered as rigid diaphragms. An east west elevation of the test cell is shown on page one of the calculations and the second floor test cell and radiography room rigid body in the horizontal plane at the second floor level is shown on page five of the calculations.

Based on the construction as described above, if an earthquake were to occur all of the horizontal loads would be transmitted into the massive concrete structure. Therefore, the first floor unreinforced masonry load bearing wall would see very little, if any, seismic load because there would no displacement of the first floor and virtually no, if any, displacement of the second floor. The second floor wall would experience some load. A simplified calculation of the shear load for and input load of 0.10 g, page 7 of the calculations, shows that the demand on the unreinforced load bearing masonry wall on the second floor is 0.82 psi verses a code allowable of 10 psi. It is well known that unreinforced masonry ultimate shear load is typically higher than 40 psi. Thus, in the critical operational areas of the AMSLF seismic loads do not place significant stress on the unreinforced masonry load bearing wall. However, as a result of the pre-existing cracking in the south east corner of the building an earthquake could initiate partial collapse. As noted in other sections of the main report, collapse of the building would not result in loss of containment or confinement of the facility.

Because of the stiffness differences between the 1958 and 1963 buildings, they will respond differently. For the low earthquake hazard, the short duration of earthquakes in low hazard zones, and the one-half inch spacing between the 1963 building and the 1958 building walls, if pounding did occur it should not be severe and only minor damage would be expected. However, damage of the roof waterproofing could occur because it will be flexed.

TORNADO ISSUES

Tornadoes can do significant damage to an engineered structure when their wind speeds exceed 120 mph. If a severe tornado having wind speeds in excess of 270 mph, where many engineered buildings can be severely damaged, were to strike the AMSLF everything except for the test cell and radiography room on the first and second floors would experience damage. The ventilation system filters related to the test cell and radiography room would be vented to the atmosphere and the doors to the radiography room would be blown outward. Because of the massive reinforced concrete structures of the test cell and radiography room, they would remain in place with very little, if any, structural damage. The walls of both room would prevent the penetration of the most severe missiles. The basement level that includes the garden room and the test cell basement would not be impacted. While the confinement of the radiography room and test cell would be breached in a

severe tornado, high concentrations of exposure of site of the facility would not occur because of the dispersing power of a tornado.

ATTACHMENT

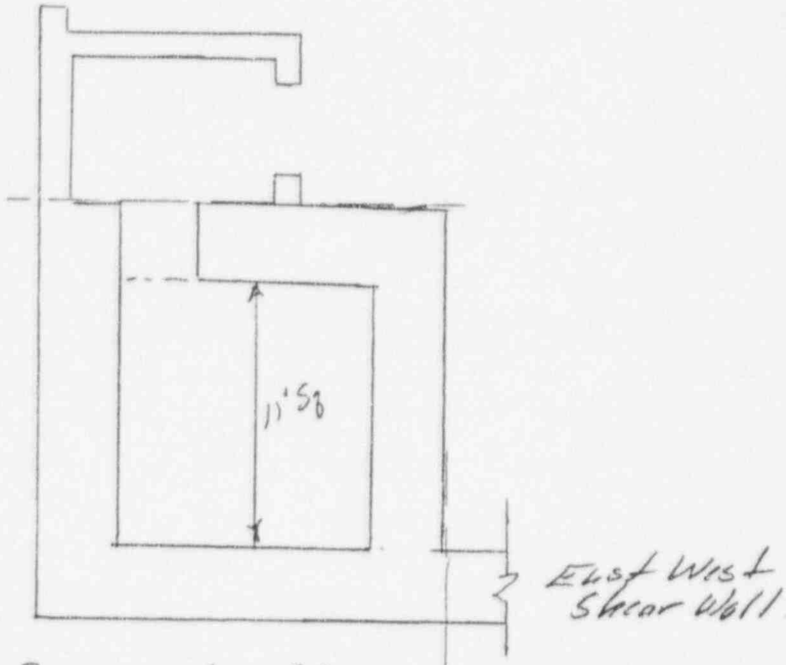
SIMPLIFIED CALCULATIONS

ATTACHMENT II-A

SIMPLIFIED CALCULATIONS

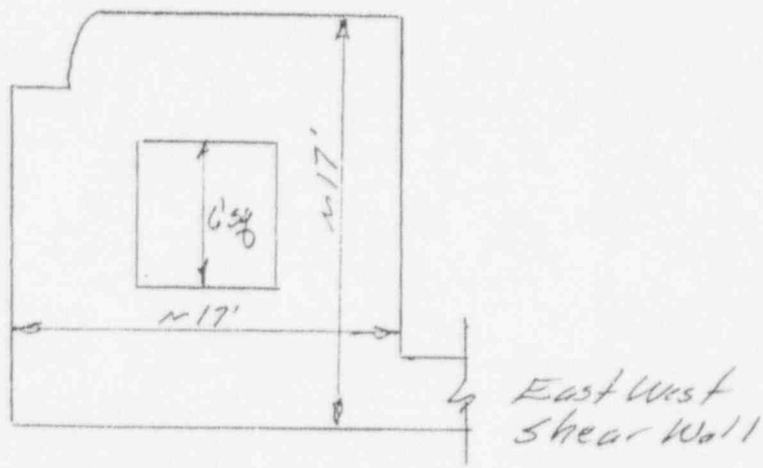
MODEL 1

Mass of Test Cell Structure



Basement Plan

Assume Rectangular Structure
and Ignore Reinforced Concrete
East West Shear Wall



First Floor Plan

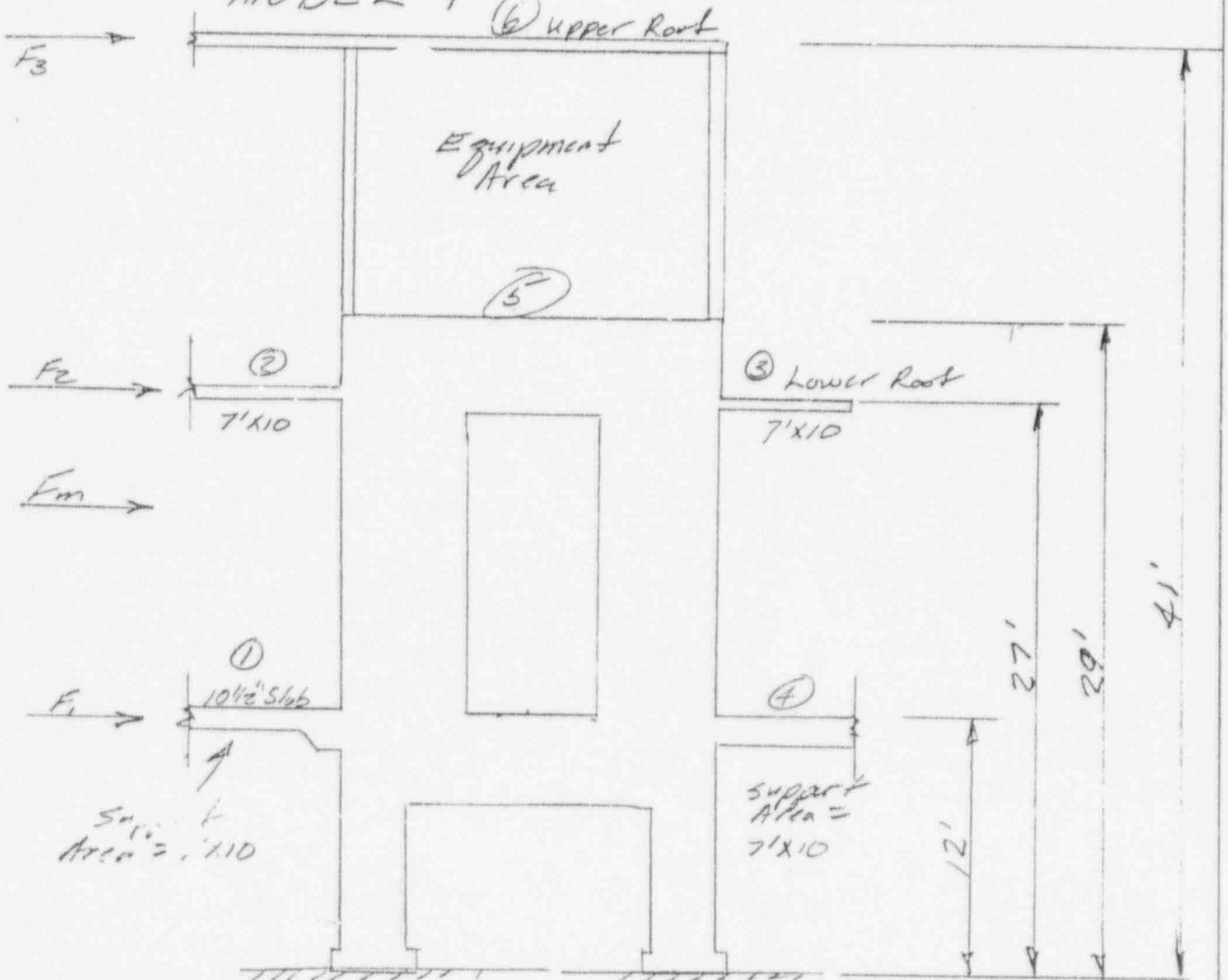
Ingnore Port Areas
This Add Mass

5/26/96

ATTACHMENT
SIMPLIFIED SEISMIC ANALYSIS
ADVANCED MEDICAL SYSTEM FACILITY

1. TEST CELL MODEL AND CALCULATIONS

MODEL 1



From Section A-A Dwg A-5

East West Section

5/26/96

Test Cell Mass = [17X17X29 - 11X11X6.5 - 6X6X13.5] X 150

Concrete Wt = 150 #/ft^3

= [8381 - 786 - 486] 150 = 1.1×10^6 lbs

= 1066 kips

Contributing Floor Loads

D.L. = 175#/ft^3 L.L. = 500#/ft^3

Floors 1&4 2X7X10X[175+500] = 94 kips

Floor 2 = 7X10X[90+150] = 17 kips

Floor 5 = 17X17X[90+150] = 61 kips

Roof 3 = 7X10[50] = 3.5 kips

Roof 6 = [17X17 + 7X10] 50 = 18 kips

Total Mass ≈ 1066k + 194k = 1,260 kips

Assume 1 dim model of test cell in east-west direction as a simple cantilever beam with total mass applied at top of test cell (a large shear wall) having a 1 foot width. This equals a 1g earthq.

Max moment = $\left(\frac{1260}{17'}\right)$ 29' = 2,149 kft

$$\sigma = \frac{Mc}{I} = \frac{2149\,[17/2\,(12)]}{\frac{12\,(17/12 \times 12)^3}{12}} = \frac{2149\,(102)}{1.1 \times 10^6}$$

= 0.207 kips/in or 207 psi << 3000 psi

Check for overturning

Downward Load = $\frac{1,260 \text{ kips}}{12(17)(12)} = \frac{1260 \text{ kips}}{2448 \text{ in}^2}$ = 0.515 kip.

= 515 psi

Downward Load of 515 psi >>> 207 psi

The BOCA code uses the NEHRP Seismic Map. Cleveland is in the low seismic zone where $A_a = 0.05g$

Actual Earthquake stress in the above model = 207 psi (.05) = 10 psi

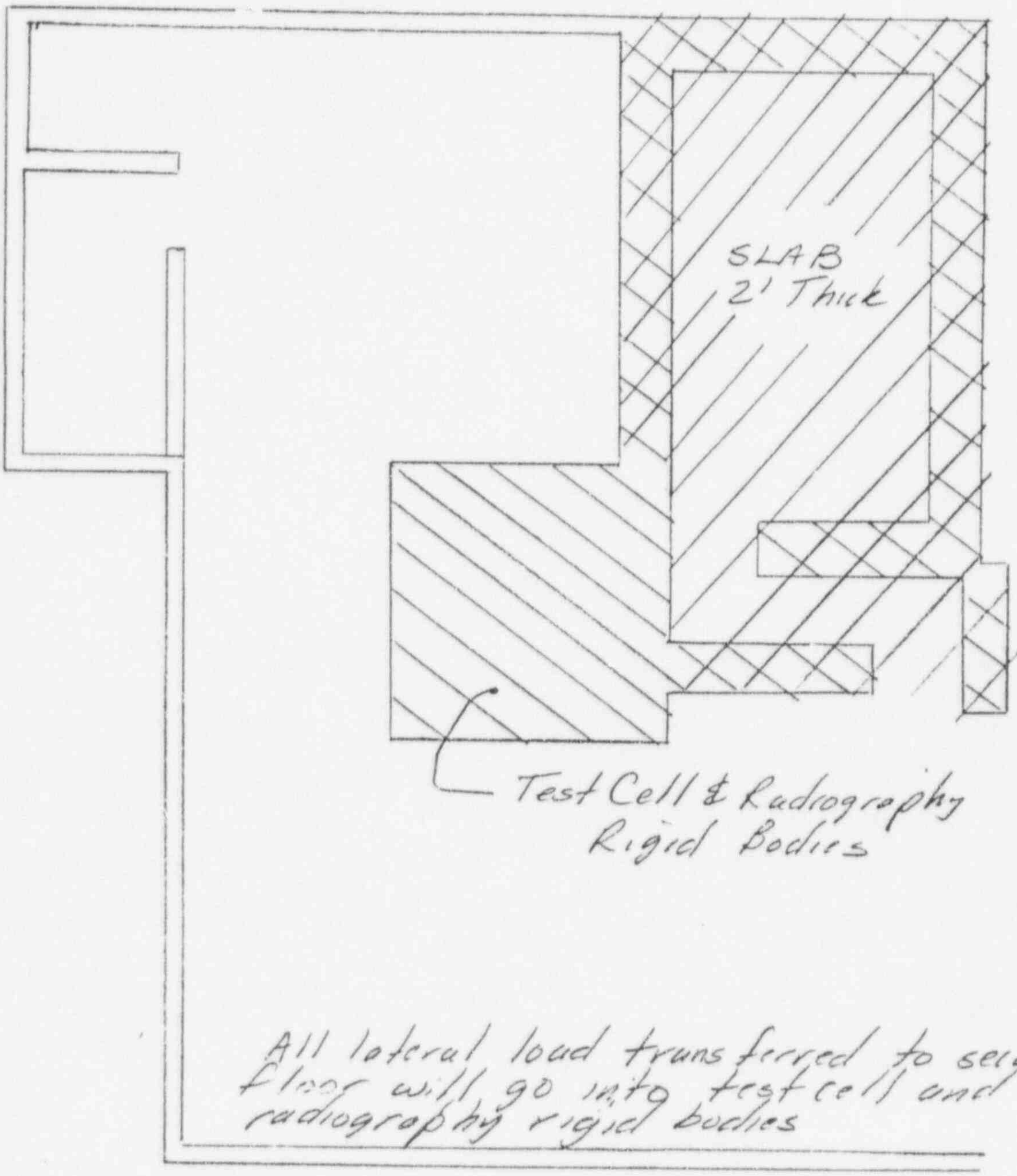
Thus, the test cell is a very rigid structure. And an area earthquake would apply almost no load on it.

2. Effect of Test Cell Rigidity on the rest of the facility

The first and second floors represent rigid diaphragms that will take all of the load along the exterior walls of the building directly into the test cell because of it rigidity. In addition, the shear wall ignored in the test cell model also adds rigidity to the system. In addition on the north west side of the test cell the concrete floor slab is two feet thick over the radiography room of the first floor. The shear wall is around the radiography room.

Due to fact the second floor acts as a rigid diagraphm and is tied into the test cell and radiography room walls, all of the load will be carried by the test cell and radiography room walls, and ~~is~~ very little, if any, shear load will be applied to the first floor exterior unreinforced masonry walls.

5/26/96

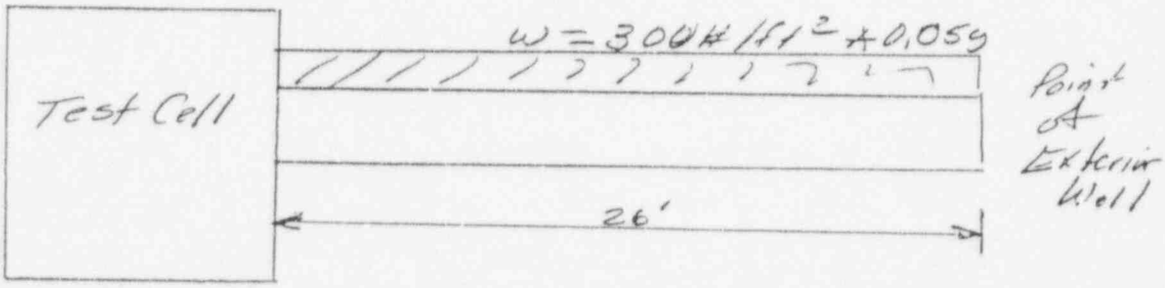


Plan To Scale

Check   Assume only test cell is rigid
Assume both roof and second floor load are on second floor

$90 + 150 + 60 = 300\#/ft^2$ of floor mass

Assume 3 ft section of floor is cantilevered out from test cell in vertical direction



$$M_{max} = \frac{w\,l^2}{8} = \frac{300(.05)(26 \times 12)^2}{8} = 183\,k\,in$$

$$\sigma = \frac{Mc}{I} = \frac{183\,k\,in\,(18)}{\frac{12(36)^3}{12}} = \frac{3294\,k\,in^2}{46656\,in^4} = 0.071\,k\,in^2$$

$\sigma = 71$ psi <<< Concrete tensile stress

$$\text{Displacement} = \frac{w\,l^4}{8EI} = \frac{.15\,[26(12)]^4}{8 \times 3 \times 10^6\,[46656]}$$

$$= \frac{.15\,(9.5 \times 10^9)}{24 \times 10^6 \times 4.6 \times 10^4} = \frac{137.32 \times 10^9}{110.4 \times 10^{10}}$$

$= 0.12\ in^2$ Very Small Displacement
Actual Displacement would be much smaller

Thus there would be no amplification of the earthquake motion at the second floore

Above the second floor you have a rigid roof diaphragm

1' Unreinforced Masonry Wall is 13' to base plate of purlins

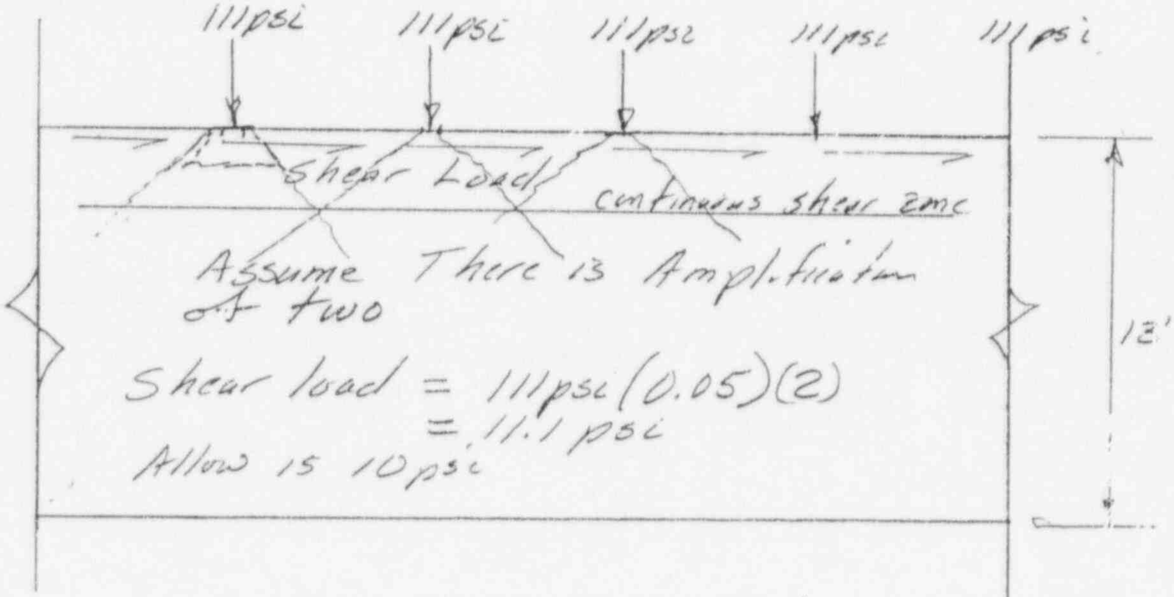
purlin load on east wall is

$26' \times 8' \times (60 psf)/2 = 6,240$ lbs

Bearing Plate Area $= 7" \times 3/8" \times 8"$

Purlins P2 = 14 W=30 Flange width = $6\,3/4"$

Wall stress $= \frac{6240}{7 \times 8} = 111$ psi



Second Floor Wall Section

Allowable Shear Load In Unreinforced Masonry is 10psi

Gross Area Supporting Purlin Load is approximately $8 \times 8' \times 12" = 768\ in^2$

Shear $= \frac{6240}{768}(.05)(2) = 0.82$ psi << 10ps

10psi is allowable most masonry ultimate shear is 40psi and above

Include mass of wall

$150\ \#/ft^3 (1ft)(1ft)(13ft) = 1950\ lbs$

$$\frac{1950}{8''(12)''} = \frac{1950}{96} = 20\ psi$$

$20 psi (.05)(2) = 2 psi$ shear load

See pg 9 for same calculations using NEHRP Provisions

NEHRP Provisions fairly conservative for unreinforced masonry.

At a 45° purlin load distribution there would be overlapping of the purlin loads at a depth of 3.6 ft in the wall. At this location there would be a continuous vertical load of P_r

$$P_r = \frac{6240}{8''(8')(12'')} = 8.125\ psi \quad \text{not } 11\ psi$$

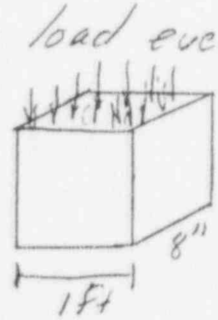
not counting wt of 3.6ft wall which is $540\#/ft^2$ = 3.75 psi

Shear Load at this location would be

$11.875\ psi (.05) = 0.594\ psi \ll\ll 10\ psi$

Another way of looking at it

6240# load over 8ft = uniform load of 780#/ft



Vertical load 780#

$A_n = 12(8) = 96$

(see next page also)

$S_{nt} = 1.5\sqrt{f'_c A_n} = 804\#$

Shear Load = 780(.05) = 39# << 804#

For unreinforced masonry the allowable shear per the NEHRP Guidelines is

$S_{ALL} = 1.5\sqrt{f'_c A_n}$ $A_n = 7 \times 8'' = 56 \text{ in}^2$ plate

Assume $f'_c = 3000$ psi Actual A_n of wall is $(12)(8) = 96$

$S_{All} = 1.5\sqrt{3000(96)} = 805$ lbs

Purlin Load is 6240 lbs

Purlin Shear Load $= 6240(.05)(2) = 624$ lbs

or $S_{All} \leq 120 A_n$ lbs $= 120(96) = 11{,}520$ lbs (No) $>> 805$

$60 A_n + 0.3 N_v = 60(96) + 0.3(6240) =$ No Good $>> 805$

With an amplification of 2 which is not likely since the unreinforced masonry is relative rigid until it cracks

Thus the actual seismic load is more like 0.05 g resulting in a shear stress value of around

$\frac{6240}{8}(.05) = 39$ lbs vs an allowable of 805 lbs

or 8.38 psi vs an allowable of 10 psi

if seismic load is 0.07 g shear stress is 55 lbs.