June 21, 1996

MEMORANDUM TO: Michael J. Bell, Chief
Engineering and Geosciences Branch, DWM

FROM: Josephine M. Piccone, Acting Chief [orig. signed by GPangburn, for]
Operations Branch, IMNS

SUBJECT: REVIEW OF TECHNICAL ASSIST REQUEST FROM REGION III OF THE STRUCTURAL INTEGRITY REPORT FROM ADVANCED MEDICAL SYSTEM, INC.

Attached is a copy of a Technical Assistance Request from Region III, requesting review of AMS' response to an inspection reported in USNRC Inspection Report No. 030-16055/95006(DNMS). As a result of the inspection referenced, NRC asked AMS to provide an evaluation of the facility's ability to provide protective confinement of stored radioactive materials over the facilities intended use period; plans for structural remediation, if warranted; and plans for periodic inspection and evaluation of the building's ability perform its function.

Some of these issues bear on the review of AMS' emergency plan, which is part of their license renewal application. Please review the report and provide comments to Joe DeCicco by July 5, 1996, so that your input can be coordinated with an IMNS response.

Attachment: As stated

CONTACT: Joe DeCicco, IMOB
415-7833
e-mail JXD1

Distribution: w/o att
~~NRC Central File~~ IMOB r/f
IMNS r/f NMSS r/f

DOCUMENT NAME: G:IMNS5398.JED [comments]

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OFC	IMOB	C	IMOB		IMOB	
NAME	JDeCicco/ad/11		GPangburn		JPiccone	
DATE	6/14/96		6/17/96		6/20/96	

OFFICIAL RECORD COPY

10/30

MEMORANDUM FOR: Donald A. Cool, Director
Division of Industrial and Medical
Nuclear Safety, NMSS

FROM: John R. Madera, Chief
Nuclear Materials Licensing, RIII

SUBJECT: REQUEST FOR TECHNICAL ASSISTANCE IN THE REVIEW OF THE STRUCTURAL INTEGRITY REPORT FROM ADVANCED MEDICAL SYSTEMS, INC. (AMS)

RIII recently received AMS' Structural Integrity Report, which was written in response to our March 12, 1996 letter (with an attached report by R. Shewmaker). The report was mailed on 6/12/96 via overnight mail to Joe DeCicco of your staff.

Please coordinate the review of the report with the appropriate people in DWM, etc., and provide written comments to RIII as soon as possible. As you know, some issues in the report have direct bearing on the review of AMS' emergency plan, which is part of AMS' renewal action.

CONTACT: Michael Weber
708-829-9825

Attachment

Advanced Medical Systems, Inc.
1020 London Rd.
Cleveland, Ohio 44110
216-692-3270

June 7, 1996

Mr. Geoffrey C. Wright
Acting Deputy Director,
Division of Nuclear Materials Safety
U. S. Nuclear Regulatory Commission
801 Warrenville Road
Lisle, Illinois 60523-4351

Re: USNRC Inspection Report No. 030-16055/95006 (DNMS)

Dear Mr. Wright:

Advanced Medical Systems, Inc. (AMS) is in receipt of your March 12, 1996 letter in regard to the referenced inspection report. In that report, the USNRC concluded that the 1994-1995 basement flooding had no observable impact on the structural integrity of the London Road facility. However, the USNRC asked AMS to provide an evaluation of the facility's ability to provide protective confinement of the radioactive materials stored therein over the facility's intended use period; plans for structural remediation, if warranted; and plans to periodically inspect and evaluate the building's ability to perform its defined functions over the intended use period.

Enclosed is the AMS response to the inspection report and to the USNRC's March 12, 1996 request. These responses are based upon the findings of an independent evaluation of the building's status that was performed by Dr. James Beavers, P.E. (MS Technologies, Inc., Oak Ridge, Tennessee). If you have any questions, please call me at (216) 692-3270.

Sincerely,

Robert Meschter

Robert Meschter, R.S.O.

enc.
cc: D. Cesar
D. Miller - Stavole & Miller
C. Berger - IEM
M. Weber - USNRC Region III

9701290128

INTEGRATED ENVIRONMENTAL MANAGEMENT

SEISMIC AND STRUCTURAL REVIEW OF ADVANCED MEDICAL SYSTEMS LABORATORY FACILITIES

by
James L. Beavers, Ph.D., P.E.
Vice President

June 6, 1996

MS Technology, Inc.
118 Ridgeway Center
Oak Ridge, Tennessee 37830

~~9701290129~~ 20pp.

Advanced Medical Systems, Inc.
1020 London Rd.
Cleveland, Ohio 44110
216-692-3270

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Sincerely,

Robert Meschter

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cc: D. Cesar
D. Miller - Stavole & Miller
C. Berger - IEM
M. Weber - USNRC Region III

9701290128

Agency Comment 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, which exhibits the effects of previous fluid penetration through the slab from above [should be addressed].

AMS Response: The engineer's report states that equipment failure in the equipment room has caused leakage on the second floor. However, he concludes that the floor slab's structural strength has not been compromised as a result of the leak.

Action Taken: None required.

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

AMS Response: The engineer's report concludes that even with no repair or maintenance the AMS building on London Road is capable of providing protective confinement for its licensed radioactive materials inventory for many years into the future. Therefore, a routine inspection program is not required.

Action Taken: None required. However, to ensure the long-term useability of the remainder of the building in light of the instances of cracking, settling, and distress that were noted by both the USNRC inspector and the AMS structural engineer, a survey program to monitor the movement of the walls for the purpose of predicting future corrective actions will be instituted. For this program, a survey crew will be contracted to set up a base of measurement for the north wall (first bay) of the 1963 building, the east wall of the 1963 building, the wall above the lobby of the 1958 building, and the southeast corner of the building. The crew will then return approximately six (6) months later to determine if any movement occurred. If none is noted, the survey will be repeated every two (2) years thereafter. However, if the six-month survey does reveal movement, a registered Professional Engineer will be asked to specify the frequency of future surveys in light of the magnitude of movement.

In addition to the survey program, the AMS radiation protection staff, as part of the routine surveillance program described in RSP-008, "Instrumentation and Surveillance", will inspect the building at the locations of interest in order to identify unusual conditions. Any follow-up action that might be warranted (e.g., repeat surveillance, repair, re-construction) will be specified by a registered Professional Engineer.

INTEGRATED ENVIRONMENTAL MANAGEMENT

SEISMIC AND STRUCTURAL REVIEW OF ADVANCED MEDICAL SYSTEMS LABORATORY FACILITIES

by
James E. Beavers, Ph.D., P.E.
Vice President

June 6, 1996

MS Technology, Inc.
118 Ridgeway Center
Oak Ridge, Tennessee 37830

~~9701290129~~ 20pp.

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 south-east 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 1 for more detailed discussion.

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.

ATTACHMENT I

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

—

A RESPONSE

by
James E. Beavers, Ph D., P.E.
Vice President
MS Technology, Inc.
118 Ridgeway Center
Oak Ridge, Tennessee 37830

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.*

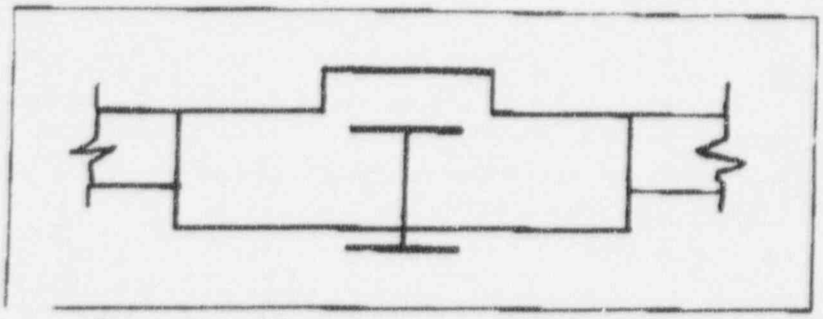


Figure A-1. Masonry pilasters and steel columns.

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.

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.

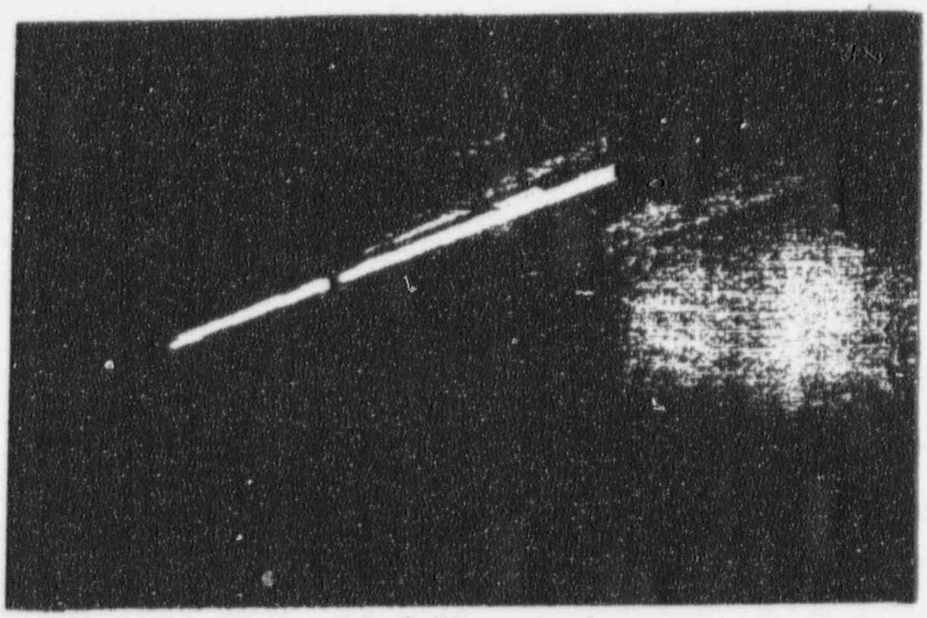


Figure A-2. Ceiling water damage.

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.

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.*

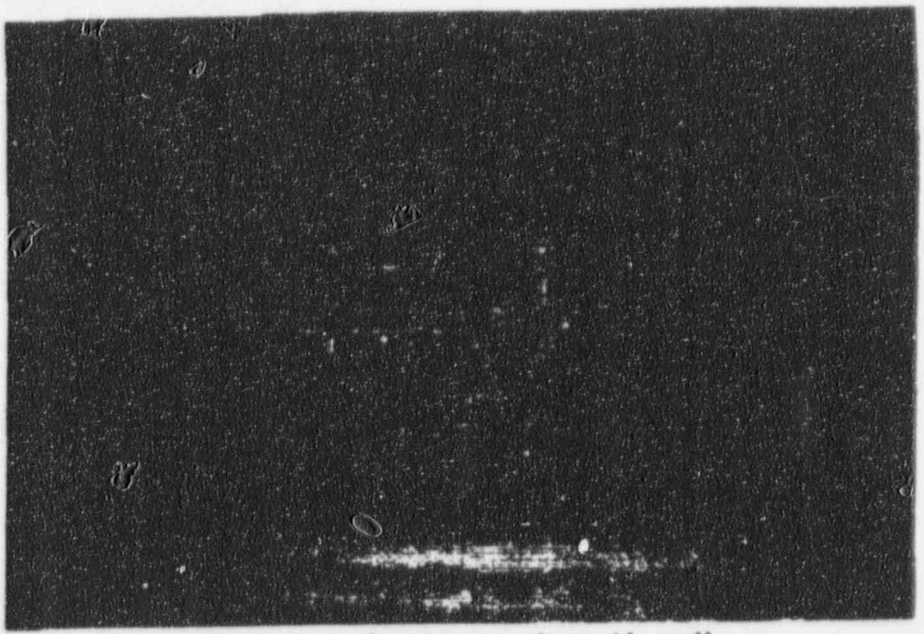


Figure A-5. Cracking on the inside wall.

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 cr k 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,

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.

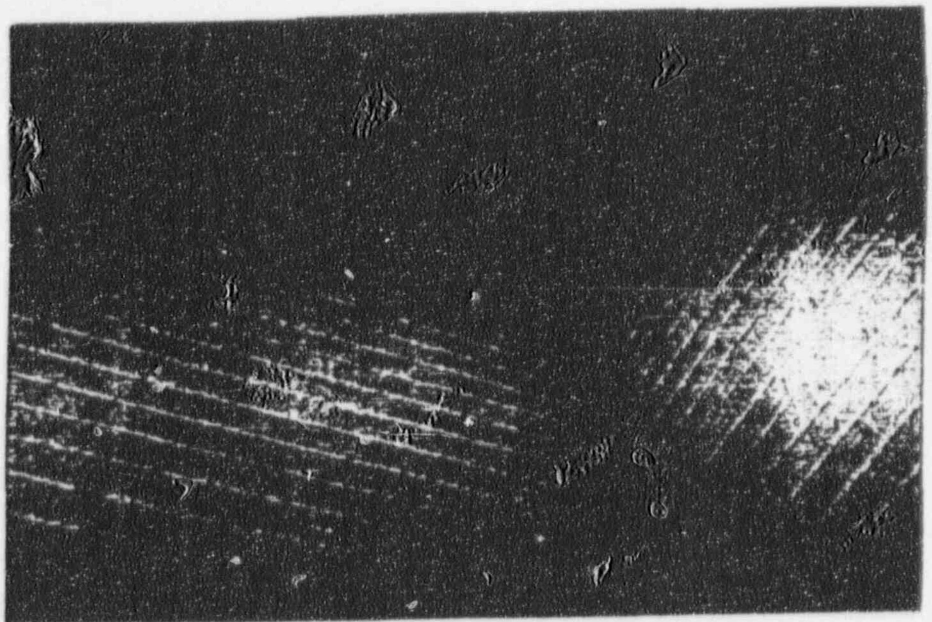


Figure A-7. Wall movement at purlin in corner.

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 c_ inion 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 moypd 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-*

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 has been caused by salting the lobby entrance to remove snow and ice. The stone facing is discolored 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.

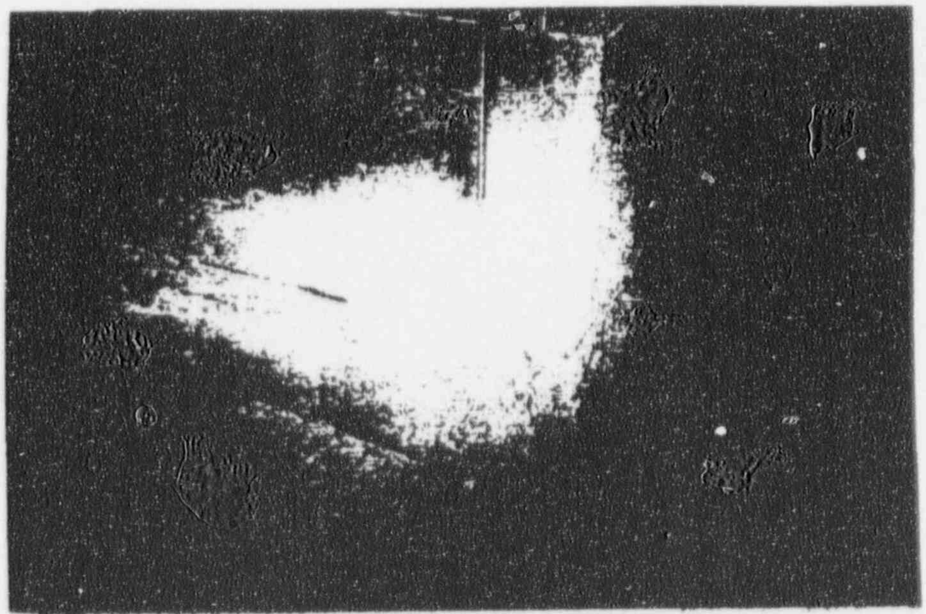


Figure A-9. Lobby stone showing salting.

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...*

REFERENCES

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.

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

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

ATTACHMENT

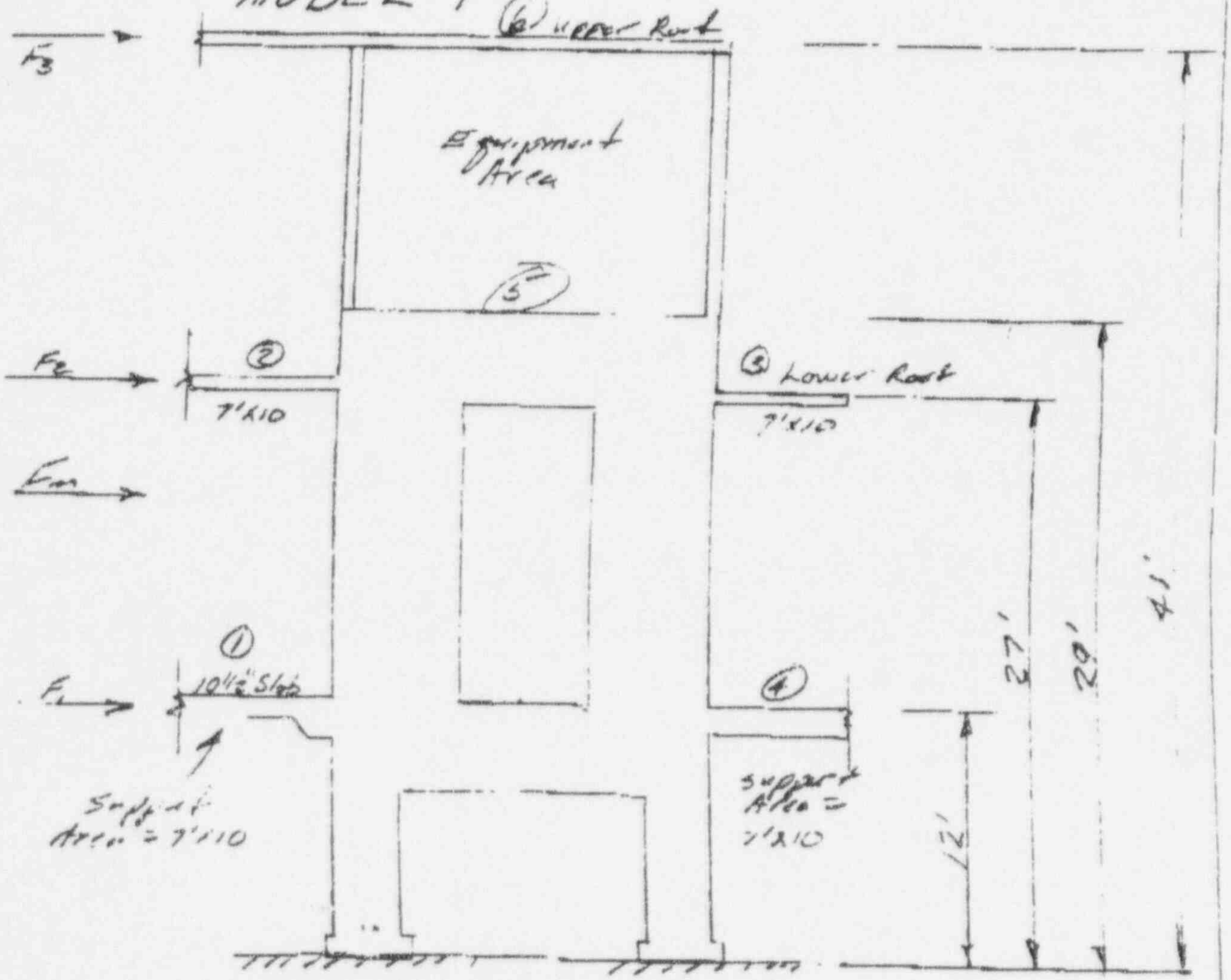
SIMPLIFIED CALCULATIONS

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 2 x 7x10 x [175+500] = 94 kips

Floor 2 = 7x10 x [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

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

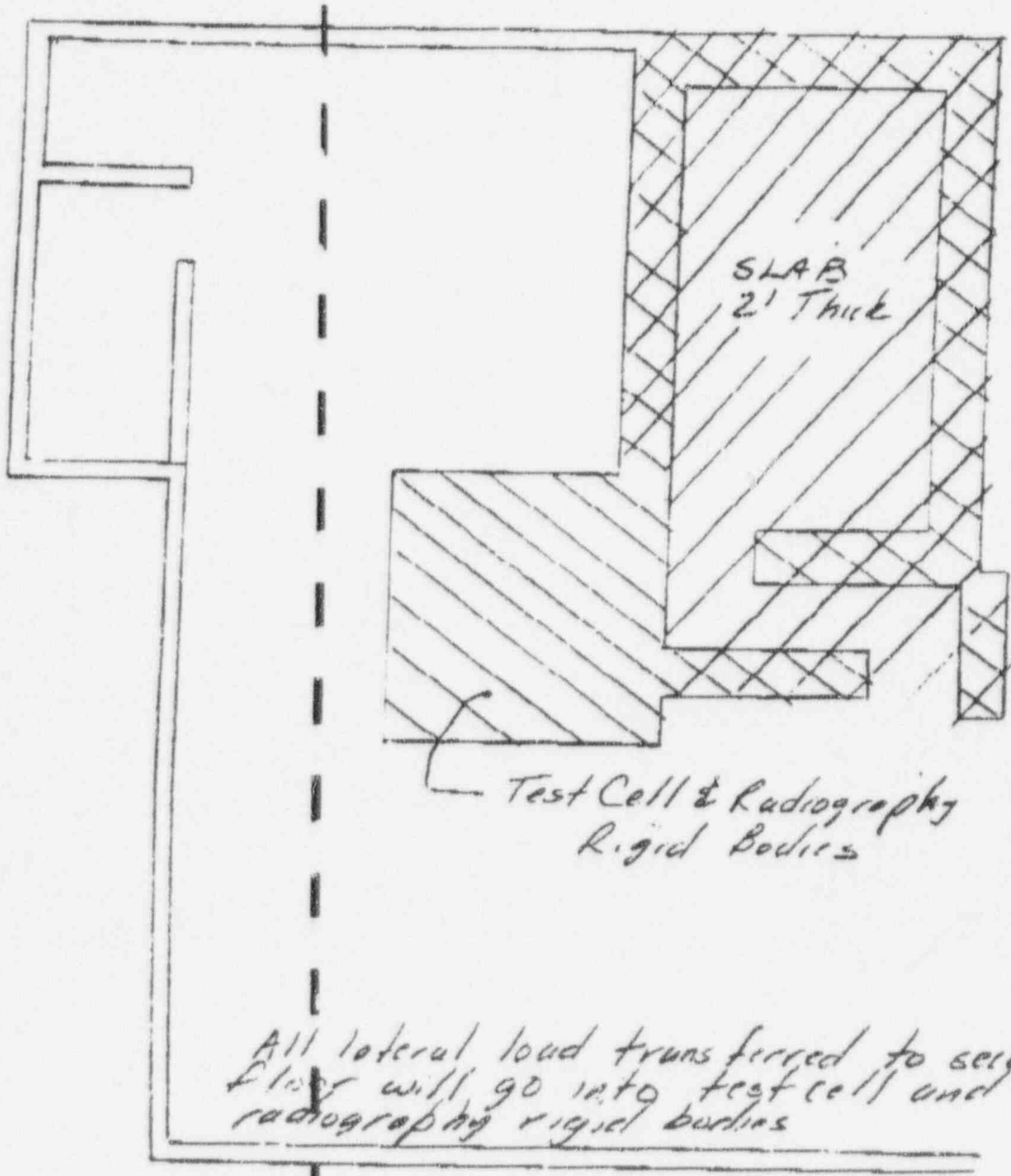
= 0.207 kips/in or 207 psi <<< 3000 psi

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

5/26/96



All lateral load transferred to second floor will go into test cell and radiography rigid bodies

Plan To Scale

Above the second floor you have a rigid roof diaphragm

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

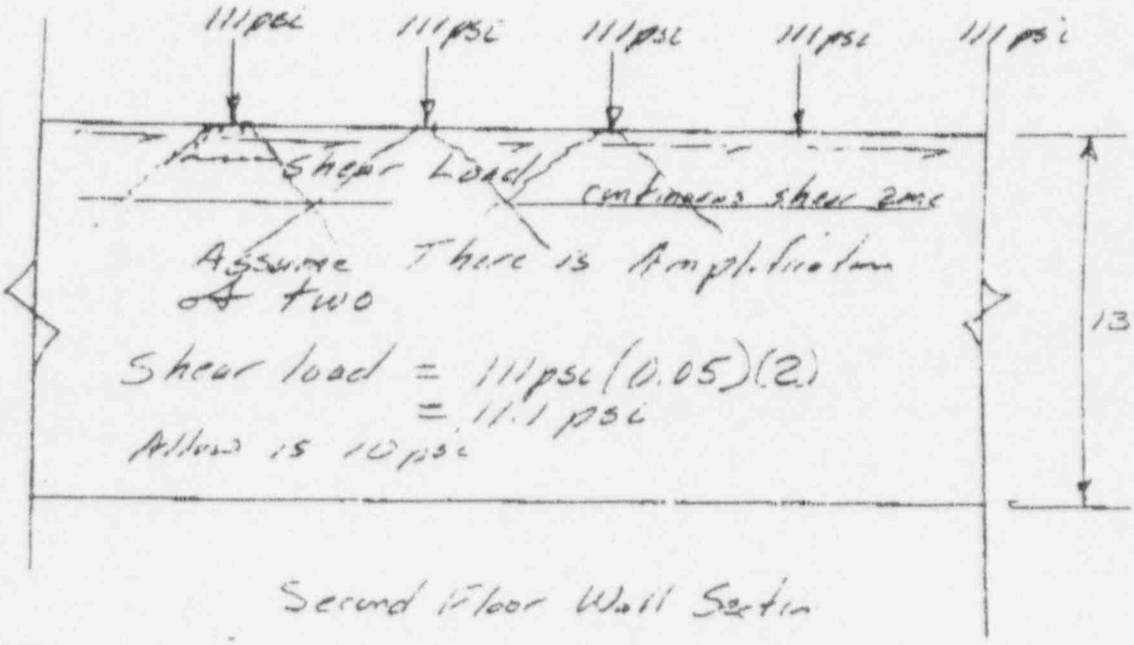
purlin load on east wall is

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

Bearing Plate Area = 7" x 3/8" x 8"

Purlins P2 = 14 WF30 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 10 psi

Gross Area Supporting Purlin Load is approximately 8 x 8' x 12" = 768 in²

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

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

For unreinforced masonry the allowable shear per the NEHRP Guidelines is

$S_{all} = 1.5\sqrt{f'_c A_n}$ $A_n = 7 \times 8 = 56$ in² plate

Assume $f'_c = 3000$ psi Actual A_n at 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) >> 80

$60 A_n + 0.3 N_v = 60(96) + 0.3(6240) =$ 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.