DEPARTMENT OF THE ARMY
SAVANNAH DISTRICT, CORPS OF ENGINEERS
P.O. BOX 889
SAVANNAH, GEORGIA 31402-0889

REPLY TO
ATTENTION OF

Design Branch

22 Aug 05

MEMORANDUM FOR Mr. Phillip Webber, Director, Chatham Emergency Management Agency, P.O. Box 8161, Savannah, GA 31412

SUBJECT: Visual Inspection of the Old Chatham County Courthouse Structure

1. At the request of Mr. Al Bungard, County Engineer, Chatham County, Messrs. Ralph Barrett, Gordon Simmons, John Roberts and Kirti Joshi visited the subject building located at 124 Bull St., Savannah, GA, on 12 Aug 05. The purpose of the trip was to gain an understanding of the county's concern regarding the structural stability of the building in the event of a category 5 hurricane striking the Savannah area. The Chatham County Emergency Operations Center (EOC) is located in the basemen of the old courthouse, with all communication lines for this center located in an attic room of the facility. This EOC must remain operational during any emergency event such as a hurricane, and the county is apprehensive about either structural or water damage that would hinder EOC operations.

2. We toured many areas within this facility where the existing structure was visible and reviewed several sets of plans for the building that were used in renovations over the past 20 years. After gaining a basic understanding of part of the structure and the areas of concern, we have concluded that the county may gain validation of the probable condition of this structure in one of two ways:

a. A complete engineering analysis can be performed on the structure of this facility to determine the exact modeled response that would be expected in a storm event, with failure areas noted (if any) and suggested upgrades identified. Because of the age of this facility, and the number of renovations that have taken place over the years, it would be very difficult to determine the exact structural framing system and connection details at each location. The material strength of many structural components would need to be verified through testing. This option would be extremely labor intensive and costly and is outside the scope of services that can be provided by the Corps of Engineers at this time.

b. A report listing existing conditions and/or deficiencies may be prepared whereby engineering judgment is used to define the most probable reaction of the structure during a storm event. It is the intention of this report to fulfill this requisite based upon our visual observations during this site visit.

3. This facility was originally built in the 1880's and appears to have been renovated many times since then. Based on a review of the available plans, it is assumed that the original structural lateral load resisting system consisted of load bearing masonry walls, with four primary lateral resisting systems in the longitudinal direction (two exterior walls and two interior walls) and several transverse walls to resist lateral loads in the shorter axis of the structure. It appears that some of the interior longitudinal load bearing walls on the 1st, 2nd and 3rd floors have been replaced in the past with a load bearing steel frame system of unknown lateral resistance capability. While it is possible that this steel frame was part of the original structure, it is unlikely based upon historical methods of construction during the 1880's. The basement appears to have maintained the four longitudinal walls.

4. The attic and roof is framed with large rough-sawn timber trusses bearing on the interior longitudinal load bearing system. Wooden roof purlins span from the peak of the roof to the interior trusses and then to the exterior load bearing masonry wall. The structure between the purlins appears to be wooden decking with an asphalt shingle roofing material over the decking. It was uncertain whether there is a cavity containing insulation in this roof structure. All the roof structure wooden members (except for the trusses) were covered by a wire mesh and cement mixture assumed to be in place for fireproofing. The roof purlins did not appear to have tie-down straps to the cap beam above the exterior walls. The cap beam was anchored into the brick longitudinal wall with bolts. The trusses had metal strap connectors on some joints, and pinned connectors on others.

5. The floor diaphragms could best be described as a hodgepodge of framing systems. Between the basement and the first floor there is one system of steel beams spanning the longitudinal walls, with brick infill arches providing support to the span between the beams. Other areas on this floor, and other floors, consisted of wooden beams spanning between the longitudinal walls, some with wooden decking between the beams and some with concrete flooring between the beams. There were several areas where renovations have been made with more conventional steel beam and concrete floor framing systems. The connections between these renovated areas and the existing structure was unclear, and it is uncertain how diaphragm shear forces are transferred in these areas. The transfer of these shear forces across the diaphragm could become a critical factor during a high wind event whereby the horizontal forces must be adequately transferred to the walls and frames.

6. The brick longitudinal walls in the basement displayed signs of deterioration due to age and moisture seepage. Mortar was weak to the touch and there were many instances of brick and mortar spalling from the wall.

7. The clock tower rises two to three stories above the remainder of the building on the northwest corner. An examination of the structure of this tower showed that the roof structure had been replaced in the past. Additionally, the stiffening beams at the roof level of this tower have been replaced with tie rods in the horizontal plane at the roof of the tower; with metal rod tie-backs to the brick at the floor level approximately 20 feet below the tower roof. These connections appeared to be the weak point of this structural system based on the reliance of the anchors into the existing brick. Cracking of the walls of the clock tower was noted around the shear re-entrant corners of the windows.

8. Based upon our site visit, it is our best engineering judgment that this structure will experience damage during a category 5 hurricane. While the structure is massive enough such that we do not foresee an overall structural failure, we believe the potential for many localized failures in the roof and/or clock tower exists. As such, the upper floors of the building would be unusable and the lower floors would experience significant water problems through openings in the roof envelope and ground water infiltration. The communication links in the attic would most certainly be compromised. This judgment is based on the following factors:

a. Current building codes for the Savannah area only require design for winds relating to approximately a category 3 storm unless specifically required to be higher due to the building function. It is assumed that building codes in the 1880's did not consider a wind load equivalent to a category 5 hurricane.

b. Preliminary calculations on wind loads during a category 5 hurricane show walls loads between 80 and 90 pounds per square foot 40 feet above the ground, and roof uplift loads greater than 200 pounds per square foot along the edges and corners (including the dormer window areas). The lack of positive connections between the roof purlins and the cap beam indicates that connection failure is likely during high wind uplift loads. It is assumed that the connections between the roof purlins and the trusses as well as the connections at the ridgeline also lack strength to withstand high wind uplift loads. We would expect several localized failures of this roof.

c. The communications room in the attic has a roof access hatch to a platform with several roof antennas. The connection of this roof hatch to the roof decking is suspect and would likely be breached with the failure of the roof-mounted antennas at that location. Flooding and wind damage in this communication room is almost certain.

d. The structure of the clock tower is suspect during a high wind event. The likelihood of failure of the upper 20 feet is high. Depending on the mode and direction of failure, the impact load from this partial collapse could cause considerable damage to this corner of the building.

e. Evidence of water damage in the basement walls leads to the suspicion that water infiltration into the basement would be likely even should there not be any roof damage. This facility is located in one of the highest points of the city with a first floor elevation of 43.5 feet (basement assumed to be 33.5 feet). However, given the high tide expected with a storm surge (25.1 feet in a category 5 storm), the likely loss of power possibly affecting sump pumps in this area, and the historical flooding of other below grade buildings in the area (i.e. the Roberson Garage), one can conclude that there is a high probability of water infiltration into the basement of this facility during a storm event. That is, the heavy rainfall expected during the storm would quickly overcome the storm drain systems in the area due to the high surge capturing system capacity.

9. A follow-up question was asked regarding the potential for damage during a category 3 hurricane. Our preliminary calculations indicate a wall load around 50 pounds per square foot 40 feet above the ground, and a roof load of approximately 120 pounds per square foot uplift. It is estimated that the dead load (self weight) of this roof is around 25-30 pounds per square foot, resulting in a net uplift of around 90 pounds per square foot during this category 3 event. This would still result in a net uplift of several thousand pounds at the purlin connections. It is our judgment that several of these connections, including those at the communications room, would fail at this load, resulting in wind and water damage on the upper levels, with the water finding its way to the basement. Without material strength tests of the bricks and mortar in the clock tower, we cannot determine nor judge whether the wind load of 50 psf would be sufficient to cause damage to the clock tower.

10. The storm surge during a category 3 storm is expected to be 18.9 feet, again capturing capacity of the storm drainage system such that rainfall would likely cause flooding and water infiltration into this facility.

11. Any questions on this report may be directed to the undersigned at 912-652-5260 or to Mr. John Roberts, EN-DAS, at 912-652-5586.

Gordon L. Simmons

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August 11, 2006

Chatham County Facilities Maintenance and Operations Division
124 Montgomery Street, Room 230
Savannah, GA 31401

Attn: Fred L. Thompson
Superintendent

Gentlemen:

As requested, we have performed a field review of the existing Chatham County Courthouse structure on Wright Square and have performed a limited wind analysis on various components. The purpose of this review was to determine the affects of and the ability to withstand wind forces imposed on the building during a Category 3 hurricane. This report was prepared at the request of Mr. Fred Thompson of the Chatham County Facilities Maintenance and Operations Division.

Part 1 - General Overview

1. Building Description:

The old Chatham County Courthouse located on Wright Square was originally constructed in 1889. It consists of a full basement, four full floors, and a partial fifth floor at the center of the building which serves as a communication room for various computer and communications equipment for CEMA.

The structure was originally constructed mostly from timber framing with some possible steel columns included in the original support system at the center of the building. The size spacing of the majority of the floor framing members are unknown due to concealment by existing ceilings, soffits, etc. The roof framing consists of a series of large heavy timber joists at varying spacings supported at the perimeter by solid brick walls and by two large heavy timber trusses on either side of the corridor down the long axis of the building (east-west direction). A cementitious type coating is applied to the underside of the majority of the timber framing and is held in place with a wire mesh lath.

At the northwest corner of the structure, a clock tower rises approximately 35 feet above the existing roof ridge and is constructed of thick brick walls supporting a pyramid shaped, timber framed roof. Adjacent to the clock tower at the northeast corner is an exterior platform at the roof level which contains a mechanical chiller.

Undoubtedly, a building of this age has been renovated many times. The most recent renovation which we are aware of occurred in 1991 and included mainly upgrades to cosmetic features of the building, however, did also include structural modifications and additions including but not limited to:

a. Addition of a double elevator from the basement to the fourth floor at the center of the building.

b. Modifications to the stairs at the east and west corners of the building.

c. Addition of fifth floor mezzanine framing for a communications/computer room for CEMA.

d. Roof platform over fifth floor level for access to a new communications antennae.

Part 2 - Wind Analysis

1. Wind Effects:

The effects which hurricane winds have on a structure and the degree of potential damage is a function of many parameters. These parameters include, but are not limited to, wind velocity, wind direction, surrounding terrain, building configuration, building height, roof slope, etc.

In the computation of the pressures which are experienced as a result of the above parameters, two types of analysis are generally performed to determine a structure's ability to withstand the lateral load effects of a hurricane.

The first type of analysis is commonly referred to as a "Main Wind-Force Resisting System" analysis, or "MWFRS". This type of analysis is used to compute pressures which are to be resisted by main lateral force resisting systems such as shear walls, rigid frames, braced frames, etc. This type of analysis typically does not consider the effects of internal pressures developed as a result of breaching of windows, doors, and other potential openings in the building envelope. At the time of design and construction of the Old Chatham County Courthouse Building, the science of wind engineering was relatively non-existent with the exception of some possible rules of thumb and general construction practices to resist wind uplifts at roof connections. The full effect of the wind forces produced on the entire building envelope and the lateral force resisting systems was not considered. The thickness of the perimeter brick walls and interior brick walls (at the basement and first floor only) would serve as shear walls to resist the wind forces computed by the MWFRS analysis. Wind forces would be transmitted to the brick shear walls at each floor level by the use of "floor and roof diaphragms". Simply put, a diaphragm is a horizontal element such as a floor or roof which serves to transmit lateral wind loads developed at the floors and roofs to the adjacent vertical resisting systems, in this case, brick shear walls.

The second type of analysis considered for wind pressures is commonly referred to as a "components and cladding" analysis. This type of analysis determines the wind effects on individual building components whose tributary areas are smaller and whose location on the building envelope produces higher wind pressures. Examples of components include fasteners, purlins, girts, studs, roof decking and roof trusses. In the analysis, the smaller tributary area, the greater the wind pressure developed. Examples of cladding include wall coverings, curtain walls, roof coverings, exterior windows and doors. Each of these two types of wind analysis are important in determining the building's resistance to hurricane winds in that the failure in either the MWFRS or the individual components and cladding can result in partial or complete failure of the structure.

2. Supporting Design Parameters:

The most commonly used methods of analysis for wind loads is that of "Minimum Design Loads for Buildings and Other Structures" (ASCE 7-98) produced by the American Society of Civil Engineers. This standard is also required to be used by the Georgia State Code which has adopted the International Building Code, 2000 Edition. The following parameters in the analysis of wind pressures on the courthouse structures are as follows:

a. Wind Speed:

As stated at the outset of this report, we have been asked to assess the effects of a Category 3 hurricane as defined by the Saffir-Simpson Hurricane Scale. Winds from a Category 3 hurricane would be in the range of 111 to 130 mph. In the calculation of wind pressures for this report, we have used the maximum 130 mph value. This wind speed is only 10 mph higher than the standard 120 mph required for the Savannah area in accordance with IBC 2000 and ASCE 7-98.

b. Building Classification:

In the computation of wind forces by ASCE 7-98, all buildings must be classified according to the use of the facility in accordance with Table 1-1. Category 1 buildings are generally described as "buildings and other structures that represent a low hazard to human life in the event of failure" while the most extreme Category 4 is described as "buildings and other structures designated as essential facilities".

Because this category classification includes "communication centers and other facilities required for emergency response", "designated earthquake, hurricane, or other emergency shelters", it is in this category that the Old Chatham County Courthouse facility would be included due to the anticipated use of the structure post-hurricane by CEMA.

c. Importance Factor:

As a result of the building classification noted in item b above, the computation of wind pressures on structures must include a "importance factor" which is determined from Table 6-1 of ASCE 7-98. The importance factor tends to assign a higher or lower value to the pressures developed based on the "importance" of the structure indicated by the building classification. In the calculation of wind forces for the Chatham County Courthouse, an importance factor of 1.15 must be used which indicates that a 15% increase above more commonly used, non-essential facilities, will be computed. It should be noted that ASCE 7-98 requires an importance factor of 1.50 for the computation of seismic loads in building category 4 structures or a 50% increase in calculated lateral loads.

d. Exposure Categories:

All buildings are classified according to ASCE 7-98 for different exposure to hurricane winds which attempts to apply higher or lower wind pressures to a structure based on the presence of and height of other structures in the adjacent terrain. For the calculation of wind loads for the Chatham County Courthouse, an exposure B is considered appropriate which describes the terrain as having "numerous closely spaced obstructions having the size of single-family dwellings or larger. Use of this exposure category shall be limited to those areas in which the terrain representative of Exposure B prevails in the upwind direction for a distance of least 1500 ft".

Typical of many model building codes, ASCE 7-98, Para. 6.5.2.1, Shielding, indicates that "no reduction in velocity pressure due to apparent shielding afforded by buildings and other structures or terrain features" is permitted. This becomes an important issue for the discussion of the Chatham County Courthouse due to the fact that there are large buildings on the north, south and east sides of the building which in reality would afford a significant amount of shielding or breaking up of direct wind velocities but is not permitted by code due to the potential that any or all adjacent structures could be removed rendering a more direct path for wind to travel toward the structure under consideration.

d. Enclosure Classification:

ASCE 7-98 indicates that for the purposes of determining internal pressures which can develop within a building envelope, all buildings must be classified as either closed, partially-enclosed, or open. Since most of Chatham County is located in what the code describes as a "wind borne debris region", all glazing in the lower 60' of Category 2, 3, and 4 buildings sited in wind borne debris regions shall be impact resistant glazing or shall be protected with an impact resistant covering. These features are required to resist breaches in the building envelope at the windows and doors which if occurred would allow increased positive internal pressures to develop from the windward side of the building envelope. This is significant for several reasons:

1) In the absence of the described glazing or protective coverings, the building must be classified as a "partially-enclosed" structure.

2) A partially-enclosed structure will develop internal wind pressures three times that of an enclosed building structure.

As will be discussed below, no such protective coverings or glazing types are currently installed on the courthouse structure.

3. Based on the above design parameters, we have calculated wind pressures on the building structure as follows:

a. Main wind force resisting systems:

Windward wall pressures =	40 PSF
Leeward wall pressures =	33 PSF

b. Components and cladding:

Exterior wall surfaces:	58 PSF
Corner wall surfaces:	69 PSF
Roof interior areas:	51 PSF
Roof edge strips:	94 PSF
Roof corner strips:	94 PSF

Part 3 - Field Observations

Our walk-through of the building structure attempted to review not only the use of the building in determining how to categorize the structure in accordance with ASCE 7-98, but also to visually assess where possible structural components of the building which either directly receive windloads or have increases in material stresses resulting from transfer of windloads to the members. It should be noted that with very few exceptions, the framing of the first, second, third, and fourth floors are concealed due to ceilings, floor finishes, soffits, etc., therefore, the condition of these framing components cannot be assessed. However, with the exception of floor diaphragms, the members in the floor framing areas described above simply function to support gravity loads only and play a negligible role in resisting wind loads on the building. The roof framing, however, becomes a major concern in the determination of the building response to wind loads due to their direct contact with the pressures developed on the roof surface. However, again, many of the roof framing members are concealed and only limited review is possible, however, based on what we were able to observe, the conditions we noted are more than likely typical of the framing for the entire roof structure.

The following is an outline of the conditions we observed during our field review:

1. We initiated our review of the structure with a review of the roof framing which based on our experience has been the most suspect for potential failure due to roof uplifts during the hurricane. Based on our analysis of the pressures which were developed on the roof surfaces during a category 3 hurricane, a calculated maximum pressure of interior roof areas of 51 psf far exceeds what we estimate as the dead weight of the roof system. This would translate to an approximate uplift load at end of the majority of the timber roof joists of approximately 2600 pounds. Based on our observations of the bearing conditions where visible, there are very few locations where any mechanical anchorage between the roof joists and the top of the walls and interior trusses which means that resistance to these high wind uplifts developed at the ends of these joists is non-existent. At the interior supports of the roof joists where they frame into hip or valley members or are supported by the large fourth floor trusses, the members are in most cases connected using "toe-nailing" of questionable uplift resistance or pull-out resistance.

   The attachment of the timber roof decking to the timber joist is also unknown. It is doubtful, however, that the attachments would be able to resist the calculated wind uplift pressure of 51 psf (interior roof zones) or 94 psf (roof edge strips and corners).

   Our review of the heavy timber trusses which span the east-west direction at the fourth floor level leads us to the conclusion that the resistance to uplift of these trusses is minimal due to the types of connections visible, i.e., steel diagonal straps. There are gaps evident at the connections between the connecting timber members. We conclude that the construction of these timber trusses only considered gravity loads of the roof and did not include any consideration for potential wind uplifts due to hurricane loads. Based on the above observations of the framing of the roof, we conclude that major failures of the structural framing in some or all of the roof framing are possible during a Category 3

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hurricane due to the high uplift pressures which would develop. The failure of the roof framing presents major problems for the structural integrity of the entire structure which include:

a. Exposure to the elements of the fourth floor in the absence of the roof would be catastrophic to the walls and contents of the fourth floor area.

b. In the absence of the roof, the rain would be permitted to enter the fourth floor level and work it's way down through the building to include potential flooding of the basement.

c. The absence of roof framing would almost certainly cause failure and collapse of the two major trusses at the fourth floor and could potentially cause collapse of the walls both at the perimeter of the fourth floor and the interior partitions.

d. Since the communications and computer equipment for CEMA is located in the fifth floor area which is inside the roof framing since the 1990 renovations, this area would be devastated in the absence of any overhead roof framing.

e. The communications antennae above the building are supported by the roof framing and would fail structurally upon failure of the building roof.

2. Floor / Roof Diaphragm:

Because of the past renovations in the floors and roof areas, the roof and floor diaphragms and their ability to properly transmit lateral forces developed at the floor and roof to the adjacent shear walls would be questionable. Because of the concealment of the majority of roof and floor decking systems, we have insufficient information to be able to conclude that the diaphragms could safely support the lateral shear and chord forces which would develop during a hurricane event and which would be needed to properly transmit these loads to the vertical shear walls. The floor and roof diaphragms toward the middle of the building have been weakened by the past renovations and it appears that due to a mixture of different types of framing systems throughout the floors, that what remains of the floor diaphragms may not perform as needed. For winds which develop in the east-west direction against the short face of the building, the diaphragm loads which might develop become less important due to the smaller surface area exposure of the building and the fact there are more closely spaced brick walls at the east and west ends of the buildings which make the diaphragms smaller and more likely to perform for winds in that direction only. For winds which develop in the north-south direction against the long faces of the building, the diaphragms will play a much more severe role in transmitting lateral loads. At the first floor and second floor levels, the diaphragms are constructed in smaller areas

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due to the fact that the brick walls are throughout the first floor space up to the second floor level. However, above the second floor, the diaphragms become much larger spanning from the east end of the building to the west end of the building with no internal brick walls available to reduce the length of the diaphragm. For wind loads in the north-south direction described, we feel the diaphragms will fail at the upper levels which could result in complete structural failure of the third floor level and above.

3. Window and Door Openings:

As previously mentioned above, the windows and doors do not have any ability to resist current code required wind-borne debris which almost certainly would be present during a hurricane of this magnitude. In the absence of impact resistant glazing in the windows or specially designed protective coverings, breaches in these openings are almost certain due to the wind-borne debris. During this event, the internal pressures of the building, which are more of a concern on the fourth floor would develop to three times the normal pressure of an unbreached building envelope due to the "partially enclosed" building condition. Breaches in the walls from the fourth floor and below have less impact on the components and cladding because they do not have any opposing forces such as developed on the roof which act in the same direction as the internal pressures.

4. Exterior Walls:

The perimeter walls consist of thick brick masonry. Should the floor and roof diaphragms sustain the wind loads developed under a Category 3 hurricane, we feel that the thickness of the perimeter walls is adequate to resist wall pressures which would be developed during the hurricane. However, should the diaphragms fail, the walls could fail either due to an increase in height between supporting floors (after the failure of the interior diaphragm) or due to instability which would occur if the top of the wall had no support from the roof framing.

5. Clock Tower:

Our observations of the clock tower from the highest available level within this portion of the structure indicates that the timber roof framing which forms the peak of the tower is insufficiently connected to the tops of the brick walls to resist any wind uplifts. It is our opinion that during a Category 3 hurricane, the roof framing on top of this clock tower would certainly fail. Due to the configuration of the clock tower, i.e., thick brick walls in a relatively small room, the remaining brick masonry walls should remain in place. There currently exists large round steel tie-downs from a series of steel beams at the clock tower roof level down to the floor framing below. We are uncertain what purpose of these tie-downs serve, however, it appears to have been integral with a previous structure for the clock tower roof and does not appear to provide any uplift resistance for the top of the clock tower. It is apparent from some of the remaining timber framing at the clock tower that major renovations to the roof structure at this area have been performed in the past.

Part 4 - Flooding

There are two potential sources for flooding of this building including the basement area as a result of a Category 3 storm. They are as follows:

1. As with any major hurricane, storm surge is probable and generally accounts for more loss of life than any other cause during a hurricane. The hurricane of 1893 which has been referred to as the "Sea Islands Hurricane" struck the Georgia coast near Savannah with 120 mph winds and reported 16 foot storm surge. The loss of life is estimated between 1000 and 2000 people, mostly due to the storm surge.

   While the current CEMA storm surge maps indicate most of downtown Savannah being protected from storm surge, the surge would isolate the area and have major impact on drainage systems in downtown Savannah.

2. Category 3 hurricanes have potential for extremely large amounts of rain. The area of the Chatham County Courthouse has historically had problems with flooding in the basements due to inadequate drainage during non-hurricane events. The rains associated with a hurricane would certainly increase the potential for some types of water intrusion or flooding in the basement area. A complete study of the storm sewer systems in the vicinity of the Chatham County Courthouse and their potential for failing during a Category 3 storm is beyond the scope of this report and outside the scope of our discipline. We would defer the discussion of this potential to a local civil engineering firm which would be in a better position to advise the County on potential problems associated with flooding.

Part 5 - Summary

As mentioned at the outset of this report, the original structure was constructed around 1889. Since the original construction of this building, there have been no Category 3 storms to directly hit Savannah. There have been four Category 2 storms to directly strike Savannah in the 1900's, the last being Hurricane David in 1979 where 92 mph maximum winds were recorded off Ossabaw Sound. These winds quickly diminished as the eyewall crossed land and therefore the downtown area did not experience hurricane force winds. The implication here is that this building has never experienced a Category 3 or higher hurricane and any conclusion which might be reached based on the longevity of the building (117 years old) would be based on a false sense of security. Newer building structures which have been completely engineered per recent building

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code requirements have been completely destroyed as a result of recent major hurricanes including Andrew and Iniki of 1992 and Hurricane Katrina of 2005.

The following summarizes the above conclusions reached in this report regarding potential for damage during a Category 3 storm:

1. We believe that based on our field observations and uplift calculations that the roof of this building including the clock tower roof would fail. This failure would have a devastating effect on the contents of the fourth floor and floor framing structures below, including the loss of the roof antenna and the CEMA communication equipment room on the fifth floor. To strengthen the existing roof would require a complete gutting of the fourth floor to expose all the connections in the timber framing including those of the two major trusses. All of the connections would then need to be retrofitted with steel plates in order to provide adequate connection between framing members and a positive mechanical connection between the framing members and the tops of the brick walls. With these members in place, a load path would then be established. The majority of failures in a roof framing system does not occur in the overstressing of the framing materials themselves but in the connections, however, with the connections properly attached and seated, there still is a potential for failure of the members due to increased bending stresses and axial stresses. As stated, the material is 117 years old, has weakened, and while appears to be performing adequately for gravity loads, has the potential for failure under wind uplift forces and the stress reversals which would occur.

   The alternative to retrofitting the existing framing would be to completely remove the roof and rebuild it with a structural steel system which could be engineered to resist the dead and live gravity loads and the high wind uplifts.

2. Based on our review of the renovation documents, the mixture of different materials used during the renovations, openings created in the diaphragms during the renovations, etc., there is a potential for failure of the floor diaphragms and roof diaphragm. As previously stated, there is insufficient information available due to concealment of these features to conclude that they would perform satisfactorily during a hurricane. Unfortunately, retrofitting a floor or roof diaphragm would be "major surgery" for this building. We believe it would be cost prohibitive.

3. As previously stated, the window and door systems have no resistance as required by code for breaches due to wind-borne debris. To provide adequate protection of these openings would require installation of impact resistant glass windows or hurricane type shutters. If this option is considered, we would recommend consultation with a vendor who specializes in this type equipment. If impact resistant windows are considered, we would suggest discussing this with one of the many window manufacturers who specialize and

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install these types of windows.

We thank you for the opportunity to provide these services and if you have any questions on the contents, please do not hesitate to contact us.

Yours truly,

W. Hunter Saussy III, P.E.
WHS/rlm

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