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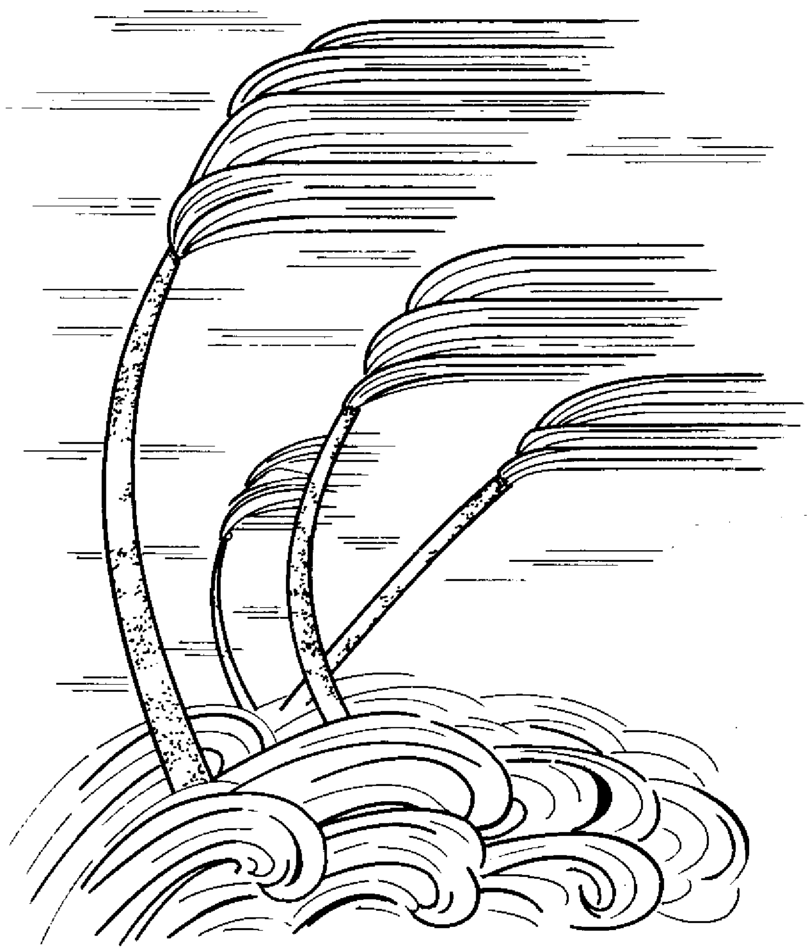
EVALUATION OF EXISTING AND POTENTIAL HURRICANE SHELTERS

Byron D. Spangler
and
Christopher P. Jones



Florida Sea Grant College Program

November 1984

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Cover Art: Face of monument dedicated to those who lost their lives during the September 2, 1935 hurricane, Upper Matecumbe, Florida Keys.

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PREFACE

Population growth in many coastal areas has led to a situation where it is no longer possible to provide sufficient warning time for everyone to evacuate. People must either evacuate before hurricane warnings are issued or risk staying behind and exposing themselves to hurricane forces. Many will take that risk.

While local governments can't guarantee the safety of those that remain, steps must be taken to minimize the risk. One option is to shelter people in buildings thought capable of withstanding the anticipated storm forces. The most desireable option here is to place people in fully engineered and well-built structures out of the influence of flooding and waves. In some cases this may not be possible and the only alternative will be to shelter people in the upper stories of buildings subject to flooding and in some cases, waves (this is called vertical evacuation).

The goal of this study was to develop a methodology for assessing the level of protection that such buildings can provide under hurricane conditions. Designated and potential hurricane shelters in the Florida Keys (Monroe County) were selected for study since this area presents us with a "worst case" situation, where vertical evacuation will be required.

It is important for everyone to realize that <u>it is not possible to make an exact determination of the level of protection that an existing building can provide under hurricane conditions</u>. There are too many uncertainties to make such a determination of anything but approximate. Uncertainties relating to the design and construction of the building and the storm forces that will act upon it tend to limit the accuracy of the findings.

The methodology presented in this report should be considered as a first step in developing methodologies for assessing the level of protection that buildings may provide under hurricane conditions. It will be refined as structures in other areas are evaluated. Care should be taken in its application. The methodology should be used only by competent professionals (those thoroughly familiar with the design and construction of buildings, storm forces and past storm damages). This report is intended to assist those professionals and not to relieve anyone of professional accountability for the design and evaluation of structures. The authors, the Florida Sea Grant College, the Institute of Food and Agricultural Sciences, the University of Florida, the Board of Regents of the State of Florida, the State of Florida, and its officers, servants, agents, or employees will not be held responsible for any and all liability, claims, demands, actions, causes of action, costs, as well as attorney's fees and court costs, arising out of or related to any loss, damage, or injury, including death that may be sustained or incurred, WHETHER CAUSED BY THE NEGLIGENCE OF THE RELEASEES or otherwise, as a result of the use of any material or methods in this publication.

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I. INTRODUCTION

Populations in coastal areas have increased dramatically in recent decades, while the incidence of hurricanes affecting our coastlines has been below the historical average. The result is very few people living along the coast have experienced the direct hit of a hurricane; fewer yet have experienced the direct hit of a major* hurricane (11). Many that think they've been through a storm have only experienced the fringes while many others have no hurricane experience whatsoever.

For example, the last major hurricane to strike the Tampa/St. Petersburg area was in 1921, when the population of the area was about 130,000. The last major hurricane to affect the lower Florida Keys was in 1948. Figures 1a through 1d illustrate this. They show the populations of selected Florida coastal counties, along with the hurricanes affecting those counties. Direct hits are indicated by a solid arrow beneath the population line while indirect (fringe) hits are indicated by dashed arrows. The numbers above the arrows correspond to the severity of the hurricanes (according to the Saffir/Simpson scale).

Another consequence of densely populated, low-lying coastal areas is that it is no longer possible to evacuate everyone within the warning time that can be provided by the National Hurricane Center (typically 12 hours or less). Recently completed regional hurricane evacuation studies (21, 27, 29) have shown that evacuation times can reach 20 or more hours, even for the approach of minor hurricanes. Residents must either evacuate before hurricane warnings are issued or risk staying behind and exposing themselves to hurricane forces. Many people will choose to remain rather than evacuate.

Unfortunately, the safety of those that remain cannot be guaranteed. One option available to local governments is to shelter people in buildings thought capable of withstanding the anticipated storm forces, thereby minimizing (<u>not</u> eliminating) the risk to those people. The most desirable alternative <u>is</u> to place people in fully engineered and well-built structures out of the influence of flooding and waves. This may not always be possible and the only alternative will be to shelter people in the upper stories of buildings whose lower floors are subject to flooding. This is termed vertical evacuation.

<u>Vertical evacuation should be used only as a last resort</u>. This should be <u>stressed to all persons living along the coast or the fear</u> of many government officials may be realized - that the designation of vertical evacuation shelters will encourage people not to evacuate. Salmon (25) discusses this and other aspects of vertical evacuation while Saffir (24) briefly describes the engineering requirements for a vertical evacuation shelter.

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* A major hurricane is defined as a category 3 or greater on the Saffir/Simpson scale (see page 2).

TABLE 1

SAFFIR/SIMPSON HURRICANE SCALE

STORM CATEGORY	CENTRAL PRESSURE MILLIBARS	CENTRAL PRESSURE INCHES	WINDS (MPH)	STORM SURGE* (FT)	DAMAGE
1	> 980	> 28.94	74 - 95	4 - 5	Minimal
2	965 - 979	28.50 - 28.91	96 - 110	6 - 8	Moderate
3	945 - 964	27.91 - 28.47	111 - 130	9 - 12	Extensive
4	920 - 944	27.17 - 27.88	131 - 155	13 - 18	Extreme
5	< 920	< 27.17	> 155	> 18	Catastrophic

* Storm surge elevations are averages - they will be higher in some locations and lower in others.

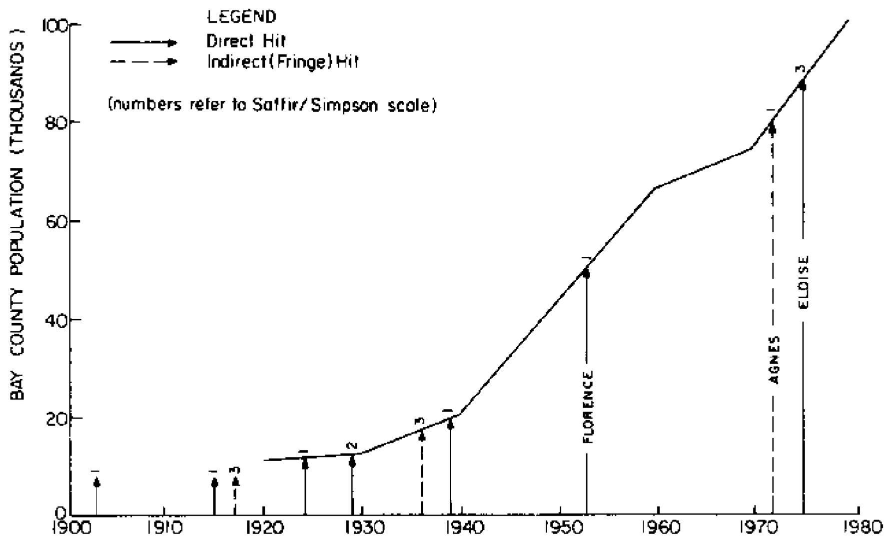


Fig Ia. Hurricane Experience of Bay County Population (adapted from reference II)

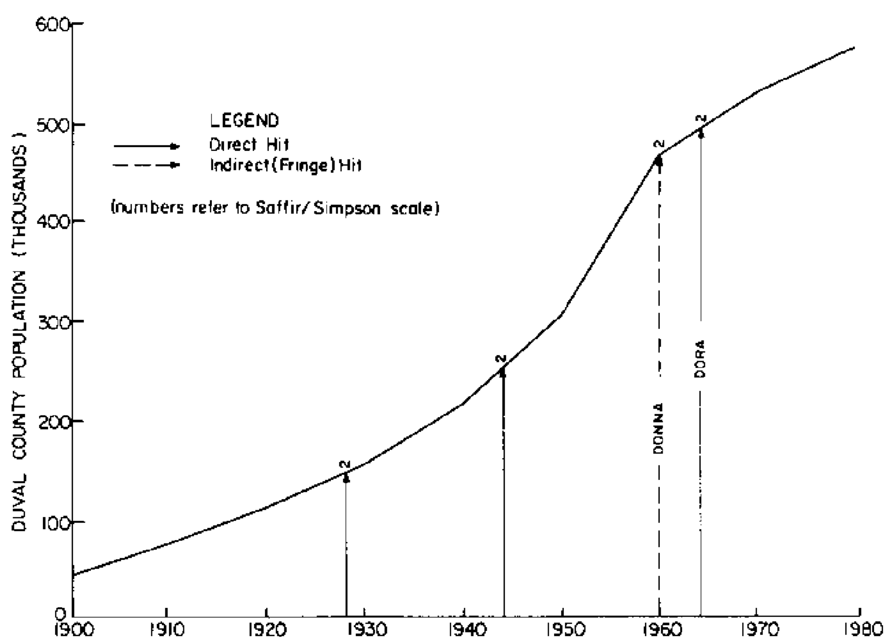


Fig Ib. Hurricane Experience of Duval County Population (adapted from reference II)

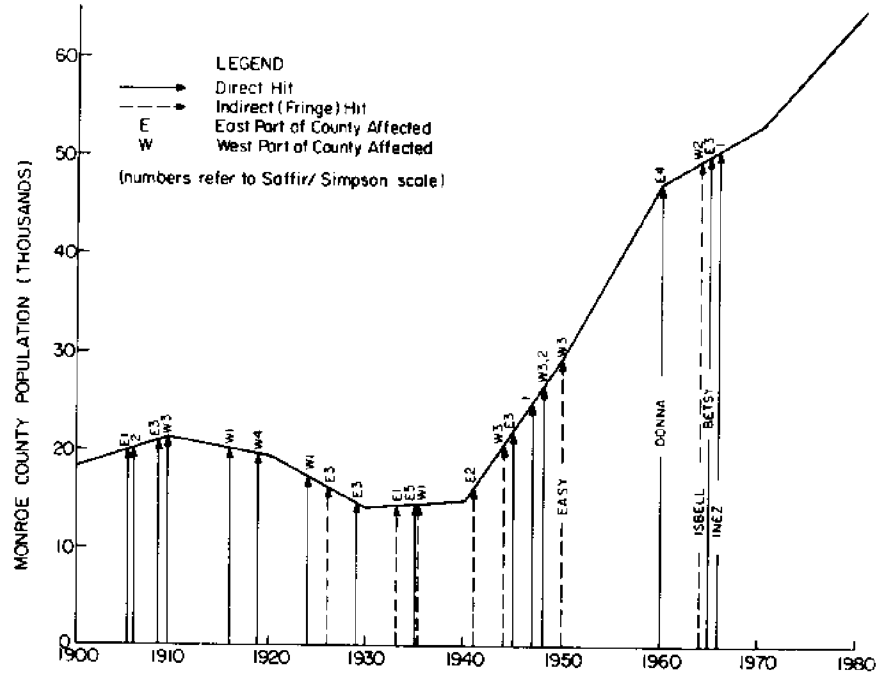


Fig. Ic Hurricane Experience of Monroe County Population (adapted from reference II)

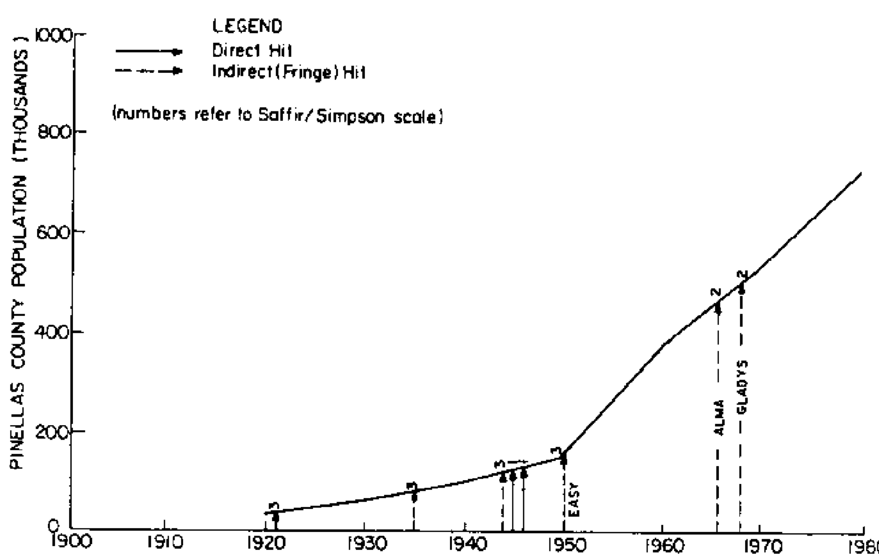


Fig Id. Hurricane Experience of Pinellas County Population (adapted from reference II)

This report presents a methodology for the structural evaluation of hurricane shelters situated in areas subject to hurricane forces. It was developed during a study of designated and potential hurricane shelters in the Florida Keys (Monroe County), where vertical evacuation will be required in some instances.

One finding of the Monroe County study (12) is that many of the buildings that are or might be designated as hurricane shelters there have never been subject to hurricane conditions. It is probable that this same situation will exist in other coastal areas as well, where schools, public buildings, churches and other buildings typically used as shelters have been built very recently.

This is of concern, given the performance of these types of structures when they have been affected by hurricanes. A damage assessment after Hurricane Frederic (33) revealed that schools, churches, public buildings and hospitals sustained extensive damage. Primary and secondary schools in six coastal counties (Harrison Co., Mississippi to Santa Rosa Co., Florida) received over $17 million in damage. Hospitals in Mississippi and Alabama sustained over $3 million in damage.

Other investigators have examined wind damage following storms and found that marginally engineered buildings - those that have received limited engineering attention and have been built with some combination of masonry, light steel framing, open-web steel joists, wood framing and wood rafters - typically sustain wind damage, even in wind regimes slightly less than code specified values (18, 20). Again, this is of concern since many shelters can be classified as marginally engineered buildings.

Experience shows, however, that buildings can withstand hurricane forces if they are designed and constructed properly. There are numerous articles in the literature that point out design and construction techniques to be avoided and those that have a high probability of withstanding extreme events (7, 8, 22, 23, 28, 34). Simple precautions prior to a storm (installing hurricane shutters, for example) can also increase the likelihood that a building will survive hurricane conditions, or at least that damage will be minimized.

Predicting the response of existing buildings to hurricane forces, however, is more difficult than designing a new building to withstand the same forces. Unless detailed "as-built" plans, specifications and other information on the design, construction and maintenance of an existing building are obtained, there will be some uncertainity in the prediction. Unfortunately, it was found during the Monroe County study that obtaining complete information on a particular building is nearly impossible. Problems that were encountered in collecting information on shelters there, and that may be encountered when shelters in other areas are evaluated, are listed below:

1. Plans were not available for some buildings.
2. Some plans obtained were incomplete.
3. Many buildings were not built in accordance with plans obtained (very few sets of "as-built" plans were obtained). In some instances the deviations from the plans on hand were minor, while in some instances they were significant, with major structural differences between the plans and the buildings.

4. In instances where buildings deviated from the plans on hand, inspection reports, change orders, etc. that might explain the variance were rarely found.
5. Specifications were rarely available.
6. When questions about the plans or construction techniques arose, attempts were made to contact the designer and/or contractor. This was impossible in some cases. Some individuals had moved and could not be located, some were deceased and some firms had gone out of business.
7. Maintenance records and information on building modifications subsequent to its original construction were rarely found.
8. On-site inspections of buildings were limited in some cases because of interference with normal building operations.

II. METHODOLOGY FOR EVALUATING SHELTERS

General Procedures

It is very important to determine, as accurately as possible, the resistance of a building to storm forces when that building will be used as a shelter. If the resistance is underestimated, the use of that shelter under some conditions will be lost. If the resistance is overestimated, the building may sustain unanticipated damages and occupants may be injured or killed.

Unfortunately, there are several factors that tend to limit the accuracy of a determination of the structural resistance of an existing building. Foremost among these are uncertainties relating to the design and construction of the building. Design information is usually incomplete, even when plans are available. The degree to which the contractor deviated from the plans (intentionally or unintentionally) may not be apparent during an on-site inspection of the building. The exact properties of the materials used for construction are usually unknown unless extensive tests are performed. A history of building modifications, maintenance and repairs is desirable, yet rarely available.

In addition, storm forces and the response of a structure to those forces are not fully understood. Small-scale spatial and temporal variations in the wind field at a building cannot be predicted accurately. Storm tide elevations can be predicted in the gross sense, but actual elevations will fluctuate about predicted values because of variations in the wind field and localized topographic conditions.

Despite these problems, the uncertainty in the determination of structural resistance can be minimized. This can be done by collecting as much information about the building and site as possible, inspecting them carefully and obtaining the best available predictions of storm forces at the building site. The performance of similar structures during other storms can also provide important information.

Culver, et al. (6) developed a methodology for the survey and evaluation of existing buildings subject to earthquake, hurricane wind and tornado forces. They developed three ways of estimating damage: a qualitative approach based upon a field survey, an approximate analytical approach and a detailed analytical approach using a computer model of the entire structure being examined. Mehta, et al. (17) also developed a methodology for predicting potential wind damage to existing buildings, using either a subjective approach based on an on-site inspection or an analytical approach involving a structural analysis based upon a knowledge of the wind-structure interaction and the strengths of the materials used in the structure. Neither methodology includes the effects of flooding and waves.

Similar procedures were developed during the Monroe County study but the effects of flooding and wave forces were taken into account. In cases where plans were not available, the procedures resembled those of the qualitative or subjective methods. In cases where plans and other information were available, the procedures resembled the analytical methods mentioned above. Unless noted otherwise during an inspection, it was assumed that the construction materials and methods were in accordance with the plans.

Summarizing, the general procedure to be used when hurricane shelters are evaluated is as follows:

1. **Identify Potential Shelters** - this should be based on location, elevation, type of construction, etc.
2. **Collect Information** - obtain plans (as-builts, where possible) and specifications; locate the building designer and contractor; obtain flood hazard data.
3. **Inspect the Building** - check all structural systems and connections, where possible; note any deviations from the plans and any defects or problems; obtain samples of materials and perform tests on building components, as required; photograph and document the building and its condition.
4. **Inspect the Building Site** - check for exposure to wind and proximity to water; photograph and document adjacent structures or vegetation that may shield or damage the building; determine the true elevation of the building site and building.
5. **Analyze all Information** - this may involve a few simple calculations or sophisticated techniques, depending upon the amount of information available and the complexity of the structure.
6. **Rate the Building** - determine the level of protection that the building can provide, assuming that extreme (i.e., tornado) conditions do not accompany the hurricane. Note any special precautions or repairs that must be made before the building can be used as a shelter.

The time required to carry out the procedure described above will depend upon the difficulty in collecting information, the size and condition of the building, and the type of analyses performed. During the Monroe County study collecting information for each building required from a few minutes (when plans and other data were readily available) to several hours, sometimes with little success. A team of two engineers (one structural, one coastal) spent between one hour and ten hours inspecting each building and site, with the average being approximately three to four hours. The analyses and reporting for each building required an average of approximately one to two man-days.

Collecting Building and Site Information

A standard form listing what the investigators consider to be the minimum information needed to evaluate the structural resistance of a building is shown on page 10. It is similar to forms developed by other investigators (6, 14). There are several items that are not included that are necessary from the standpoint of overall shelter suitability: available space, emergency power source, kitchen facilities, emergency supplies, restroom facilities, shelter manager, etc. These can be added easily.

The following paragraphs are intended as a commentary on the form and its use. Specific items and problems to look for as the form is completed for a shelter are listed below:

A. **GENERAL DATA.** This section will provide background information on the building and the site.

number of stories - of value in determining the possibility of vertical evacuation.
building height above grade - the height of the building is used to determine the wind and wave forces for overturning and sliding potential.
grade elevation - used in determining depths of flooding and maximum wave heights that can be expected at the site.
code used - information from the code is used to determine live loads and wind loads used in the design if they are not indicated on the plans.
design data - used in the structural evaluation to determine the resistance to storm forces (includes live loads, wind loads, soil bearing capacity, materials strength data, etc.).
designer and contractor - it may be necessary to contact them if the owner and building official can't supply plans; it may be necesary to contact them for answers to questions about the building and details of construction if they are not shown on the plans.
exposure - note any structures or vegetation that may shield the building from winds and waves, or that may damage the building by supplying windborne or waterborne debris; look for trees, towers, etc. that may fall on the building.
flood hazard data - collect this from all available sources (FEMA, Corps of Engineers, etc.) and compare the data for discrepancies; this information is used in determining depths of flooding and wave forces on the building.
type of investigation - indicate if plans (incomplete, complete, as-builts) were reviewed, the thoroughness of the inspection and if contact was made with the designer or contractor.

B. **FOOTINGS.** Indicate the following for both column and wall footings:

type - spread, thickened slab, driven pile, auger pile, etc.
elevation - this is particularly important in areas subject to scour.
condition - where visible, check for cracks, spalling, exposed reinforcing steel, etc.; check for signs of settlement.

C. **COLUMNS.** Indicate the following for both exterior and interior columns.

type - tie columns, reinforced tied columns, filled block cells, steel columns, etc.
connection to structural system - check the connections to intersecting members for continuity and structural stability.
condition - check for cracks, spalling, rust, etc. (Figure 2).

SHELTER SUMMARY FORM

STRUCTURE LOCATION

A. GENERAL DATA:
   1. DATE OF CONSTRUCTION:
   2. BUILDING TYPE/STRUCTURAL SYSTEM:
   3. NUMBER OF STORIES:
   4. BUILDING HEIGHT ABOVE GRADE:
   5. GRADE ELEVATION:
   6. CODE USED:
   7. DESIGN DATA:
   8. DESIGNER AND CONTRACTOR:
   9. EXPOSURE:
   10. FLOOD HAZARD DATA:
   11. TYPE OF INVESTIGATION:

B. FOOTINGS:
   1. COLUMN FOOTINGS:
      a. TYPE:
      b. ELEVATION:
      c. CONDITION:
   2. WALL FOOTINGS
      a. TYPE:
      b. ELEVATION:
      c. CONDITION:

C. COLUMNS:
   1. TYPE:
   2. CONNECTION TO STRUCTURAL SYSTEM:
   3. CONDITION:

D. BEARING PARTITIONS:
   1. TYPE:
   2. CONNECTION TO STRUCTURAL SYSTEM:
   3. CONDITION:

E. FLOORS:
   1. FIRST FLOOR:
      a. TYPE:
      b. CONNECTION TO STRUCTURE:
      c. ELEVATION:
      d. CONDITION:
   (*repeat as needed for upper floors*)

F. ROOF:
   1. TYPE:
   2. STRUCTURAL SYSTEM:
   3. ELEVATION:
   4. CONNECTION TO STRUCTURAL SYSTEM:
   5. CONDITION:
   6. DRAINAGE:
      a. SCUPPERS AND DRAINS:
      b. CONDITION:
      c. POTENTIAL FOR STANDING WATER:

G. EXTERIOR WALLS:
   1. TYPE:
   2. CONNECTION TO STRUCTURAL SYSTEM:
   3. OPENINGS (*list separately for each floor*):
      a. WINDOWS:
      b. DOORS:
      c. OTHER:
   4. CONDITION:
   5. OPENING PROTECTION:

H. PROJECTIONS:
   1. STRUCTURAL:
      a. TYPE:
      b. CONNECTIONS:
      c. CONDITION:
      d. HAZARD POTENTIAL:
   2. MECHANICAL:
      a. TYPE:
      b. CONNECTIONS:
      c. CONDITION:
      d. HAZARD POTENTIAL:

I. OTHER
   1. SCOUR POTENTIAL:
   2. RESISTANCE TO SLIDING/OVERTURNING:
   3. INTERIOR SHELTER POTENTIAL:

J. COMMENTS:

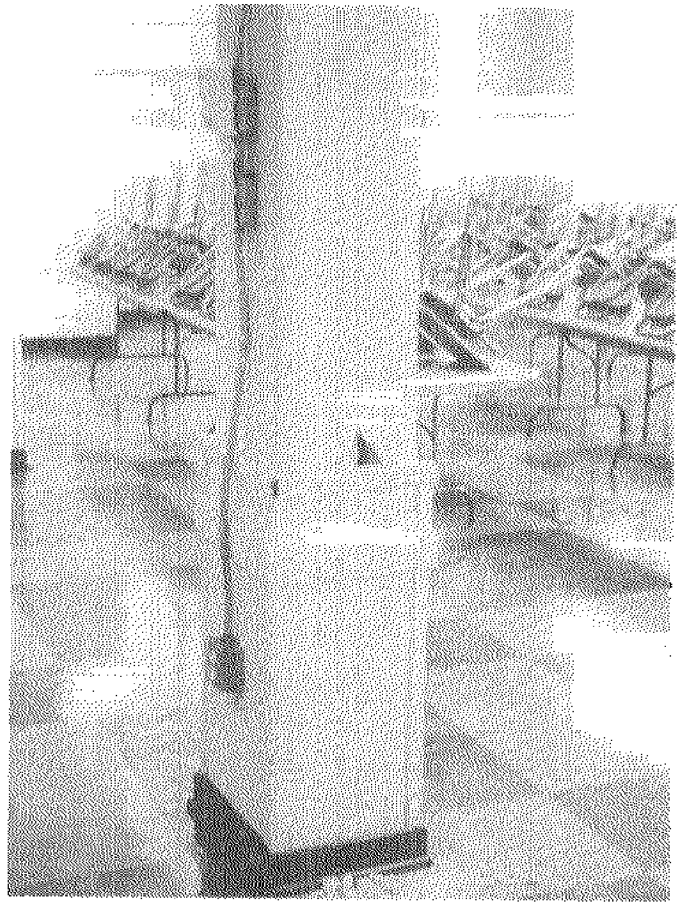


Figure 2. Deterioration of Interior Column

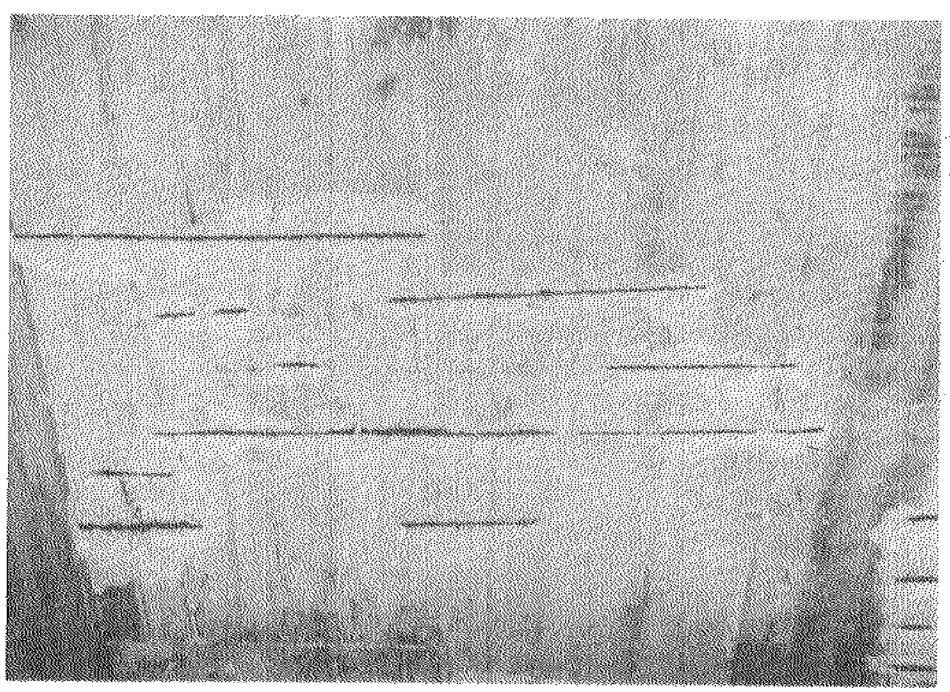


Figure 3. Spalling on Underside of Slab

D. **BEARING PARTITIONS.** Check to see if upper stories and the roof depend on interior partitions for support. Can the partitions withstand anticipated pressures if windows or openings in exterior walls fail?

E. **FLOORS.** Repeat the following for all floors.

type - wood, concrete slab on grade, reinforced concrete structural slab, etc.; indicate the method of support (on grade, open-web steel joists, etc.); for concrete slabs indicate thickness and type of reinforcement.
connection to structure - this is particularly important in areas subject to scour; a slab on grade not connected to the structure will collapse if the underlying fill is eroded.
condition-check the underside and top for problem areas which can include cracking and spalling in concrete, termites and rot in wood (Figure 3).
elevation - important in determining the possibility of flooding.

F. **ROOF**

type - indicate the shape of the roof and the type of roofing and sheathing.
structural system - reinforced concrete slab, double tees, open web steel joists, timber trusses, etc.
connections - check to see if the roofing and sheathing are fastened properly to resist peeling off; check to see that the roof system is connected to the building frame to resist uplift; check for shear connections allowing the roof to provide diaphram action if required (Figure 4).
condition - check for signs of deterioration, soft spots, holes, condition of anchorage, etc. (Figure 5).
drainage - check the capacity and condition of all drains and scuppers; check for roof overload if drains and scuppers become clogged, i.e., look at the height of the parapet that can retain water (Figure 6).

G. **EXTERIOR WALLS**

type - concrete block (indicate thickness), poured concrete, precast insulated panels, wood frame, glass, etc.
connection to structural system - walls subject to lateral loads must be designed to resist those loads and tied to the columns or pilasters to resist shear forces; ties may be ladder or truss-type wire reinforcement extending into columns, galvanized sheet metal ties in keys in the columns or other shear connectors designed properly. Masonry walls without adequate ties and unreinforced masonry walls do not withstand strong wind forces or wave forces.
openings - list windows, doors and other openings separately for each floor; sizes are needed if they are to be shuttered.
condition - check the condition of the opening frames and their attachments to the walls (wood door jambs and window frames are subject to termite attack, rot, etc.)
opening protection - inspect any existing covers or shutters for their ability to protect glass from small missiles

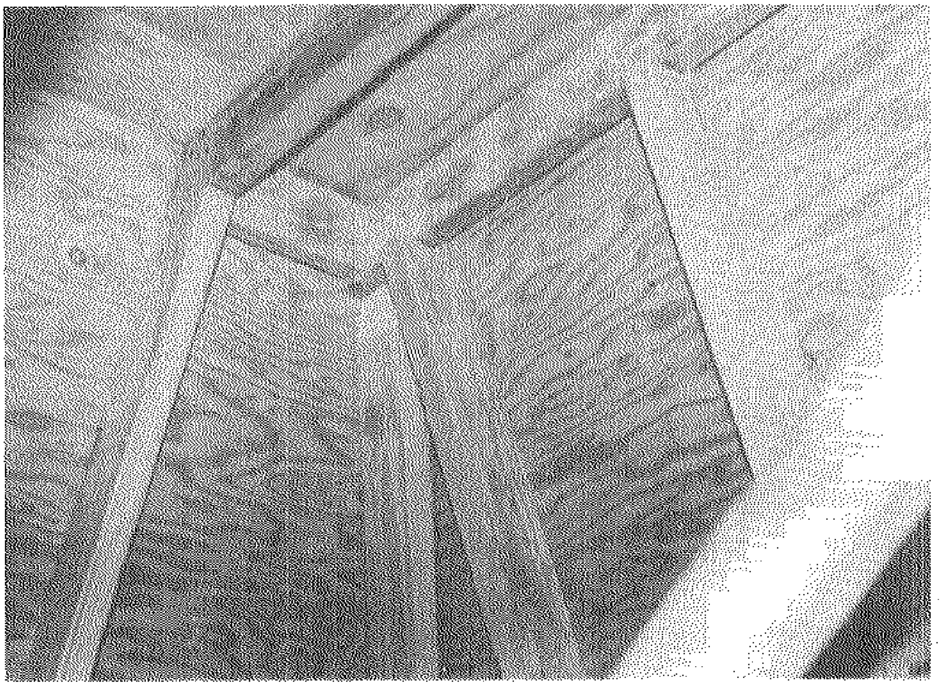


Figure 4. Timber Roof with Inadequate Fasteners

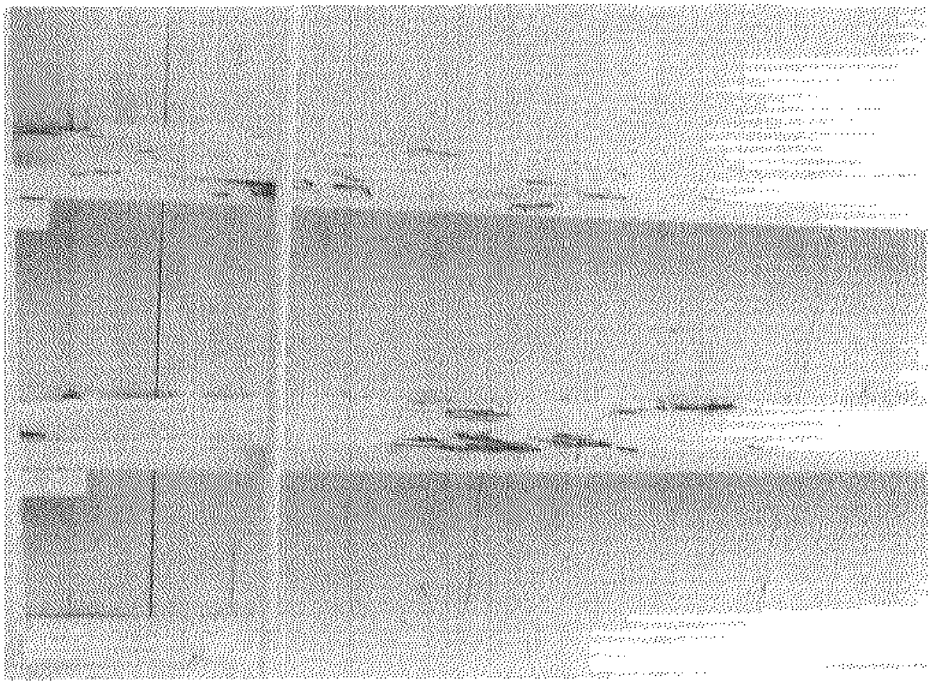


Figure 5. Deterioration on Underside of Timber Roof

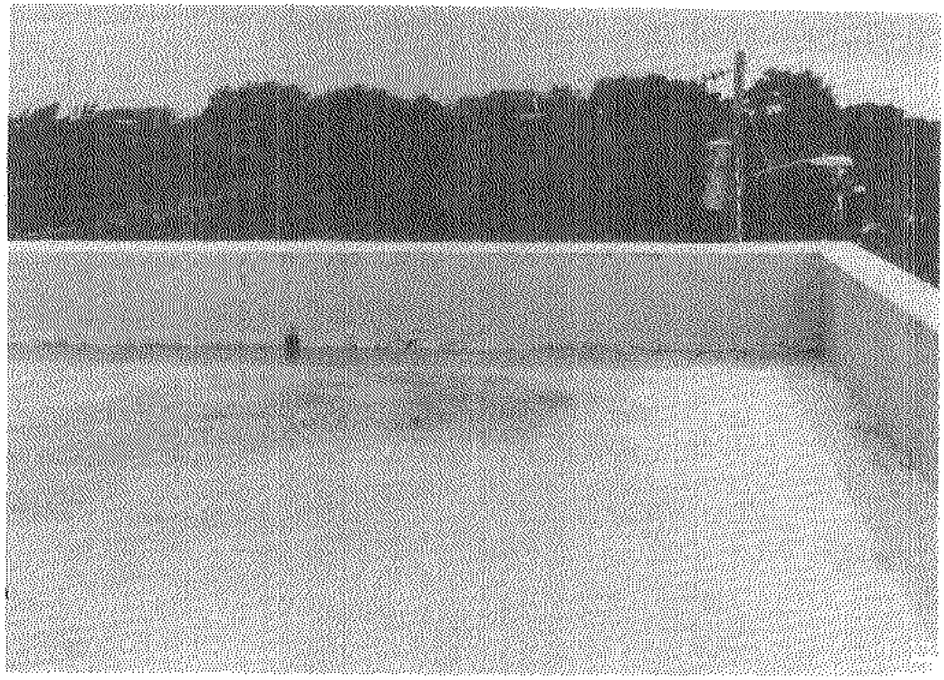


Figure 6. Parapet that Will Trap Water if Scuppers Clog

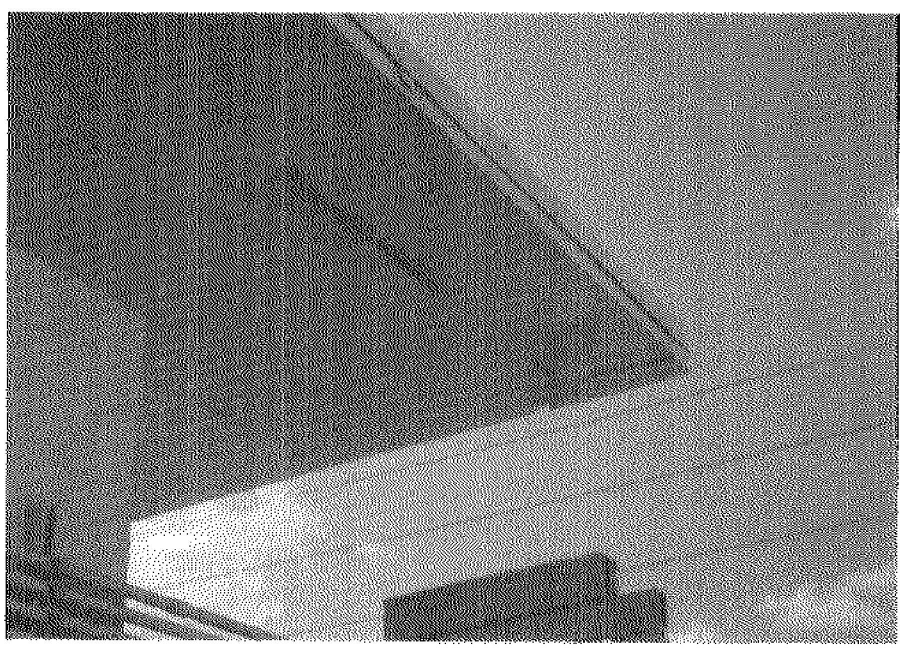


Figure 7. Deterioration on Underside of Roof Overhang

(including roofing gravel) and large objects; check existing covers or shutters for adequate attachment to the structure (whenever possible, they should be attached to the structural frame or walls, not to window or door frames); they should be able to withstand both positive and negative pressures.

H. **PROJECTIONS.** Check both structural and mechanical projections for their condition and their connection to the structure (especially resistance to uplift). If any projections fail, will this jeopardize the building or a part of the building?

structural - includes projections of structural elements (rafters, trusses), eaves, slab overhangs, ramps, stairs, etc. (Figure 7).

mechanical - includes air conditioning units, air handlers, fans, etc. (Figure 8).

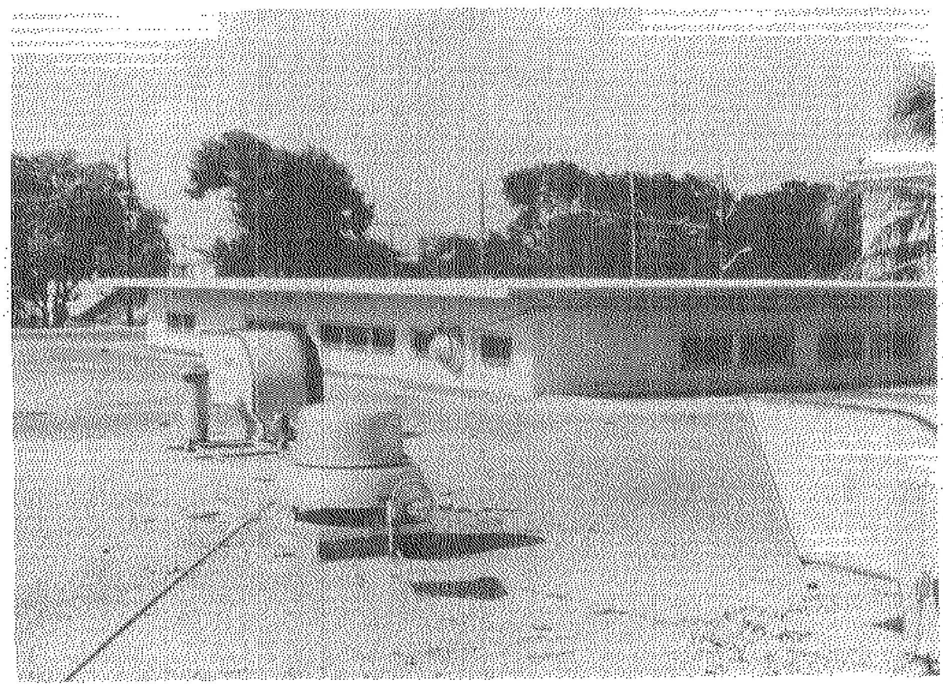


Figure 8. Mechanical Projections that Could Allow Rainfall to Enter Building and Could Damage Clerestory Windows if They Fail.

I. **OTHER**

scour potential - is the building situated so that fill can be eroded beside it or beneath it? If so, will this cause foundations or floor slabs to fail? Does any portion of the building rely upon a retaining wall for protection of its supporting fill? If so, determine the strength and condition of the retaining wall. Under what conditions would the retaining wall be expected to fail?

resistance to sliding or overturning - consider both wind and wave forces; consider the possibility of hydrostatic pressure lifting the structure, or reducing its resistance to lateral forces.

interior shelter potential - identify and list any interior areas (i.e., without exterior walls) that may provide shelter if portions of the exterior fail.

J. **COMMENTS.** Some structures may present unusual characteristics that don't fall into the above classification. Point these out here, as well as any areas of particular concern.

Resistance of Shelters to Storm Forces

Storm forces can be divided into two broad categories: wind forces and water forces. The former includes positive and negative wind pressures and the effects of wind driven missiles. The latter includes flooding, hydrostatic forces (including flotation), hydrodynamic forces (including scour), breaking wave forces and the effects of water-borne debris. Estimates of wind and water forces should be tied to the Saffir/Simpson scale (page 2) since evacuation plans and decisions are usually based upon the category of the hurricane approaching an area.

Wind speeds and storm tide still water levels should be taken as the maximum that can occur for a given category storm. Wave heights should be taken as the maximum that will likely occur. This approach is necessary since the exact point of landfall of an approaching storm cannot be predicted. It must be assumed that the shelter will be subject to the most severe conditions.

The resistance of a shelter is defined as the highest category storm where it can withstand both wind and water forces without jeopardizing the occupants. Note that this definition does not exclude the possibility of minor damage to the structure, as long as the major structural elements (foundation and structural frame) and protective elements (walls, openings, roof) are intact. Thus, the use of a shelter will be limited by the lesser of its resistance to wind forces and its resistance to water forces. This is an important point. A building safe from flooding, current and wave effects during a category 4 storm will be of no use under those conditions if the structure can only withstand category 2 wind forces; a single story building that can withstand category 3 wind forces will be of no use if it is flooded during a category 1 storm.

Nevertheless, it is useful to determine and list a shelter's resistance to wind and water forces separately. There may be instances where a shelter can be strengthened or modified so that its resistance against the limiting force can be improved.

Determining a shelter's resistance to storm forces may involve a few, relatively simple calculations or may involve a more detailed structural analysis. The more detailed analysis may be needed in the following instances:

1. Where the code under which a structure was designed is unknown.

2. Where an on-site inspection reveals questionable construction practices.
3. Where modifications or additions have been made to the original structure, changing the loads on it.
4. Where an on-site inspection reveals deterioration of portions of the structure.
5. Where the structure appears to have a greater structural capacity than indicated by the plans and/or design data.

Wind Forces

Most coastal counties in the State of Florida have adopted the Standard Building Code (SBC). In those counties, structures must be designed to withstand wind loads computed using the methods prescribed therein or by methods resulting in a greater factor of safety. Section 1205 of the SBC specifies wind loads according to the following relationship.

$$P = 0.00256\ V^2 \left(\frac{H}{30}\right)^{2/7}$$

where:

P = velocity pressure at height H (lb/ft^2)

V = fastest-mile wind speed at 30 ft. above grade (mph)

H = height above grade (ft)

The basic velocity pressures are then modified by shape factors (also called pressure coefficients) and other factors. Shelters can be analyzed using the above relationship, with allowance for internal pressures and any other factors deemed necessary. Different building codes may present variations on the above.

It should be pointed out that section 1205 is based on the requirements of ANSI A58.1-1972 (1), with some modifications. Although the basic wind load pressures in section 1205 are based upon the smooth terrain roughness category (exposure C), they do not account for gust response (15). Thus, the wind pressures are somewhat less than those in the ANSI standard.

It should also be pointed out that the more recent ANSI A58.1-1982 (2) revises pressure coefficients and adds a new terrain roughness category (open water or coastal - exposure D), which can result in even higher wind pressures near the shoreline than the SBC, section 1205, and the ANSI A58.1-1972 standard (16).

Section 1206 of the SBC (for low rise buildings) is based upon much of the same data that the ANSI revisions are based on, but like section 1205, is based on exposure C. Engineers may employ either section 1205 or section 1206 for certain buildings, but should not mix basic wind speeds or pressure coefficients from the two sections (26).

Studies of wind damages to buildings (19, 20) point out several things that should be examined carefully when the wind resistance of a shelter is determined. These include:

flat, built-up roofs - these roofs are susceptible to failure, especially near windward and leeward edges and corners. In cases where the roof insulation, tar and gravel are supported on corrugated sheet metal forms welded to open web steel joists, check the welds between the forms and joists carefully (Figure 9).

timber truss roofs - these roof systems are susceptible to uplift failures unless hurricane clips, straps or other connectors are installed. Shingles and sheathing are susceptible to failure unless they are properly nailed, screwed, wired or stapled to supporting members (Figure 10).

overhangs and eaves - these areas are susceptible to uplift failures. Check for adequate connection to the main structure.

masonry walls - if masonry walls are not reinforced properly and tied to adjacent columns they can blow in or out, depending upon wind pressures. Horizontal truss-type reinforcement and ties to columns are necessary for them to withstand lateral loads.

window and door glass - unprotected glass is extremely vulnerable to flying debris. Even roofing gravel can break glass when it is propelled by hurricane winds; in some areas this has been found to be the dominant cause of glass failure (10). Some studies have found that the strength of glass decreases significantly with age (4), further pointing out the need for protection.

window and door frames - in many instances the design of frames, especially their attachments to walls, or their deteriorated condition have led to failure during high winds.

shutters - shutters are important, not only because they protect glass or other vulnerable areas, but also because they can prevent undesireable internal pressure increases in buildings. Such pressures can increase the likelihood of roof or wall failure. Shutters should be able to protect against large and small debris. Expanded wire mesh grills (Figure 11) are <u>not</u> adequate since they allow small debris to break glass and since wind pressures still act on the areas behind the grills. Shutters (Figure 12) should be attached to the main structure and not to the window or door frame (to avoid the problems mentioned above).

overhead folding doors - these doors have been damaged frequently during past storms, in some cases leading to progressive damage to structures (10, 18).

Water Forces

Shelters should be evaluated for their susceptibility to flooding and their resistance to hydrostatic, hydrodynamic, breaking wave and floating debris effects for various storm categories.

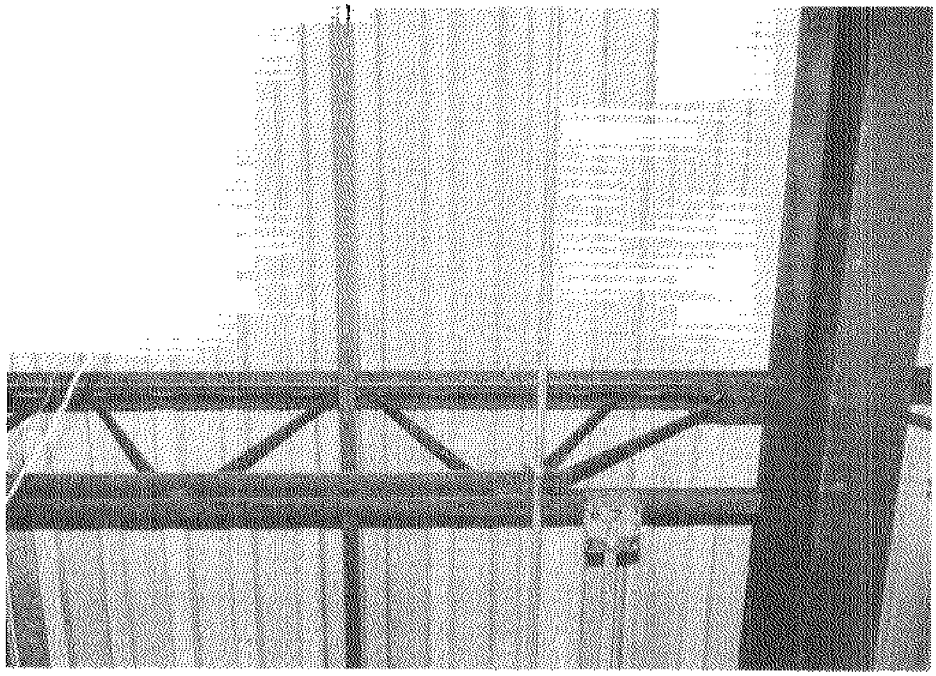


Figure 9. Check Welds Between Joists and Corrugated Forms

Figure 10. Roof Tiles Were Not Fastened Properly and Were Removed By Hand

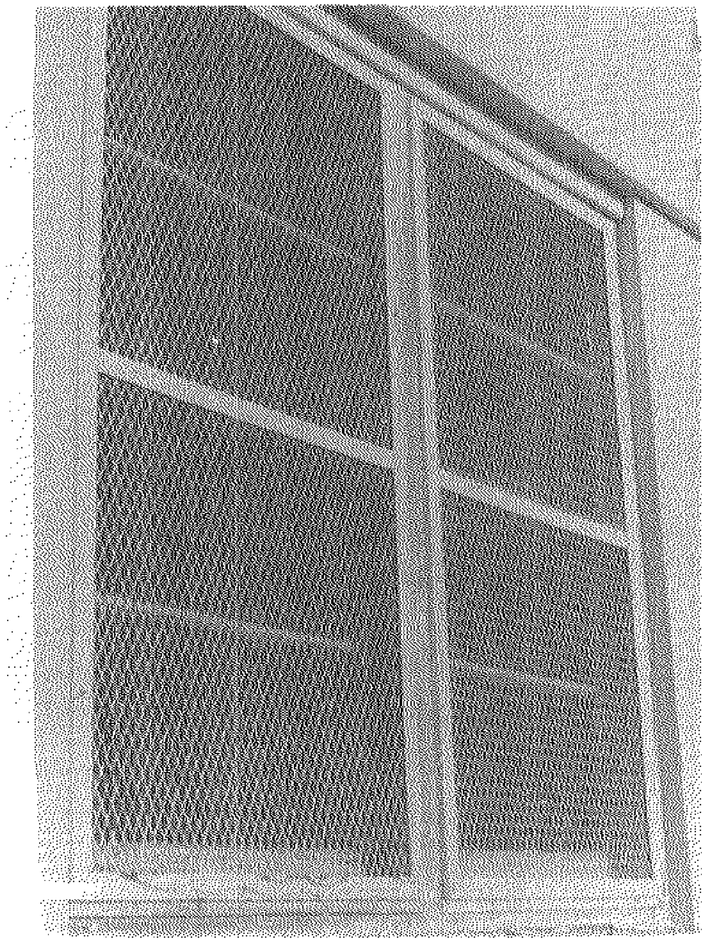


Figure 11. Expanded Mesh Grills Will Not Protect Windows Against Small Missiles

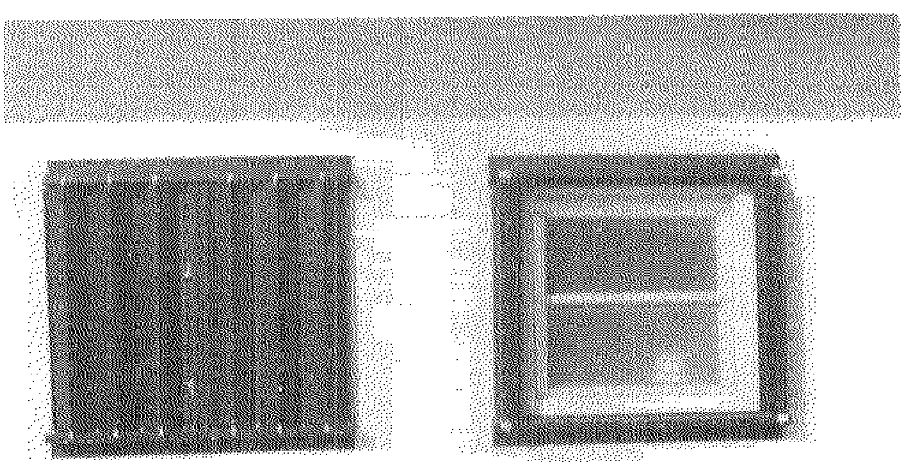


Figure 12. An Example of Shutters Properly Fastened to Structure

When single story shelters are evaluated, their resistance against water forces during a given category storm should be considered inadequate if any of the following can occur:

1. If water can rise above the level of the floor.
2. If the building foundation or floor cannot resist scour by currents or waves without settlement.
3. If the building cannot withstand anticipated breaking wave forces acting in conjunction with wind forces.

When multistory shelters are evaluated, the entire shelter should be considered unuseable if any of the following can occur:

1. If the dead weight of the building cannot prevent flotation.
2. If the building foundation cannot resist scour by currents and waves without settlement.
3. If the structural frame cannot withstand anticipated wind forces acting in conjunction with breaking wave forces (acting either directly on the frame or transferred to the frame by walls).
4. If the structural frame cannot withstand battering by floating debris produced by structures and objects immediately adjacent to the shelter (see discussion of debris effects on page 25).

Upper stories of multistory shelters are considered adequate if they do not flood and if the shelter can resist the forces mentioned above (even if lower floors flood).

The first step in evaluating a shelter's resistance to water forces is to determine the total water level at the site. This includes the storm tide still water level (swl), plus the height of any waves above the swl. The maximum storm tide that can be expected to occur should be computed for each category storm. Wave heights should then be added to each, using the methodology developed by the National Academy of Sciences for FEMA (9).

Storm tides should not be confused with storm surge. The latter can be defined as the rise in water level above normal, due mainly to wind stress, bathymetric and barometric pressure effects. Storm surge computer models used in evacuation studies (SPLASH and SPLOSH) do not account for astronomical tide and wave setup contributions to the swl. These must be added to storm surge levels to arrive at storm tide values.

Storm tide values should also account for the uncertainty in the storm surge levels predicted by the computer models. The models cannot account for small scale variations in bathymetry and shoreline configuration. These variations are "smoothed" for modeling purposes. Thus, surge levels at a particular location may be higher than predicted.

There are three fundamental principles involved in the computation of the wave heights that are added to the swl. They are:

1. depth-limited breaking waves have a maximum height of 0.78 times the still water depth, and 70 percent of the wave height lies above the still water level.
2. wave energy (and heights) can be dissipated by obstructions (vegetation, buildings, sand dunes, etc.).
3. waves can be regenerated in open areas.

The first principle results in the following relationship:

$$Z_{max} = SWL + 0.55\ (SWL - Grade)$$

where: Z_{max} = maximum flood level, including wave heights (ft. above msl)

SWL = storm tide still water level (ft. above msl)

Grade = grade elevation (ft. above msl)

The second principle allows for the flood level to be reduced as the waves encounter obstructions. Thus, the actual flood level at a shelter, Z, will be less than Z_{max} if there are any obstructions between the shelter and the shoreline. However, the value of Z will never fall below the storm tide still water level.

The third principle allows for wave heights to increase in open areas. The amount of increase will depend upon the wind speed, the water depth and the size of the open area. Generally speaking, the effect will be insignificant for areas less than 0.1 mile across and minor for areas less than a few hundred yards across, regardless of depth and wind speed. For open areas with shallow flooding the effect will be minor since wave heights cannot increase beyond their depth-limited value (0.78 times the water depth). Figure 13 illustrates how the total water level might vary for a transect taken normal to the shoreline under given storm conditions.

In the Monroe County study values of SWL were taken from Table 11 of reference 21, and values of Z_{max} were computed for each shelter for each storm category. Values of Z were estimated, based upon the exposure of each shelter. Depths of flooding at each shelter were found by taking the difference between Z and the shelter floor elevation; these values were reported in the findings and recommendations for each shelter (see Section III - Case Studies). A similar procedure should be followed when other shelters are evaluated.

Given the total water level at a site, the second step in evaluating a shelter's resistance to water forces is to estimate the magnitude of any hydrostatic forces, either lateral or vertical (i.e., uplift) on the structure.

The third step in evaluating a shelter's resistance to water forces is to estimate the effects of currents on the structure. In general, there may be two major effects: scour at the base of the structure and direct hydrodynamic forces acting on the structure. Since it is very difficult to predict the magnitude and direction of currents around a structure during hurricane flooding, it is difficult to quantitatively estimate either of these. Guidance for estimating scour and hydrodynamic loads (assuming current velocities are known) can be obtained from reference 30 and 31.

During the Monroe county study the effects of currents were determined subjectively, and were assumed to increase as the depth of flooding increased and/or as the distance between the shelter and the shoreline decreased. Shelters near the shoreline or located where the water depth was greater than a few feet were not recommended for use unless their

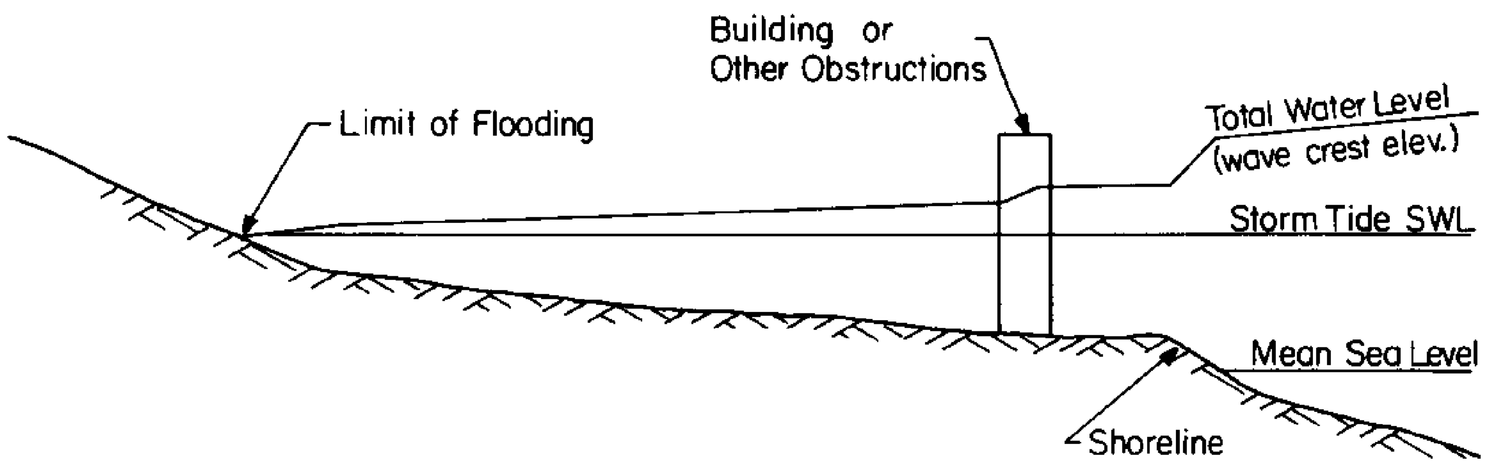


Figure 13. Variation in Total Water Level Along Transect

foundations were capable of withstanding scour without settlement. Although hydrodynamic loads were not determined explicitly, it was assumed that these would be small in comparison with the breaking wave forces that could affect a shelter.

Determination of breaking wave forces, then, is the fourth (and one of the most critical) step in determining a shelter's resistance to water forces. This is also one of the most difficult problems encountered in evaluating a shelter. Methods to calculate breaking wave forces are crude and the results are approximate at best. Despite the inaccuracies it is known that, while normal loads on a structure are on the order of tens of pounds per square foot, breaking wave pressures can reach hundreds or thousands of pounds per square foot. Although intense, these breaking wave pressures (sometimes called shock pressures or impact pressures) are of a very short duration. Hence, they are treated as an impact load in the structural analysis.

The Corps of Engineers (32) investigated the problem through storm damage surveys and a structural analysis of wave pressures on a typical dwelling. Minikin's equation was used to estimate breaking wave pressures (details of this method are contained in reference 31). The results indicated that a three-foot wave was capable of damaging a typical wood-frame structure. Other storm damage surveys show that unreinforced masonry walls also fail under such conditions.

A recent laboratory study (13) shows that the Minikin approach underestimates breaking wave pressures on intermediate slopes. Results from this study were used in the Monroe County investigation. Breaking wave pressures were estimated using the following relationship:

$$P = \text{const.}\ \gamma\ H_b$$

where: P = maximum breaking wave pressure (lb/ft^2)

const. = a constant which varies with the slope of the ground in front of the wall

γ = unit weight of water (approx. 64 lb/ft^3)

H_b = breaking wave height (ft.)

The constant reaches a maximum value of 15 for a slope of 1/10. For slopes steeper than 1/10 the value of the constant decreases rapidly; for slopes flatter than 1/10 the value of the constant decreases gradually. The reader is referred to references 13 and 31 for more details. Regardless of the method of calculating breaking wave forces though, the structural analysis of a shelter should consider the wind forces, hydrostatic forces, hydrodynamic forces and the breaking wave forces acting concurrently.

Studies of hurricane damages following a storm usually show that water damages are most severe at the shoreline and that they diminish rapidly as one moves inland. This is due chiefly to the effects of breaking waves. The studies also point out several things that should be examined carefully when the water resistance of a shelter is determined. These include:

exposure - buildings located on the shoreline or those situated with few obstructions between them and the shoreline will be exposed to the most severe conditions.

foundations - pile supported structures are susceptible to failure unless the pilings are deep enough to support the structure in the event that fill around and beneath the structure is washed away. Structures supported on spread footings cannot accommodate much scour without collapsing (Figure 14).

connections to foundation - unless the main structure is tied to the foundation it may float or slide off. The dead weight of the structure alone may not resist flotation and lateral forces.

structural frame - the structural frame of a building must be able to withstand all anticipated current, wave and debris forces acting on it concurrently with wind forces.

lower story walls - will the walls break out under wave and other forces or will they remain intact, transferring the loads to the structural frame?

spaces that confine waves - will corners and intersections of walls and floors trap waves, causing uplift forces? Were the floors designed for upward as well as gravity loads? Will a failure of a slab or other member compromise the stability of the main structure (through loss of diaphram action, etc.)?

Debris Effects

The effects of wind-borne and water-borne debris are difficult to calculate explicitly since it is almost impossible to predict the type, size, speed and point of impact of debris that will strike a shelter. During the Monroe County study, it was assumed that the probability of large debris (other buildings, towers, tanks, trees, etc.) striking a shelter was small, unless the site inspection showed the condition of structures and trees immediately adjacent to the shelter to be questionable (Figure 15). Buildings were not recommended for use as shelters unless solid shutters (not expanded wire mesh or other grillwork) were in place and operable. Reference 30 contains suggested design debris loads for new structures, but these cannot be applied readily to existing structures.

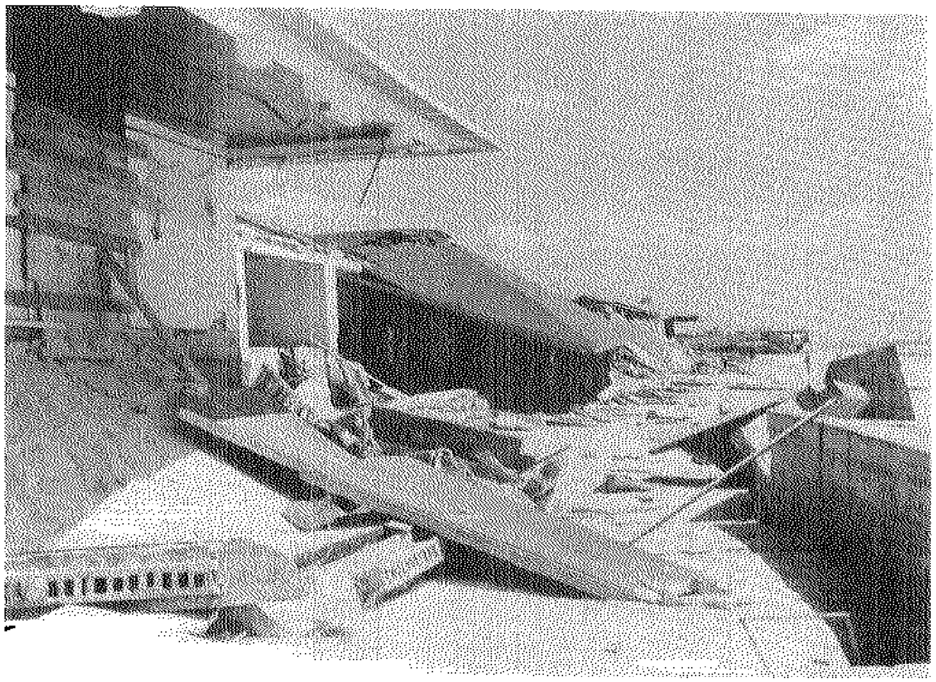


Figure 14. Structures with Spread Footings or Other Shallow Foundations Cannot Withstand Scour

Figure 15. An Example of Debris that Could Strike a Shelter

III. RESULTS OF MONROE COUNTY STUDY

Fifty designated or potential hurricane shelters were examined during the Monroe County study. Sufficient information was obtained for 31 to be evaluated. Some information has been obtained and partial inspections have been made on the remaining 19 but final recommendations have not been made at this time.

The shelter inspections revealed that only a few of the shelters had adequate shutters. Most had no shutters at all (Monroe County is in the process of installing shutters now). In many instances, buildings could not be recommended for use as shelters until repairs were made. The needed repairs were minor in some cases: replacement of shutter anchors, repair of deteriorated timber door jambs and window frames, cleaning out roof drains, etc. The repairs were major in others: roof repairs, removal and replacement of deteriorated concrete members, reinforcement of columns and beams, etc.

The results of the evaluations show that even when shutters are installed and when repairs are made, very few of the buildings will be capable of providing shelter during a major hurricane. The breakdown is as follows: four should not be used as shelters under any circumstances, three could be used during category 1 storms, 14 could be used during category 2 storms, eight could be used during category 3 storms, one could be used during category 4 storms and one could be used during category 5 storms.

The results are not surprising when one considers the fact that most of the buildings in the region have been designed to withstand 100 to 120 mph winds (category 2 and 3 winds) and that many have not been designed to withstand scour and wave forces. The results are also consistent with the post storm damage surveys mentioned previously (18, 33).

The study also revealed another very important point: the resistance of some shelters had been reduced considerably from original due to building modifications (additions; installation of utilities, plumbing or air conditioning, etc.) and/or inadequate maintenance. Thus, shelter evaluations must be made periodically to ensure that a shelter's resistance has not diminished with age or been affected by building modifications.

IV. CASE STUDIES

Two buildings were selected as case studies to illustrate the use of the methodology presented in Section II. One is a single story U. S. Navy structure in Key West (Galley Building) and the other a multi-story church school in Islamorada (Island Christian School). These two cases were selected because they illustrate many of the deficiencies and problems typical of buildings that may be used for shelters.

Each case study will be presented in the following format. The shelter summary form will be included first, followed by a commentary and finally by the findings and recommendations for that shelter.

CASE STUDY 1: GALLEY BUILDING

Shelter Summary Form

STRUCTURE	LOCATION
1000 MAN SUBSISTENCE (GALLEY) BUILDING	KEY WEST, TRUMAN ANNEX

A. GENERAL DATA:

1. DATE OF CONSTRUCTION:

   1962. An addition was made to the south half of the west side at a later date.

2. BUILDING TYPE/STRUCTURAL SYSTEM:

   Reinforced beam and column with double tee roof, 8" concrete exterior wall for original building. Addition is timber frame and flat roof.

3. NUMBER OF STORIES:

   One.

4. BUILDING HEIGHT ABOVE GRADE:

   High roof: Varies from 20.0' to 21.0' at top of parapet
   Low roof: Varies from 12.92' to 13.92' at building edge

5. GRADE ELEVATION:

   Varies from about 7.5' to 8.5' at building edges. Slopes to about 4.5' to 7.5'

6. CODE USED:

Unknown.

7. DESIGN DATA:

Roof:	30 psf
Floor:	100 psf
Loading and Storage:	250 psf
Wind: 0'-10'	30 psf (127 mph @ 10')
10'-20'	36 psf (125 mph @ 20')

8. DESIGNER AND CONTRACTOR

Designer: Watson, Deutschman and Kruse (Miami, Florida)
Contractor: Unknown.

9. EXPOSURE:

North:

Limerock surfaced approach to loading platform, one and two story masonry structures beyond.

South:

Street, parking lot and two story masonry barracks beyond. Loose coral rock is piled at edge of parking lot.

East:

Open considerable distance to trees beyond.

West:

Elevated tank, junked automobiles and appliances. Several large Australian pines very close to building at west side toward north end.

10. FLOOD HAZARD DATA:

a. FEMA:

100-yr. base flood elevation (including wave height) is 8.0 ft. msl. The building lies in an A 10 zone (all data from City of Key West, preliminary FIRM, map 1490; October 16, 1981).

b. CORPS OF ENGINEERS:

Worst probable storm tide still water level by storm intensity (from Table 11, page 37 of Technical Data Report, Lower Southeast Florida Hurricane Evacuation Study, June 1983):

Saffir - Simpson Category	Storm Tide SWL above MSL
1	5 ft.
2	7 ft.
3	10 ft.
4	13 ft.
5	> 15 ft.

11. TYPE OF INVESTIGATION:

Site visit and review of as-built plans dated April 27, 1964 for original structure. No plans were available for the addition.

B. FOOTINGS:

1. COLUMN FOOTINGS:

a. TYPE:

10" x 10" and 12" x 12" square reinforced concrete piling driven to specified bearing capacity. Pile caps are placed at column locations.

b. ELEVATION:

Top of pile cap:	+7.33'
Top of pile:	+5.58'
Bottom of pile cap:	+5.33'
Bottom of pile:	varies (driven into rock)

c. CONDITION:

Unable to determine.

2. WALL FOOTINGS

a. TYPE:

Reinforced concrete grade beam supported by pile caps and piling. Reinforcing from piling extends into grade beam.

b. ELEVATION:

Top:	+9.75'
Bottom:	+7.08'

c. CONDITION:

Unable to determine.

C. COLUMNS:

1. TYPE:

Reinforced concrete tied.

2. CONNECTION TO STRUCTURAL SYSTEM:

Dowels extend into columns from footings and from columns into roof beams.

3. CONDITION:

No deterioration observed.

D. BEARING PARTITIONS:

None.

E. FLOORS:

1. FIRST FLOOR:

a. TYPE:

4" concrete slab with 6" x 6" - 10/10 wire mesh, except 6" reinforced slab in meat freezer. All slabs are on fill.

b. CONNECTION TO STRUCTURE:

None. Separated from wall by expansion joints.

c. ELEVATION:

+10.5' (Machinery room floor on east side of Kitchen area is at elevation + 8.0')

d. CONDITION:

Good.

F. ROOF:

1. TYPE:

Flat, prestressed double tee, built-up tar and gravel. Slight slope: 12" from high edges to drains. Parapet around high roof 8" to 1'-8" high. Timber sheathing with built-up tar and gravel on addition.

2. STRUCTURAL SYSTEM:

Prestressed double tees are supported on perimeter beams. Timber sheathing is supported by timber joists.

3. ELEVATION:

Low roof: 21.75' at outer edge of building
High roof: 27.75' at parapet

4. CONNECTION TO STRUCTURAL SYSTEM:

Beams are set in place and the space between stems is filled with a secondary pour of concrete. Steel plates embedded in the top of the perimeter beams and bottom of the double tee stems are welded. Timber members on addition are tied together with hurricane anchors.

5. CONDITION:

Roof over original building is in good condition. There is rot in the eave of the addition roof on the north side.

6. DRAINAGE:

a. SCUPPERS AND DRAINS:

High roof

One - 6" x 5" metal scupper 3" above roof
Two - 12" x 5" metal scuppers 3" above roof
One - 5" roof drain
One - 4" roof drain
Two - 3" roof drains

Low roof

No scuppers are required since there is no parapet

Two - 3" roof drains
One - 4' roof drain
One - 5" roof drain

b. CONDITION:

Scuppers are in good condition. Drains were partially plugged with pine needles and other debris.

c. POTENTIAL FOR STANDING WATER:

Investigation of the roof revealed that pine needles and other debris had partially plugged the drains on both high and low roof. The high roof had water standing approximately 4" deep at the 5" drain. Without a proper maintenance program there is a potential for overloading this roof. The same condition could occur on parts of the low roof.

G. EXTERIOR WALLS:

1. TYPE:

8" concrete block on walls around low roof area (north side and north half of west side). 8" concrete block with 4" glazed structural units inside at exterior walls under high roof, up to level of low roof. 12" concrete walls with 2" grooved panel on exterior above level of low roof. Plywood or composition panels on addition.

2. CONNECTION TO STRUCTURAL SYSTEM:

No horizontal reinforcement or ties to columns indicated on plans.

3. OPENINGS:

a. WINDOWS:

North:

None.

South:

Six - 6'-0" x 7'-8"

East:

Four - 6'-0" x 7'-8"
One - 3'-0" x 7'-8"

West:

Ten - Approximately 4'-0" x 8'-0" in addition, covered with plywood.

Three - Approximately 6'-0" x 7'-8" in wall separating dining area from addition.

b. DOORS:

North:

One pair - 2'-8" x 6'-8" with approximately 5" x 8" glass port

One pair - 3'-0" x 7'-0" glass with 6'-0" x 3'-0" fixed glass above

Three - 5'-6" x 8'-0" heavy rolling doors into storage areas

South:

Two pair - 3'-0" x 7'-0" glass with 6'-0" x 3'-0" fixed glass above

East:

One pair - 2'-8" x 7'-0" glass with 5'-4" x 3'-0" fixed glass above

One pair - 3'-0" x 7'-0", grilled door
One - 3'-0" x 7'-0" solid core
} to transformer and equipment rooms.

West:

None.

c. LOUVERS*

North:

Three - 3'-0" x 3'-0" in low wall

One - 16'-9" x 3'-0"
One - 10'-1" x 3'-0"
} in high wall above low roof

South:

One - 3'-0" x 3'-0" in low wall

East:

One - 10'-1" x 3'-0" in high wall above low roof.

West:

Five - 3'-0" x 3'-0" in low wall

One - 10'-1' x 3'-0" in high wall above low roof

One - 14'-11 1/2" x 6'-11" fixed

One - 14'-11 1/2" x 3'-9 1/4" fixed behind 14'-11 1/2" x 6'-11"

* All louvers are operable except those designated "fixed".

d. OTHER:

Twenty-two - 5" x 1'-0" vents between alternate double tee stems into ceiling space at loading platform on north side.

4. CONDITION:

No deterioration observed on masonry walls. Wind resistance of wall on addition is questionable.

5. WINDOW PROTECTION

Metal shutters are provided for windows and doors on south, west and east sides of dining room and serving area. The shutters are on rollers at the top and are fastened into expansion anchors set in the window and door sills with 1/4" bolts. The expansion shields are filled with sand and the screws cannot be fastened. There is a wide space between the bottom of the shutter bolt lugs and the sills that will result in the bolts being subjected to bending and shear. This is not a desirable loading condition and may result in failure in a low category storm.

The shutter fasteners on one door were badly deteriorated and will not provide protection.

H. PROJECTIONS:

1. STRUCTURAL:

a. TYPE:

1. Reinforced concrete canopy over south and east walkways.
2. Prestressed concrete canopy over north loading platform.

b. CONNECTIONS:

1. Walkway canopies are placed integrally with the perimeter beam and are reinforced for gravity loads.

2. The prestressed concrete canopy is a cantilevered portion of the main roof system and the space between the webs is encased in a secondary concrete pour.

c. CONDITION

No deterioration was apparent on either canopy.

d. HAZARD POTENTIAL:

Although the canopies are reinforced for gravity load only, their weights are such that they offset uplift for some category 4 storms (140 mph).

2. MECHANICAL:

a. TYPE:

Five ventilating fan covers.

b. CONNECTIONS:

Plans indicate a concrete curb around the fan openings with a timber member on top. The method of fastening the timber to the curb or the fan cover to the timber is not shown.

c. CONDITION:

Covers are rusting badly.

d. HAZARD POTENTIAL:

Covers are light enough that no damage will occur to adjacent structure if they fail.

I. OTHER

1. SCOUR POTENTIAL:

Elevations of the retaining wall footings around south, east and west sides are unknown. There is a possibility of scour if the bottoms of these footings are not deep. Scour would not affect the pile supported footings. However, a retaining wall failure could contribute to scour and settlement under the concrete sidewalk outside of the dining area and under the dining and serving area slab. If door shutters are attached to the sidewalk, protection will be lost if the walk settles.

There is a possibility of scour under the grade beams and beneath the slab in the kitchen area (northwest corner of building) during high category storms.

2. RESISTANCE TO SLIDING/OVERTURNING:

Foundations are deep enough and sufficiently tied to the structure that there is no chance of sliding or overturning.

3. INTERIOR SHELTER POTENTIAL:

The kitchen is the only area with interior shelter potential. It is not recommended because of lack of space due to equipment.

J. COMMENTS:

There was considerable debris on both roof levels. The roof drains were partially blocked by pine needles and other materials and there was 3" to 4" of water in the vicinity of the drains as a result. The debris on the low roof could damage louvers in the walls supporting the high roof.

The shutters are poorly maintained and are not, in the present condition, capable of adequate fastening in an emergency. The hangers at the top are severely deteriorated and anchors for stove bolts cannot be used.

Commentary on Galley Building

The Galley Building (Figure 16) was designed for the Department of the Navy by the architectural-engineering firm of Watson, Deutschman and Kruse of Miami, Florida. The code under which the building was designed is unknown, but some design data were included on the plans. The building was constructed in 1962 and an addition was made on the southwest side at a later date.

Figure 16. Galley Building

The investigators were provided with a partial set of "as-built" plans dated April 27, 1964. These consisted of one sheet of architectural floor plan and finish schedule, one sheet of piling and grade beam framing plans, two sheets of wall sections, one sheet of exterior elevations, one sheet of roof plan and details and one sheet of structural sections and details (seven sheets out of 31 total). No specifications were available. Those plans that were available were well prepared and gave excellent details for joint connections and methods of construction required to provide structural continuity.

Using the methodology developed, the drawings were examined and appropriate data were recorded. An on-site visit was made following the plan review to verify the plans and the condition of the structure. The plan review was not intended to be a critical review for the purpose of checking the design. As plans were incomplete and specifications were not available the on-site inspection provided supplemental information used in the evaluation.

The design data on the plans show wind loads of 30 psf from 0-10 ft above grade and 36 psf from 10-20 ft above grade, which represents a wind velocity of about 126 mph (a category 3 storm). This assumes that the loads were velocity pressures and were applied as most codes specify, that is, that shape factors were applied to these pressures depending on the surface exposed to the wind.

The Galley Building is situated as shown in Figure 17. On the west side there is a storage area for old vehicles, appliances, building materials, etc. These are separated from the building by a chain link fence. The materials in the storage area have the potential to damage the structure as they could be carried with considerable force by both wind and water. Although there is an elevated water tank in the storage area, it would not be expected to strike the building if it fell. Australian pines located on the west side are close enough to cause damage to the structure if they break or are overturned as a result of wind and/or water forces.

The parking and loading approach area on the north side is finished with a coral rock surface and has many loose pieces that could become wind borne missiles. There are no windows on this side of the building, however, and no damage to the Galley is anticipated here. The south side is exposed to the same type of parking area located approximately 400 ft. away. In the event of a shutter failure, missiles from this area are a potential hazard.

The structure is far enough inland and surrounded by enough structures that wave forces should be minimal during category 1, 2 and 3 storms. In a category 3 storm it is expected that there would be up to one foot of water in the main structure and up to three feet in the machinery room.

The general structural system of the building is: reinforced concrete column and tie beams, floor slab on grade, prestressed double tee roof members and 8" exterior block walls on the lower section and 12" concrete walls on the upper section.

The foundation of the building consists of a combination of reinforced concrete pile caps and grade beams supported on reinforced concrete piles driven to a specified bearing capacity. The elevation of the bottom of the piles is unknown, but they are driven well below mean sea level and will not be affected by scour if it should occur.

The bottom elevation of the grade beam is indicated at +7.08' msl, which is slightly below grade elevation at the building edges. Retaining

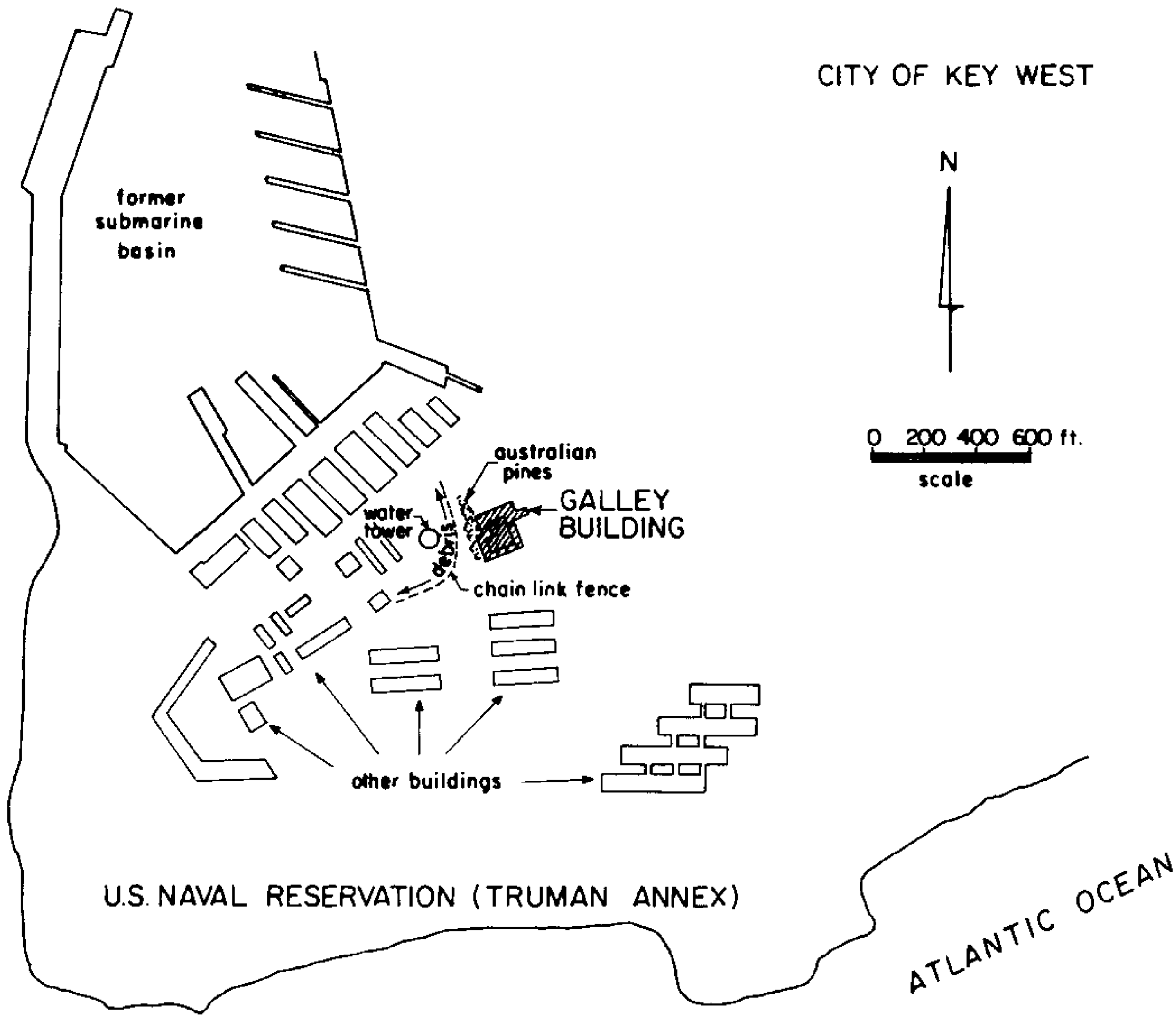


Figure 17. Location Map

walls maintain the grade elevation on the south side and on portions of the east and west sides; the grade elevation outside of the walls varies from approximately +4.5 to +7.5' msl. The plans available to the investigators do not show the elevation of the foundation of the retaining walls. Therefore, the ability of these walls to provide protection from scour resulting from currents and wave action is difficult to estimate. If these walls should fail, the soil behind them may be eroded to the extent that the grade beams are exposed and scour may occur beneath the floor slab. The plans do not indicate any attachment of the slab to the structural system and show a separation from the block walls by a cleavage joint. In the event of scour beneath the slab, it could fail. This is not expected to be a significant problem during category 1, 2 and 3 storms.

Scour behind the retaining walls could also result in settlement of the walkway around the south and east sides of the building. Failure of the walkway would render the door shutters useless since the bottoms of these shutters are attached to anchors set in the walkway.

The structural roof is a prestressed concrete double tee system and is attached to the supporting beams by welding plates cast into the tees and the beams. The weight of the members alone is enough to resist the negative wind pressures in excess of the design loads. The roof is sealed with a built-up tar and gravel cover. There are no openings in the upper wall section that will be affected by impact from wind driven gravel from the lower roof.

The drainage will be adequate if there is assurance that the drains and scuppers will be clear of debris. The on-site inspection revealed that a number of them were clogged with pine needles from nearby trees and with other debris that had collected on the roof. About four inches of water had collected around one drain on the high roof (Figure 18). A maintenance program should be required to prevent blockage of the drains and scuppers that could result in roof overload. The same debris problem was noted on the lower roof, although there is no parapet to retain the water in the event of drain blockage.

The exterior of the building is concrete block with 36 percent glass openings on the south side and 31 percent on the east side of the serving and dining area. There are also glass openings on the west side which are shielded by the addition. The addition has glass jalousie windows that are now covered with thin plywood, toenailed in place. The plywood covers and jalousies will fail during a hurricane, exposing the openings on the west side of the serving and dining area. These must be shuttered.

Sliding metal storm shutters are provided for all windows in the serving and dining area. Some of the rollers attaching them at the top were broken and the anchors at the bottom were not usable. In their present condition the shutters will not resist anticipated wind loads. All fasteners should be checked and repaired as necessary. An additional concern about the method of anchoring the shutters in the closed position is the distance between the bottom of the shutters and the window and door sills (Figure 19). This will subject the anchor screws to a combination of bending and shear and may result in failure if subjected to alternating loads.

Figure 18. Clogged Roof Drain and Standing Water

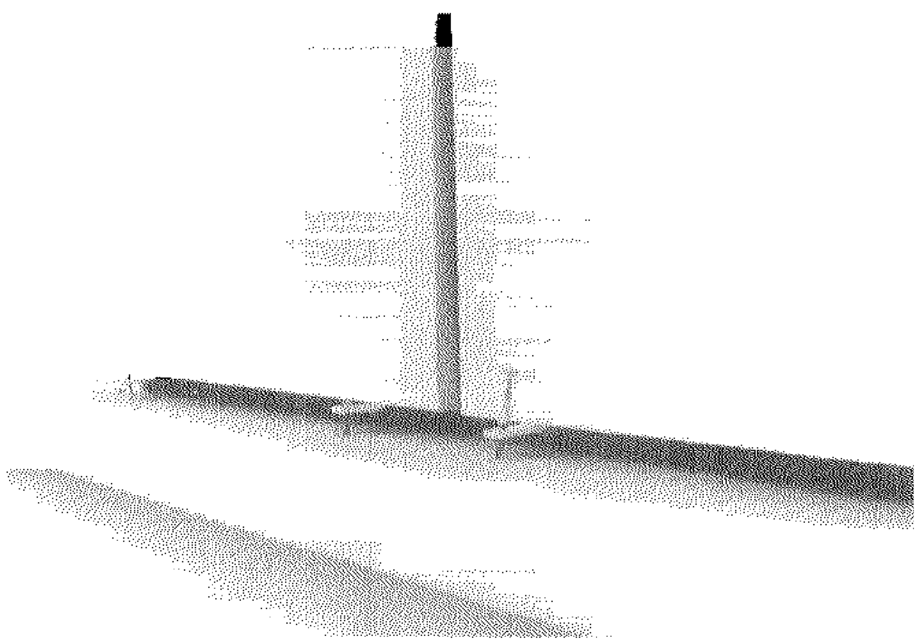


Figure 19. Bottom Shutter Fastener

Other openings are 22 - 8 inch x 16 inch screened vents placed between alternate double tie stems just below roof level on the north wall. If these are not closed before a storm, damage may occur in the ceiling of the structure and forces may be generated against the exterior walls that may not have been considered in the original design.

The plans do not indicate horizontal reinforcement between the block courses on the exterior walls. The only reference to horizontal reinforcement is in the wall between the kitchen and dining room to tie glazed tile to the concrete block wall and this has no effect on the exterior wall resistance to lateral forces.

Reinforced concrete tied columns vary in size according to loads to be carried. They are supported by grade beams and the plans specify that dowels extend sufficiently into the members to fully develop their bending and bearing capacity. At the top, the column reinforcement is developed into the tie beams to develop design stresses.

Structural projections in this building were the reinforced concrete canopies over the walkway on the south and east sides and the prestressed concrete double tees over the loading platform on the north side. The plans show a similar reinforced concrete canopy on the southern 1/3 of the west side of the building. This has become part of the addition roof. The canopies are reinforced for gravity loads only, but their weights are such that they will offset the maximum uplift produced by the design wind loads.

The mechanical projections on the roof were exhaust fans and machinery covers. The plans specify a concrete curb around the openings with a timber member fastened to the top lip. However, the plans do not show how the timber is fastened to the concrete or the covers to the timber. The investigators were unable to determine the manner of fastening. The covers were rusting badly and damaged fan blades were left on the roof when they were replaced. Without proper maintenance the covers could fail and provide access for rain during a storm. Also, the old blades can collect debris and retard the flow of water to or cover the drains creating a drainage problem. Badly deteriorated covers should be replaced and all debris should be removed from the roof.

The dead weight of the structure and the type of foundation will prevent sliding or overturning due to design wind forces and water forces during category 1, 2 and 3 storms.

Findings and Recommendations - Galley Building

This building should not be used as a hurricane shelter unless new shutter fasteners are provided for existing shutters over windows and doors (including those separating the original structure from the addition on the west side). Existing shutter fasteners are inadequate. The addition on the west side of the building should not be used for shelter space under any circumstances.

If new fasteners are installed, the original portion of this structure should provide adequate shelter against winds during category 1, 2 and 3 storms.

This structure should provide adequate shelter against flooding during category 1 and 2 storms. A category 3 storm would be expected to flood the shelter to a depth of 1 foot. Waves and currents during a category 4 storm (or higher) might scour the sloping fill around the structure and underneath the grade beams, causing a settlement of the floor slab. The retaining wall in front of the structure (details of construction unknown) might fail under similar conditions, leading to scour and floor slab settlement problems as well. The walkway around the south and the east sides of the dining area could settle if the retaining wall fails; shutters covering doors should not be fastened to the walkway as the existing ones are.

The Australian pines and debris (automobiles, appliances, etc.) on the west side of the building could cause damage to the structure upon impact. The trees should be cut down and the debris should be removed from the area.

Standing water on the roof might lead to roof overload unless adequate drainage is ensured. Roof drains and scuppers should be cleaned of pine straw and other debris periodically.

CASE STUDY 2: ISLAND CHRISTIAN SCHOOL

Shelter Summary Form

STRUCTURE	LOCATION
ISLAND CHRISTIAN SCHOOL	ISLAMORADA

A. GENERAL DATA:

1. DATE OF CONSTRUCTION:

   1976 with enclosure of lower story in 1980.

2. BUILDING TYPE/STRUCTURAL SYSTEM:

   Reinforced concrete beam and column, slab floor and roof, concrete block walls. Walls added in 1980 are reported to be breakaway walls.

3. NUMBER OF STORIES:

   Three.

4. BUILDING HEIGHT ABOVE GRADE:

28.25'

5. GRADE ELEVATION:

Approximately 7' (measured with respect to USC & GS benchmark).

6. CODE USED:

Unknown.

7. DESIGN DATA:

None given.

8. DESIGNER AND CONTRACTOR:

Designer: Charles H. Markel, Architect (address unknown)
Contractor: Tom Harden, Key Largo

9. EXPOSURE:

Northwest:

Open - approximately 75 yards to wooded area.

Southeast:

Open to Highway US1 (300' to centerline), then a motel and restaurant complex with Atlantic Ocean beyond. Relatively open - especially from the east and north.

Northeast:

Wooded area approximately 100 yards.

Southwest:

Wooded residential.

10. FLOOD HAZARD DATA:

a. FEMA:

100-yr. base flood elevation (including wave height) is 14 ft. msl. The building lies in a V 17 zone (all data from Monroe County FIRM, map 1117; December 1, 1983 ed.).

b. CORPS OF ENGINEERS:

Worst probable storm tide still water level by storm intensity (from Table 11, page 37 of Technical Data Report, Lower Southeast Florida Hurricane Evacuation Study, June 1983):

Saffir - Simpson Category	Storm Tide SWL above MSL
1	5 ft.
2	7 ft.
3	10 ft.
4	13 ft.
5	> 15 ft.

11. TYPE OF INVESTIGATION:

Site visit, review of incomplete plans and discussion with contractor and building occupants.

B. FOOTINGS:

1. COLUMN FOOTINGS:

a. TYPE:

4'-0" x 4'-0" x 1'-0" reinforced concrete under all interior columns. Exterior columns are supported on grade or wall footings.

b. ELEVATIONS:

All footings specified to bear on rock.

c. CONDITION:

Unable to determine.

2. WALL FOOTINGS:

a. TYPE:

2'-0" wide x 1'-0" thick reinforced concrete continuous around perimeter and under interior bearing wall.

b. ELEVATIONS:

Variable. Bear on undisturbed sand.

c. CONDITIONS:

Visible portion in good condition.

C. COLUMNS:

1. TYPE:

Exterior columns are 8" x 12" reinforced tie columns. First story interior columns are 16" x 16" block, reinforced and filled with concrete. Second and third story interior columns are 3 1/2 inch square steel columns filled with concrete.

2. CONNECTION TO STRUCTURAL SYSTEM:

Exterior block walls and reinforcement were placed before concrete for exterior columns was placed. Contractor stated that steel was extended and developed into the tie beams. Contractor stated that steel from first story interior block columns was developed into second floor slab. End plates on second and third story steel columns are bolted to floor slabs and beams.

3. CONDITION:

Good with exception of column in northwest corner at first floor level. Center portion has been drilled out to pass electrical conduct through to the electrical meter on outside of building.

D. BEARING PARTITIONS:

None. Occupants of the building stated that first floor walls specified as reinforced and concrete filled core walls which were indicated on the plans as bearing walls were changed to unreinforced walls.

E. FLOORS:

1. FIRST FLOOR:

a. TYPE:

4" thick reinforced concrete slab on compacted fill.

b. CONNECTION TO STRUCTURE:

None indicated on plans.

c. ELEVATION:

Approximately 7.5'

d. CONDITION:

No apparent deficiency.

2. SECOND FLOOR:

a. TYPE:

Six inch reinforced concrete slab.

b. CONNECTION TO STRUCTURE:

Negative steel is developed into exterior perimeter beams (discussion with contractor).

c. ELEVATION:

Approximately 15.0'

d. CONDITION:

Good.

3. THIRD FLOOR:

a. TYPE:

Six inch reinforced concrete slab.

b. CONNECTION TO STRUCTURE:

Negative steel is developed into exterior perimeter beams (discussion with contractor).

c. ELEVATION:

Approximately 24.2'

d. CONDITION:

Good.

F. ROOF:

1. TYPE:

Flat - Concrete with sealant.

2. STRUCTURAL SYSTEM:

Six inch reinforced concrete slab.

3. ELEVATION:

Approximately 33.4'

4. CONNECTIONS:

Negative reinforcing developed into perimeter beam.

5. CONDITION:

Good.

6. DRAINAGE:

a. SCUPPERS AND DRAINS:

1. Number: Three 4" round scuppers. No drains.
2. Condition: Clean, good condition.

b. POTENTIAL FOR STANDING WATER:

1. Scuppers are sufficient when clear, but might clog with small debris.

G. EXTERIOR WALLS:

1. TYPE:

Eight inch concrete block

2. CONNECTION TO STRUCTURAL SYSTEM:

No ties specified on plans. Concrete for columns was placed after block walls were constructed to provide bond between block and columns.

Two walls in first story are bearing walls (contractor's statement). Originally, when built in 1976 only the bearing walls were constructed. "Breakaway" walls were added in 1980 to enclose library. Inspection revealed that the two bearing walls were not reinforced and filled with concrete as the plans called for.

Steel and concrete may have been left out to ensure compliance with flood insurance requirements. It is not known if the structure was redesigned to account for the loss of support if the walls are destroyed.

3. OPENINGS:

a. WINDOWS:

Southeast:

None.

Southwest:

Eleven - 2'-0" x 6'-0"
One - 2'-0" x 3'-0"
second and third floors

Northeast:

Seven - 2'-0" x 6'-0"
Four - 2'-0" x 4'-8"
One - 2'-0" x 3'-0"
One - 2'-0" x 1'-6"
second and third floors

Northwest:

None

b. DOORS:

Southeast:

Two - Double 3'-0" x 6'-8" solid, second and third floors

Southwest:

One - Approximately 6'-0" x 8'-0", first floor
One - 3'-0" x 6'-8" solid, first floor

Northeast:

Three - 3'-0" x 6'-8", two on first floor, one on second floor

Northwest:

One Double 3'-0" x 6'-8", second floor

4. CONDITION:

Exterior walls in second and third stories are in good condition. See G.2 for description of first story walls.

5. WINDOW PROTECTION:

3/4" plywood panels fastened by bolting with 1/4" bolts into inserts in drilled holes in concrete block. Inserts are not tightly in place and can be easily worked loose. These fasteners are not considered adequate. Plywood panels do not cover entire window area, but should afford sufficient protection.

H. PROJECTIONS:

1. STRUCTURAL:

a. TYPE:

1. Ramp and landing to second floor at front entrance on southeast side.

2. Stair and landing to third floor at front entrance on southeast side.

3. Stairs and landings to second floor on northeast and northwest sides.

4. Overhead canopy at front stair entrance projecting 3'-0" from wall and 10'-0" long with an architectural treatment 4'-0" high

5. Similar canopies over second story entrances on northeast and northwest sides.

b. CONNECTIONS:

1. The landing is tied to the tie beam at the second floor with reinforcing steel. The ramp is supported on piers and spread footings. Details of reinforcement are not shown on plans.

2. The landing is tied to the tie beam at the third floor with reinforcing steel. The stairs are supported independently by piers on spread footings. Details of landing reinforcement to tie beam are not shown on plans.

3. The landings are tied to the second floor tie beam with reinforcing steel. No details are shown for this or for the stairs.

4. & 5. The canopies are tied into tie beams. Details of the reinforcing steel and the details of the architectural structure are unknown. The plans do not show the canopies over the northwest and northeast entrances.

c. CONDITION:

1, 2 and 3. Good.

4, 5. No defects observed.

d. HAZARD POTENTIAL:

1, 2, and 3	Appear to be substantially built. No spalling or defects were observed. Minimal hazard potential.
4, and 5	The canopies appear to be substantially built, since method of construction of the architectural treatment or the canopies is unknown, the hazard potential is difficult to assess. Little damage should occur to the building if these elements fail.

2. MECHANICAL:

a. TYPE:

Five air conditioning units. Roof hatch. Two solar heating panels.

b. CONNECTIONS:

Air conditioning units are fastened to roof with one 1/4" bolt in each leg. Solar panels are fastened to a pipe support with 1/8" screws. Roof hatch is fastened with hinges on one side and lock and hasp on the other.

c. CONDITION:

Good.

d. HAZARD POTENTIAL:

Although these units are rather insecurely fastened they will present no critical hazard to the structure if they are torn loose. Rainfall will enter building if roof hatch is torn loose.

I. OTHER:

1. SCOUR POTENTIAL:

Excavation around the footings during an on site visit revealed sand and shell around the wall footings. Local scouring may possibly occur around corners of building and supports of exterior stairs, but not to sufficient depth to cause failure of the structure.

2. RESISTANCE TO SLIDING/OVERTURNING:

Factor of safety against overturning is greater than 5. Sliding is prevented by penetration of footings to rock.

3. INTERIOR SHELTER POTENTIAL:

None.

J. COMMENTS:

The on site visit revealed several things that are of concern to investigators:

1. The plans indicate two bearing walls in the rear (northwest end) between the foundation and second floor. Inspection revealed that there was no reinforcing in the walls and the block cells were not filled with concrete, as called for in the plans. The reinforcement and concrete may have been left out to ensure compliance with flood insurance requirements.

   It is not known if the structural frame was redesigned to account for the loss of support if the walls are destroyed. In any event, reinforcing steel in the tie beam above one of the walls was improperly placed and is exposed--the strength of the beam is questionable. This brings up a "no win" situation if the walls are subject to wave forces. If the walls are not destroyed, this may transmit the forces to the frame and the building may collapse (it was designed for wind loads only). If the walls are destroyed, their support will be lost and the building may collapse.

2. Examination of the tie beam above one of the bearing walls shows exposed reinforcing steel that may corrode, lose its strength and cause deterioration of structural members.

3. During installation of the electrical conduit to connect to the meter on the building exterior, a large portion of the mid-width of the column in the northwest corner was cut out. The manner in which the exposed steel appeared left a question as to continuity of either perimeter beam or column steel.

4. Anchors provided for window shutters are not securely fastened and may pull out under wind forces.

5. The roof hatch, although heavy, might fail during hurricane winds. Rainfall will enter the interior stairwell between the second and third stories.

Commentary on Island Christian School

The Island Christian School (Figure 20) was designed by Charles Markel, architect (address unknown), and built by Tom Harden, a general contractor based in Key Largo. The code under which the building was designed is unknown, but it was probably the Southern Standard Building Code, which was in effect when the building was constructed in 1976. The first (ground level) floor was enclosed in 1980 with, what the contractor called, breakaway walls.

Two sheets of plans and a site plan were provided to the investigators. The site visit revealed that the structure was not built according to the plans and that no "as-built" plans were available. Discussions with the contractor, maintenance superintendent and the pastor of the church contributed information on changes and reasons for them.

The structure is located at the north end of Upper Matecumbe Key, and is within 900 ft. of both the Atlantic Ocean and Whale Harbor Channel. Figure 21 shows the view from the roof at the front of the building (southeast side), looking toward the Atlantic. Figure 22 shows the location of the structure with respect to its surroundings.

A heavily wooded area lies about 50 ft. from the building along its southwest side; residences lie beyond. The northwest and northeast sides face open areas 150 ft. to 350 ft. wide, with dense vegetation and mangroves beyond. The southeast side is open to US 1, with only a few palm trees and small one story buildings between the highway and shoreline. The elevation of the highway is approximately the same as the grade elevation at the shelter - about 7 ft.

During category 3 conditions the situation is likely to be that shown in Figure 23. The storm tide swl will stand at 10 ft. and the value of Z_{max} will be 11.7 ft. msl. Since the ground between the building and the Atlantic shoreline is relatively flat and since there are few obstructions, it is reasonable to assume that the actual total water level, Z, will be very near the maximum depth limited value Z_{max}. Waves striking the front of the building will thus be 2.4 ft. in height (recall that the difference between Z and SWL represents 70% of the wave height). Breaking wave impact pressures were estimated at 1,500 lb/ft^2 acting at an elevation of 11.0 ft. msl.

The building is a concrete block structure with exterior tie columns and beams poured after the blocks were laid. Interior columns on the first floor are concrete column block filled with concrete and interior columns on the upper floors are square steel tubes filled with concrete. The first floor is a slab on grade. The second and third floors and roof are shown on the plans as two way slabs.

When the structure was built in 1976 the first story was open, with a concrete block wall at the rear (northwest end) and another concrete block wall at the second row of columns from the rear. The plans showed these first floor walls to be bearing walls with steel reinforcmeent and concrete in each cell. An on-site inspection revealed that the cells are not reinforced or filled. The investigators were informed that the method of construction was changed so that the structure would comply with the local flood insurance requirements.

The foundation for the interior columns consists of reinforced concrete footings bearing on rock. The foundation for the exterior columns and original first floor walls consists of grade beams specified to bear on rock or undisturbed sand. Because of the variable elevation of rock in the area, several of the grade beams projected above grade elevation (they appeared to be in good condition).

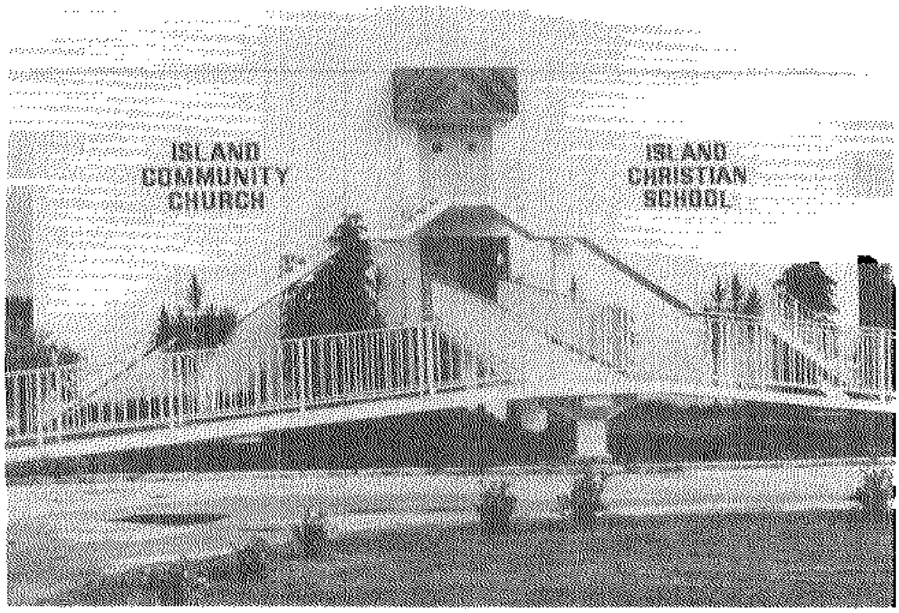


Figure 20. Front View of Island Christian School

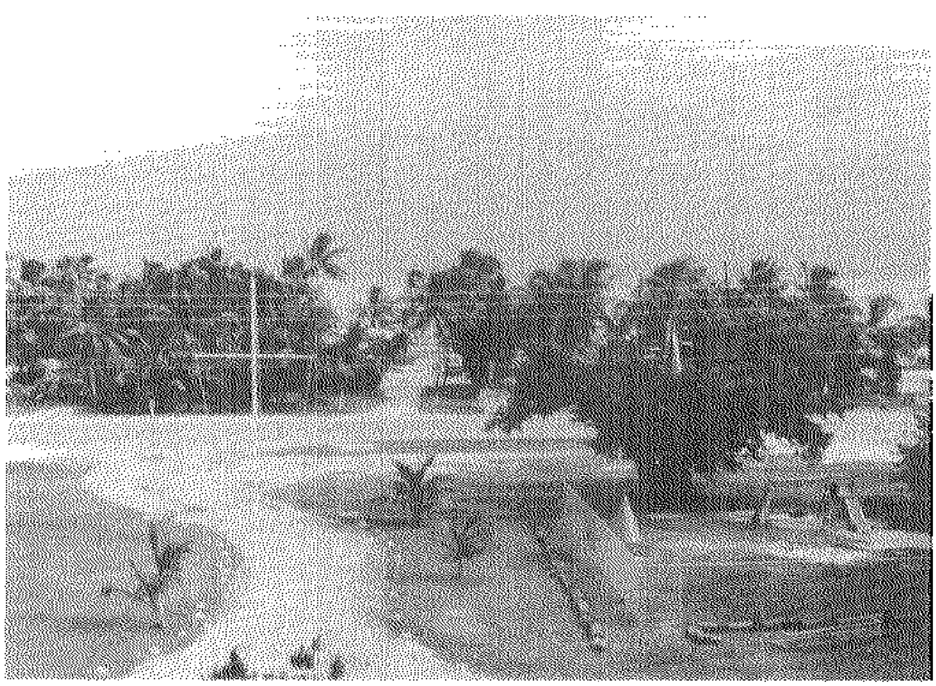


Figure 21. View Toward Atlantic from Roof

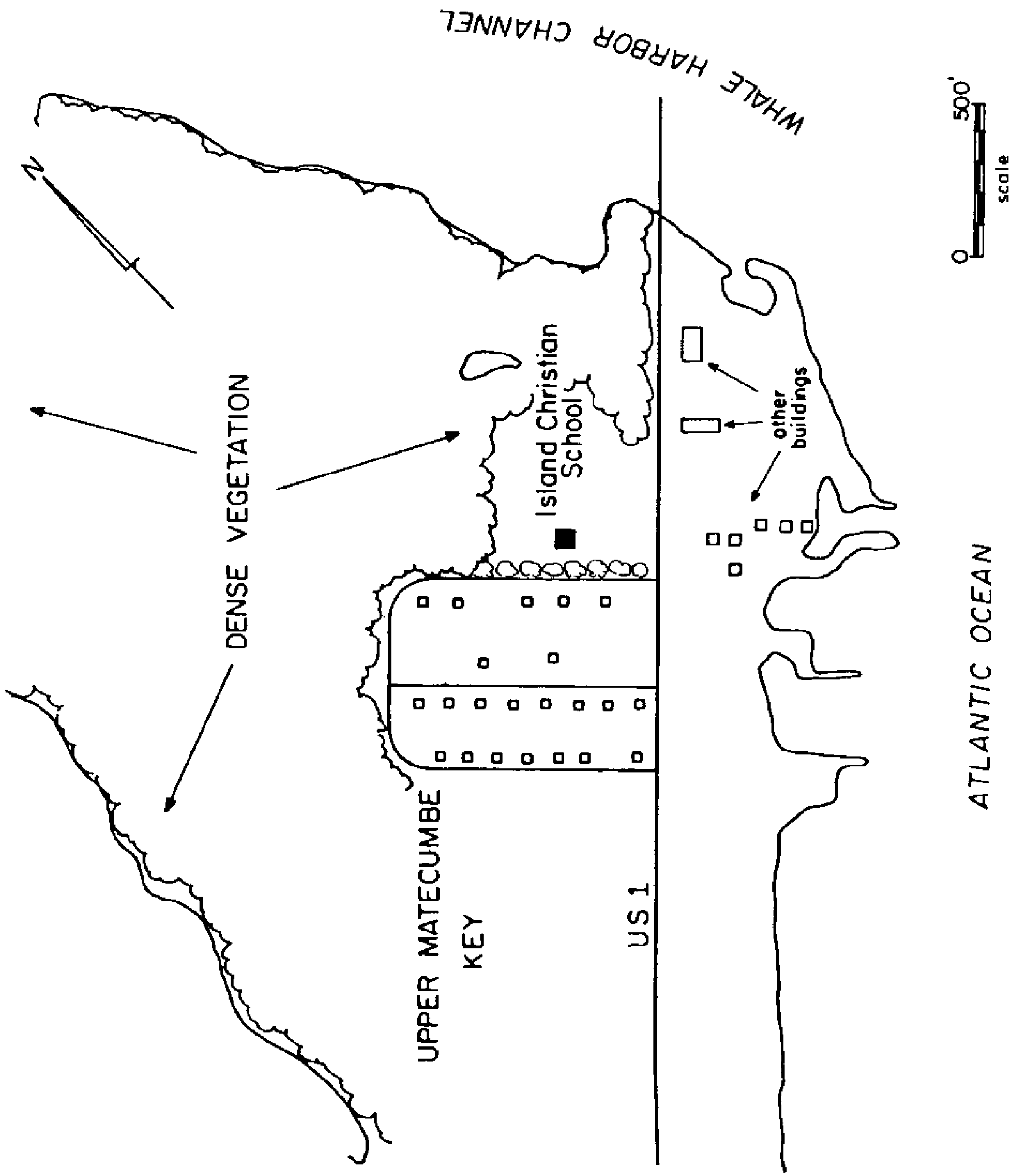


Figure 22. Location Map

The first floor slab on grade is 4 inches thick and is reinforced with wire mesh. No connection to the structure is shown on the plans. The elevation is approximatley 7.5'. The second and third floor and the roof slabs are six inch reinforced, two way concrete slabs. The contractor stated that the slabs were tied into the perimeter beams to develop negative moment.

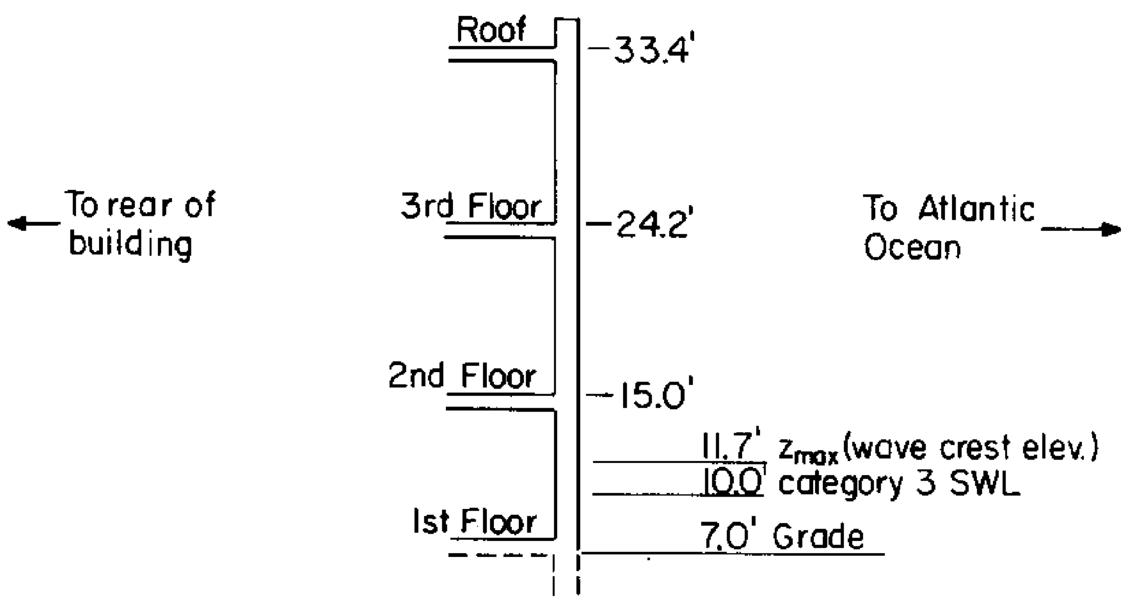
Exterior columns are reinforced tie columns that were placed after the walls were in place. Interior columns on the first floor are 16 inch column block, reinforced and filled with concrete. The contractor stated that column steel was lapped sufficiently to develop full continuity between connected members. Interior columns on the second and third floors are 3 1/2 inch square, 3/16 inch thick concrete filled steel members with end plates bolted to the floors at the bottom and support beams and slabs at the top. The interior steel columns will offer little resistance to lateral forces on the structure. Hence, these forces must be transmitted to the outer walls through diaphragm action of the floors.

Exterior walls at the second and third levels are 8 inch concrete block. No horizontal reinforcement in the walls or ties between the walls and columns were specified on the plans. However, the contractor stated that the concrete columns were poured after the block walls were laid, to ensure bond between the walls and columns. The first floor walls added in 1980 are also 8 inch concrete block, but the contractor stated they were built to break away during a storm (details of construction unknown).

Bolts in expansion anchors were installed to fasten 3/4 inch thick plywood shutters to protect second and third floor windows. The shutters do not cover the full opening, but leave small gaps at the top and bottom of the windows. The expansion anchors were found to be loose, and in some instances, could be pulled out of the concrete block with little effort. They should be firmly grouted in place to provide resistance against negative wind pressures.

Structural projections are a ramp to the second floor on the southeast side, stairs to the third floor on the southeast side and stairs to the second floor on the northeast and northwest sides. There is a concrete canopy over each stair entry. An architectural treatment has been provided for each of the canopies that is not described on the plans. The method of attachment to the structure is unknown. Reinforcement for the canopies is not shown on the plans, but the contractor stated that it was tied into the perimeter beams to resist gravity loads. The stairs are not designed as an integral part of the building, but are designed to stand alone against wind forces. Their failure should not jeopardize the main structure.

Mechanical projections are five air conditioning units, a roof hatch and two solar heating panels. The air conditioning units and panels may be torn loose during a hurricane, but their failure would present no appreciable hazard to the building. If the roof hatch fails, rainfall would enter the interior stairwell between the second and third floors. The only threat to the structure would be if the projections tore loose and blocked the scuppers that drain the roof. Unless all three scuppers become clogged, however, this should not be a problem.



$$
\begin{aligned}
z_{max} &= SWL + 0.55\,(SWL - Grade) \\
&= 10.0' + 0.55\,(10.0' - 7.0') \\
&= 11.7'
\end{aligned}
$$

take $z \cong z_{max}$ since there are few obstructions between shelter and shoreline

Figure 23. Anticipated Category 3 Conditions

An on-site inspection revealed that the center third of one first floor corner column had been drilled out to permit the installation of an electrical conduit (Figure 24). In doing so, one reinforcing bar was exposed and bent out of position. It could not be determined if this was column steel or perimeter beam steel. The inspection also revealed that bottom reinforcing steel in the tie beams above the two original first story walls was exposed (Figure 25). Further it appears that the block for these walls were laid before the adjacent columns were placed and this raises the question as to whether these walls are truly breakaway walls.

Given the problems discovered during the inspection, three questions arise:

1. Were the tie beams redesigned to support the loads they would carry if the walls failed?
2. If the tie beams were redesigned to carry the loads they are subjected to, is there enough steel embedment to act as they were designed?
3. Because of the nature of the construction, how much load would be transmitted to the structural frame before the walls failed?

Because these questions cannot be answered and because calculations show that the structural frame cannot withstand the anticipated wind and wave loads during category 3 conditions if the original first story walls do not break away, the structure cannot be recommended for use as a shelter above category 2 conditions.

Findings and Recommendations - Island Christian School

This structure should not be used as a hurricane shelter until shutter anchors are inspected and replaced as necessary. The first story (i.e., ground level) should not be used for shelter during any hurricane conditions.

An examination revealed that some anchors are missing and others can be removed easily by hand. All anchors should be examined for adequate bond. All missing and insecure anchors should be replaced. With shutters securely fastened, the upper stories of this structure should provide adequate shelter against winds during category 1, 2 and 3 storms.

The second and third stories should provide adequate shelter against flooding during category 1 and 2 storms. While the elevation of the third floor is above the expected level of flooding during higher category storms, it should not be used because of the building's questionable resistance to wave forces during category 3, 4, and 5 storms.

A category 3 storm would be expected to flood the first floor to a depth of 5 feet. Waves during a category 3 storm would probably knock out the "breakaway" walls enclosing the library (since the details of wall construction are unknown, an exact determination of when this would occur cannot be made).

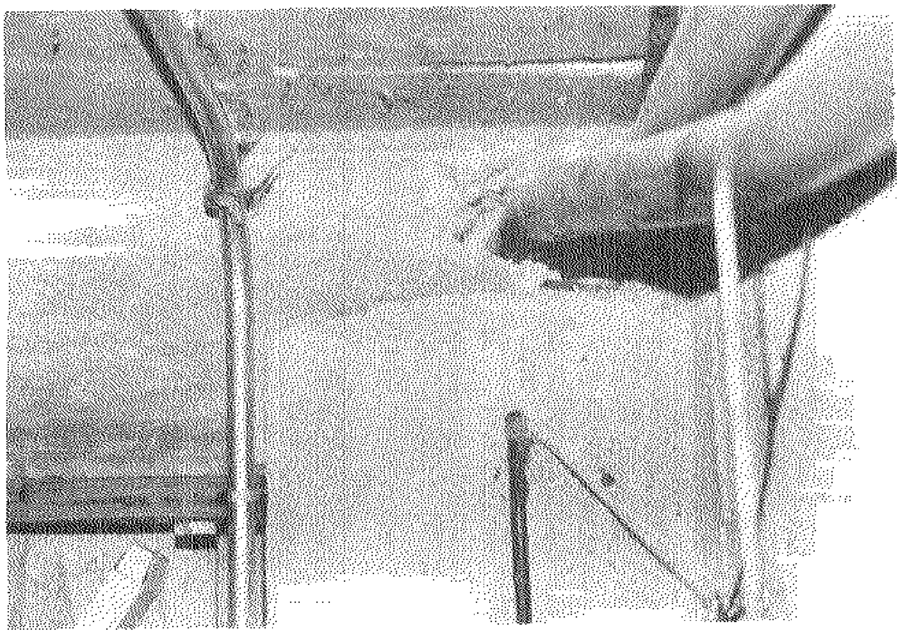


Figure 24. Corner Column was Cut Out to Pass Electrical Conduit

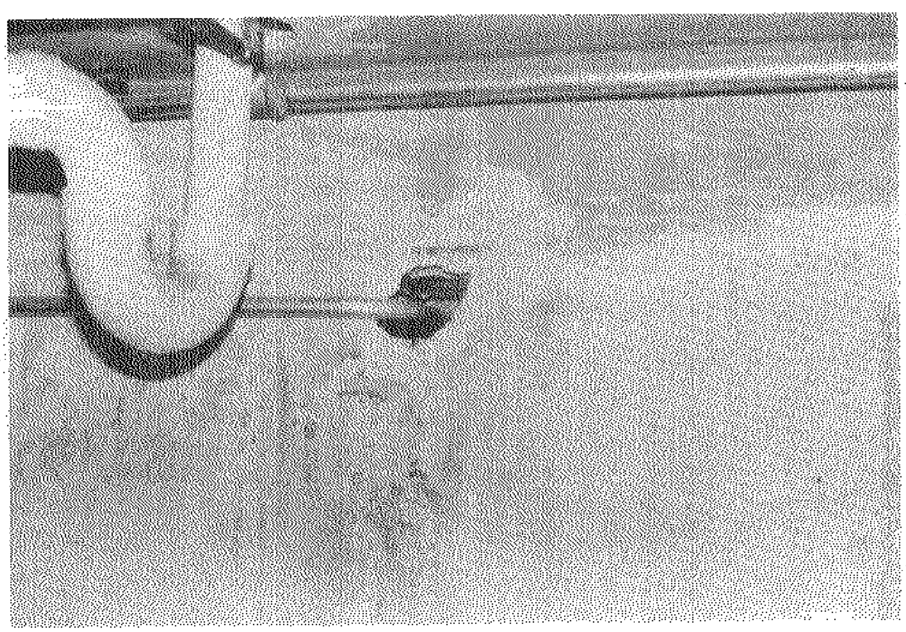


Figure 25. Bottom Steel in Beam is Exposed

Once the "breakaway" walls have failed, the two original walls underneath the second floor slab would be susceptible to wave forces. Calculations show that the wave forces during a category 3 storm, coupled with the wind forces on the upper portions of the building, would exceed the resistance of the building to such forces, if these walls stand. If these walls fail, it is not certain whether or not the building would stand without them.

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