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PRELIMINARY ASSESSMENT OF DAMAGE TO ENGINEERED STRUCTURES CAUSED BY HURRICANE ANDREW IN FLORIDA

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FORWARD:

A post-disaster investigation and data collection team, organized to represent the Applied Technology Council, conducted a field investigation in Florida in the aftermath of Hurricane Andrew. Funding for travel and expenses was provided by the Natural Hazards Research and Applications Information Center. The Lindbergh & Associates field investigation team consisted of two experienced practicing structural engineers and one engineering technician. Dr. Charles Lindbergh, President, Lindbergh & Associates, preceded the team and provided advance coordination and preliminary survey of the damaged areas.

This field investigation provided for the collection of data in the highly populated area along the east coast of Florida, south of Miami, including Coral Gables, Homestead, Florida City, Homestead Air Force Base, and other locations of interest in the vicinity. The investigation emphasized and surveyed damage of primary interest to the practicing structural engineer. One- and two-family residential structures are of significant interest and are, by far, the most frequently damaged. However, this investigation concentrated on non-residential structures which fall into the categories of engineered, pre-engineered, and marginally engineered structures as defined in ATC-26-2.[1] Some data on the fourth category of non-engineered structures, such as most one- and two-family residential construction, was included. The purpose of this investigation is to evaluate how engineered designs performed and what improvements could be made in design methods, construction materials, construction technology, and building code requirements.

This report provides the preliminary observations of the field investigation team. It is emphasized that the findings are based only on the initial assessment of the damage and that further consideration may result in modified conclusions after further study.

METHODOLOGY:

The methods of ATC-26-2[1] and ATC-26-3A[2] were used to guide the data collection effort as related to evaluation criteria, types of damage, human factors and field safety. Photographs of each facility were taken to record: a) the condition of the facility, b) significant damage to structural members and connections that relate to the performance of the structure or non-structural elements, and c) failure mechanisms. A photograph log was maintained to record the location of photographs and features shown.

The team members arrived in Miami the morning of September 5, 1992. The field investigation was conducted during the afternoon of September 5 and all day on September 6 and 7, leaving Miami the evening of September 7. The entire day on September 6 was used to survey damage on Homestead Air Force Base. The remainder of the time was used to survey damage in Florida City, Homestead, and communities north along the South Dixie Highway (US 1) toward Miami.

THE STORM AND GENERAL DAMAGE ASSESSMENT:

The heavily damaged area was bounded on the north by North Kendall Drive (SW 88th Street) approximately 10 miles south of the Miami International Airport and extended approximately 20 to 25 miles south to Homestead and Florida City. Key Largo, approximately 30 miles south of Florida City, was not heavily damaged. East to west, the damage extended throughout the populated area from the coast to approximately 15 miles inland where the storm passed on to the Everglades National Park. Thus, the heavily damaged area was limited to approximately 300 to 400 square miles.

The storm position reported at 11 pm, August 23, 1992, was 25.4 degrees North, 78.1 degrees West as the eye approached the Florida coast. The storm had maintained an essentially constant latitude at 25.4 degrees North which positioned the center of the eye approximately 7 miles south of Homestead Air Force Base. Much of the observed damage in the local communities as well as on the air base indicated the wind to be from a southeasterly or easterly direction consistent with the track of the eye to the south. The National Hurricane Center (NHC) estimated that "the central pressure was 926 mb at landfall near Homestead AFB, Florida at 0905 UTC (5:05 A.M. EDT) 24 August."[3] Preliminary "best track" data from the NHC gives landfall as 24/0905 (UTC), Lat. 25.5 N, Lon. 80.3 W, wind speed 125 kt.[3]

Hurricane Andrew was a much more compact storm than hurricane Hugo. The diameter of the eye of hurricane Hugo was about 30 miles, while hurricane Andrew was approximately 20 miles. Damage from Andrew was minimal north of the center of Miami, approximately 35 miles from the eye, while damage from Hugo was extensive from the storm track just north of Charleston to Myrtle Beach, 100 miles north. On September 20, 1992, The Post and Courier,[4] Charleston, South Carolina, reported the following comparison of Hugo and Andrew citing as sources: The Associated Press, South Carolina State Climatology Office, SCE&G, state Division of Research and Statistical Services.

Description	HUGO	ANDREW
Category	4	4
Hit land	11 p.m., 9/21/89	5:30 a.m., 8/24/92
Sustained winds	135 mph	145 mph
Gusts	Up to 160 mph	Up to 175 mph
Hurricane-force winds	140 miles from eye	45 miles from eye
Tropical-storm winds	250 miles from eye	140 miles from eye
Storm surges	5 to 20 feet	5 to 8 feet
Rainfall	5 to 10 inches	2 to 5 inches

While some news media have reported even higher wind speeds, the above cited wind speeds are those estimated by the National Hurricane Center. These may differ from winds speeds used for engineering design which are based on a fastest-mile wind speed at a standard height of 33 feet above ground in open terrain with scattered obstructions including flat open country and grasslands. The Wind Engineering Research Council[5] reported: "wind speeds, converted to standard engineering reference conditions, of between 110 and 125 miles per hour." Further study will refine the wind speed estimates. It is not within the scope of this investigation to address the wind speeds which occurred.

The observed damage was indicative of the differences between Andrew and Hugo. Andrew appeared to be a more intense storm causing heavy damage in a limited area that was densely populated. The damage from Hugo was probably almost as intense in the area of Awendaw and McClellanville, approximately 20 to 40 miles north of Charleston; however, the area affected by this high intensity was much less densely populated. Had the track of hurricane Hugo crossed the South Carolina coast 20 to 30 miles further south, the effect of the most intense winds and flooding on the peninsula city of Charleston and the nearby suburbs would have been much more disastrous. Likewise, Hugo continued causing widespread damage extending 200 miles inland, whereas Andrew quickly passed the populated area to the Everglades and continued over water where it ceased to be a hazard to Florida.

FACILITY DAMAGE OBSERVATIONS, HOMESTEAD AIR FORCE BASE:

Approximately 40 percent of the field investigation time was devoted to Homestead Air Force Base (HAFB) which provided a unique laboratory for damage evaluation. HAFB consisted of an excellent concentration of industrial facilities ranging from large aircraft hangars and aviation fuel storage tanks to office and recreation buildings. The construction types included concrete frame, precast concrete tilt-up, steel frame, metal buildings, engineered timber, wood frame, and reinforced and unreinforced masonry. The major construction is presumed to have been designed and built to military standards and inspected during construction by representatives of the Air Force as well as the designated construction agent (Corps of Engineers or Naval Facilities Engineering Command). Therefore, it is expected that the construction would exceed the norm in terms of design and construction quality at the time the facilities were built (mostly post World War II, with many facilities of 1950's and 1960's vintage). However, various facilities would be classified as engineered, pre-engineered, and marginally engineered. Additionally, the base contained family housing (non-engineered) which was not surveyed by this team. Observations of damage to facilities on Homestead Air Force Base follow:

- Virtually all aircraft hangars had hangar doors blown off. Most of the doors did not structurally fail and were not extensively damaged. The primary cause of failure appears to be loss of support cause by uplift on the roof structure by the wind. Some doors were vertically supported by a bottom track and laterally supported by rollers in a channel at the top (for example the largest hangar) so that the roof uplift raised the channel freeing the rollers at the top, allowing the doors to fall into the hangar or outside of the hangar depending on the wind direction. Some doors were vertically hung from a track at the top as well as laterally supported by a track at the bottom (the F-16 alert hangar). In this case, the uplift on the roof lifted the door off the bottom track allowing the wind to swing the door inward. As it swung inward, one door bottom scraped against a masonry wall showing that the door had swung to an almost horizontal position. A number of the older hangars collapsed probably due to internal wind pressure adding to external suction. The doors of most hangars, with the exception of the largest hangar, faced southeast. The F-16 alert hangar had doors facing southeast and on the back side facing northwest. The doors on the largest hangar were on the northeast and southwest sides. Greater consideration of deflections of structures due to wind uplift is required. Consideration should be given to retrofit of hangars using cable tiedowns which could be installed temporarily in advance of a hurricane to limit upward deflections at mid-span of openings and other appropriate locations.

- Metal roll-up doors failed, usually due to positive wind pressure on the windward side. Failure of supports at the top occurred in most cases, sometimes associated with failure of the side tracks or the door itself. In some cases, failure of roll-up doors triggered more catastrophic structural failures due to opening of the facility to internal wind pressure. Sturdier roll-up doors are required as well as greater attention to the attachment of the doors to the building structure.

- Metal roofing and metal siding did not perform well. Hangars lost major sections of roof decking on both flat and sloping roof surfaces, probably due to addition of internal wind pressure after failure of hangar doors. Many failures of metal roofing and siding were observed that were not related to internal wind pressure or other factors beside the weaknesses of the metal cladding system. Bending of metal siding occurred without failure of the fastening and some failure of girts supporting siding was noted; but the major problem was failure of the fastening system to resist complete removal of the roofing or siding. Standing seam roofs generally showed evidence of failure at the attachment cleat interface with the metal roofing. The cleats and the panel clips remained attached to the purlins. Heavier gage metal roofing and siding is required as well as improved fastening systems.

- Numerous concrete block masonry buildings failed by collapse of the walls, usually in conjunction with loss of roof framing which voided the lateral support for the wall at the top. Masonry construction on Homestead Air Force Base did not incorporate the tie beams and tie columns required by the empirical provisions of the South Florida Building Code[6] but probably satisfied the code provisions for masonry designed at the time of construction. Most walls had reinforced bond beams at the top and reinforced pilasters spaced at 20 to 25 feet. Generally, cells between pilasters were not reinforced and grouted. Joint reinforcing was used, sometimes at every block course, but was not effective. It is apparent that walls need to be tied to the bond beam with vertical cell reinforcing. Minimum reinforcing of masonry walls, comparable to that required for earthquakes, is required for wind. Existing masonry buildings should be inspected and retrofited by adding reinforcing or other strengthening measures where analysis establishes hazardous conditions.

- The F-16 alert hangar had 25 to 30 feet high concrete block bearing walls supporting steel truss joists on the hangar ends and between bays. The concrete block walls were undamaged except for the bond beam at the top. The truss joist anchors, consisting of a plate with three bent bars, pulled out of the bond beams resulting in the loss of significant portions of the roof structure. A deficiency was found in these connections which had the anchor bars bent parallel to the bond beam reinforcing rather that perpendicular such that they would hook around the bond beam reinforcing. Thus, the failure of the anchor system resulted from tensile failure of the bond beam concrete without significant interlock with the bond beam reinforcing.

Greater attention to standard construction details and practices is required, including construction inspection.

- The performance of precast concrete tilt-up wall construction was varied. Performance of precast T-beam and double T-beam roof structures depended on the supporting structural systems and the connection between the roof and wall structure. Some tilt-up wall construction and precast T-beam roof structures were essentially undamaged. Other tilt-up and T-beam structures had collapsed. A fairly new medical supply warehouse at the air base had only minor damage except for the northeast end wall. Of the six precast flat wall panels on the northeast end, two panels had fallen outward and a third panel was near failure, having separated approximately one foot from the roof. The failure apparently occurred after destruction of two large roll-up doors on the southeast side, resulting in interior pressure adding to suction on the side wall. The failure of the wall was caused by weakness of the top connection of the precast panels anchor bolted to an angle (approximately 3 x 3 x 1/4) which was welded to the bar joist bridging (approximately 1 x 1 x 1/8 angles). Greater attention to the design of the top connection was required.

- The structural system for the Base Chapel was a laminated timber "A-frame" with 4 inch timber tongue-and-groove roof decking. The building had partial poured-in-place concrete end walls. The ridge of the building was oriented north-south. Winds causing failure were apparently from an easterly direction. Failure probably occurred at the lower end connections for the "A-frame" which consisted of two 7/8 inch bolts with shear plate connectors connecting the laminated timber to a steel box. The steel boxes were distorted, the head ends of most bolts were in their holes, but the threaded ends were not in their holes. Except for one of approximately 16 connections, there were no nuts on the bolts and the bolts had not sheared or failed other than minor bending. It appears that the laminated timber connections failed due to inadequate end distance, pulling the bolts through to the end of the timber. This failure mode may have been aggravated by the lack of nuts on the bolts allowing the threaded end of the bolts free from the supporting steel box. This scenario would allow bending of the bolt, cause high concentrations of stress between the bolt and the wood, and defeat the effectiveness of the shear plate connectors. More consideration of uplift and overturning forces and the end distance capacity of the bottom connection was required. Quality inspection of construction should have caught the deficiency if the nuts were not on the connection bolts.

- There was some damage to an aviation fuel storage tank which buckled under the wind pressure. The tank was estimated to be 100 feet in diameter and 48 feet high. The maximum fuel height was painted on the tank as 44'-6" (calculated 2.6 million gallon capacity) and the sight gage read 28'-2.5" (calculated 1.66 million gallons) on

September 6, 1992. It is not known whether the fuel level had changed since 24 August when hurricane Andrew struck. The fuel tank was contractor operated for the Air Force and the contractor was not available at the time of the inspection. The buckling was principally above the 32 foot height with some indentation extending down to the 28 foot level, approximately the height of the fuel as of 6 September (and perhaps on 24 August). The buckling was on the southeast side of the tank, giving an excellent confirmation of wind direction coming from the southeast as observed from other damage. Consideration should be given to improving procedures for increasing fuel storage levels in advance of a hurricane.

FACILITY DAMAGE OBSERVATIONS, COMMERCIAL AND INDUSTRIAL:

The following observations were made during approximately one and one-half days of surveying facilities in the high damage areas, mostly in Homestead and Florida City. Residential construction will be addressed in the next section.

- The structural frames of high rise buildings were relatively undamaged. High rise buildings did suffer considerable damage to roofs, glazing, and some siding. Observations of high rise buildings during this investigation were minimal.

- Many pre-engineered buildings suffered significant damage or collapse. As observed in previous wind damage to pre-engineered buildings, one major cause was the failure of roof purlins in upward buckling under the combined loads of wind uplift and compression on the windward end of the building. In some cases, the failure of purlins may have been aggravated by supporting downward loads on the purlins, such as air conditioning equipment, which may not have been included in the design. Failures of base connections were observed, including anchor bolt pullout, tension failure of anchor bolts, weld failure between the column and the base plate, and base plate failure. However, many of these failures were probably caused by prying action of higher than design moments resulting during collapse of the building. Greater attention to the strength and stiffness of purlins, girts, and metal roofing and siding is required. Additional redundancy of pre-engineered structures would help. Careful consideration of uplift is required in the design of foundations and base anchors.

- As on Homestead Air Force Base, metal roofing and siding performed poorly and, without doubt, contributed to building damage and collapse. Siding failed in bending under positive pressure on the windward wall and negative pressure on the leeward wall and side walls. Some failure of girts supporting siding was noted. The most significant failure was the attachment of the roofing and siding. A major percentage of metal roofing and siding panels was missing. These panels became large hazardous missiles that wrapped around whatever was in their path. Standing seam roofs generally failed at the interface between the cleat and the metal roofing seam. Predominantly, the cleats and panel clips remained attached to the purlins. Heavier gage metal roofing and siding is required in addition to significantly improved fastening systems.

- As noted earlier, some precast concrete tilt-up wall construction performed well. Performance of precast T-beam and double T-beam roof structures depended on the supporting structural systems and the connection between the roof and wall structure. The connection of the structural components to resist lateral wind loads, and to a lesser degree uplift, is critical. The failure of a large furniture warehouse, approximately 400 feet long, 140 feet wide, and 45 feet high (42 feet to bottom of roof structure) was most notable. The building had tilt-up single-T walls on the long southeast side and on both ends. The long wall on the northwest side was concrete block with a grid of heavy poured concrete tie beams and tie columns (or a concrete frame with 12 inch concrete block masonry infill). There was a line of precast concrete columns the length of the building at 80 feet from the southeast wall (60 feet from the northwest wall). The roof structure consisted of precast single-T's spanning 60 and 80 feet across the width of the building. The roof structure was supported by shelf brackets poured with the precast tilt-up southeast wall, on an inverted "T" on top of the columns in the center, and embedded in the poured concrete tie-beam at the top of the northwest wall. The two ends of the building remained standing except for the north corner. The center half to two-thirds of the building had collapsed with the southeast wall falling onto the floor of the building. The failure of this building was apparently caused by horizontal flexural deflection of the roof diaphragm. Without interior shear walls, the roof diaphragm span was 400 feet and the steel plate connections between single-T's apparently did not provide sufficient diaphragm stiffness. The resulting deflection resulted in the center portion of the building becoming unstable. The end shear walls provided adequate stiffness to prevent collapse of the two ends of the building with the exception of the north corner. The configuration of the north corner is unknown but probably had large openings as indicated by a wide ramp. This failure resulted from winds from a southeasterly or southerly direction causing internal pressure in the north corner of the building. A contributing cause of failure could have been inadequate connections between the roof structure and the columns and walls. Greater attention to deflection of the diaphragm was required as well as connection details for precast concrete structures.

- The performance of masonry structures varied. Where major wall failures occurred, the failure usually involved loss of wall support at the top or openings allowing internal pressure effects. One predominate weakness was observed in the attachment of masonry veneer to the structural masonry walls; often, only sparsely spaced corrugated metal ties could be found. Many cases were seen where veneer was lost in high suction areas near corners or parapets. Parapet damage also occurred frequently.

FACILITY DAMAGE OBSERVATIONS, RESIDENTIAL:

Although residential construction was not the primary focus of this investigation, some common observations were made regarding the types of failures and strengths or weaknesses.

- Use of metal ties (hurricane clips) was predominantly evident. This probably accounts for minimal failure of the connection between the walls and roof at the eaves. No complete separation of intact roof structures from walls was observed. However, failures of gable ends occurred extensively throughout most housing areas. The gable ends were not adequately tied back to the roof or attic structure. In the majority of instances, the roof sheathing provided the only lateral support for the gable ends and racking resistance of interior trusses. In some cases, the failure initiated at the ceiling line with both the gable end and the wall below being sucked out. In other cases, the lack of restraint of the gable end against positive windward pressure resulted in the gable end and roof trusses collapsing into the structure in domino fashion. The lateral support for the gable end derived from the roof sheathing and sometimes 2 x 4 bracing tied back to truss web members or other locations. However, the connection of the 2 x 4 bracing was frequently made with toenails or end nailing. It was not uncommon for the metal ties between the gable end and the wall to hold, leaving the gable end suspended upside down against the lower story. The loss of gable ends also appeared related to loss of the attached roof sheathing in the high suction area of the rake. The nailing schedule for roof sheathing required by the South Florida Building Code[6] is consistently not being met. Generally, only 50 to 70 percent of the required nails were found in the roof sheathing. A closer nailing schedule is required in the high suction areas around the perimeter of roofs, especially into the gable end framing and the adjacent truss. Improved bracing of gable ends is required as well as adequate nailing of the roof sheathing.

- Hip roofs performed well, not having the susceptibility of the gable end.

- Loss of roofing was extensive resulting in water damage to the interior of the house and the contents. Tile roofs, both clay and cement, were common and suffered greatly, with the additional hazard that the loose tiles became heavy airborne missiles. The only observed attachment for tile was setting the tile in a cement mortar. More wind resistant shingles or other roofing systems are required. The used of tile roofs should be evaluated and/or improved attachment methods developed.

- The initial impression of total destruction was misleading. Many houses will be repaired or rebuilt since most walls were standing with relatively little damage and significant percentages of the roof structures were intact. Careful post disaster inspection of houses should be done by qualified persons before a demolish or repair decision is made. The inspection should carefully search for hidden damage or weaknesses. The results of the inspection should be documented for follow-up by building inspectors.

- Mobile homes were virtually demolished. The design, construction, and tie-down methods used for mobile homes requires thorough review. Establishment of new standards for their construction and tie-down should receive a high priority.

SUMMARY AND CONCLUSIONS:

Much can be learned by studying the performance of buildings that have been stressed to the limit during a natural hazard such as hurricane Andrew. Of great concern is that we continue to observe damage that we have had the technical ability to prevent, having observed the same deficient performance before and developed an understanding of the failure mechanisms. Unfortunately, there is a significant breach between building well and building cheap and cheap usually wins out. No one bears all the blame, nor is any sector of the building team faultless; not the engineers or architects, not the building inspectors, not the construction materials manufacturers or fabricators, not the contractors, not the home builders, and not the owners. Contrary to the attitude of some, the damages suffered due to hurricane Andrew and other such natural disasters are not unpreventable acts of God. Most, if not all, failures of facilities can be attributed to some identifiable shortcoming in the design and/or construction technique that might have been foreseen and minimized. Some thoughts follow.

- Building codes and standards should be improved, but tighter codes will not solve the problem without well qualified and trained building inspectors. Improved building codes will, at best, only solve the problem for buildings to be built under the new codes. Since the life of most buildings is 50 years or more, there needs to be a requirement for upgrading existing buildings or the hazards will not be eliminated and the losses due to natural disasters will continue beyond our lifetimes.

- More observation of structures during and after construction should be done by the design team of engineers and architects, a service for which owners are generally reluctant to pay.

- Design for natural hazards should be stressed with more consideration given to deformation of the structure under lateral loads and uplift. Much greater attention to detail is required, especially to connections.

- Much of the building design is defaulted to the construction material manufacturer or fabricator. Improved standards for all construction materials are required, especially cladding and associated attachments. The non-structural elements of buildings are just as important as the primary structure and the losses caused by their failures are probably greater.

- Contractors and home builders should be held accountable for the adequacy of their work. More frequent and thorough inspection will help. Increased requirements for licensing of contractors should be established and feedback on substandard contractor performance should be considered in license renewal. Unlicensed builder and contractor activity should be restricted.

- Owners must be required or provided with the incentive to invest the extra small percentage to build to a higher standard. The small increase in cost is cheap insurance against future disaster. Insurance rates should consider quality and adequacy of construction. If an owner builds to a lower standard, he should share in the risk.

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