Building Damage in South Carolina Caused by the Tornadoes of March 28, 1984 (1985)

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Building Damage in South Carolina Caused by the Tornadoes of March 28, 1984

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For:

Committee on Natural Disasters
Commission on Engineering and Technical Systems
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FOREWORD

The Committee on Natural Disasters of the National Research Council was formed to study the impact of natural disasters such as earthquakes, floods, tornadoes, and hurricanes on engineered structures and systems. The objectives of the committee's work are to improve protection against disasters by providing factual reports of the consequences of these extreme events of nature and to stimulate research needed to understand the hazards posed by natural disasters.

When the tornadoes of March 28, 1984, struck a number of small towns in South Carolina, Peter Sparks of the Department of Civil Engineering, Clemson University, was asked by the committee to survey the damage to engineered structures and prepare a report. Professor Sparks is one of several structural engineers who had previously volunteered to make postdisaster studies for the committee. Following the field survey, a structural analysis and a wind tunnel test of one of the damaged commercial buildings were completed at Clemson University. This report to the Committee on Natural Disasters covers Professor Sparks' field survey immediately following the tornadoes and the subsequent analyses and wind tunnel test.

Kishor C. Mehta, Chairman
Committee on Natural Disasters

ACKNOWLEDGMENTS

The initial cost of this damage survey was borne by the Committee on Natural Disasters of the National Research Council. Subsequently, the College of Engineering, Clemson University, provided funds to enable a structural analysis and wind tunnel test to be made for one of the damaged buildings. The support of these organizations and of the two graduate students, Vipul Desai and David Wright, who assisted with these investigations is gratefully acknowledged. In addition, the assistance of Drake Rogers in providing details of buildings in Bennettsville, South Carolina, and James Altman in providing information about the implementation of building codes in South Carolina is gratefully acknowledged.

CONTENTS

1

INTRODUCTION AND SUMMARY

On Monday, March 26, 1984, a low-pressure system formed in West Texas. As it crossed the Midwest it increased in strength. The hot, dry air from the southwest gathered moisture from the Gulf of Mexico and met with cold dry air from the interior of the continent, creating an unstable air mass. By the morning of Wednesday, March 28, heavy rain and strong winds were being experienced in Georgia. By the time the storm reached South Carolina, conditions were ideal for the creation of tornadoes. The National Severe Storms Forecast Center of the National Weather Service issued its first tornado watch at 2:15 p.m. Record low pressures were recorded as the storm passed, accompanied by severe thunderstorms, heavy rain, and hail.

The first tornado was reported by a South Carolina state trooper near Ware Shoals, about 20 miles from the Georgia border, at 4:35 p.m. The Columbia office of the National Weather Service issued a tornado warning at 4:45 p.m. Between 4:35 p.m. and 10:50 p.m. the storm traveled northeast across South Carolina and North Carolina, killing at least 57 people, injuring approximately 1,300, causing over $200 million of damage, and leaving more than 3,000 people homeless.

Figure 1 shows the track and intensity of each tornado as identified from an aerial survey by T. Theodore Fujita and his colleagues at the University of Chicago (National Oceanic and Atmospheric Administration, 1984). The intensities reported in the survey ranged from F1 (moderate damage) to F4 (devastating damage) on the Fujita damage scale. Path lengths varied from 1 to 45 miles, and the mean path width from 0.1 to 1.5 miles. The longest tornado (the McColl tornado) was also the widest, with a maximum width of 2.5 miles.

Although no more tornadoes were reported after the storm left North Carolina, strong winds and heavy rain or snow occurred along the Atlantic Coast into New England. Further details of the storm can be found in the report produced by the National Oceanic and Atmospheric Administration (1984).

This report discusses the damage observed in three locations in South Carolina: Newberry in Newberry County, Winnsboro in Fairfield County, and Bennettsville in Marlboro County. Although damage occurred in other parts of the state, these three locations accounted for the majority of the damage in South Carolina. They also exhibited damage to the greatest variety of buildings. The damage discussed in this report was inspected on March 29 and 30.

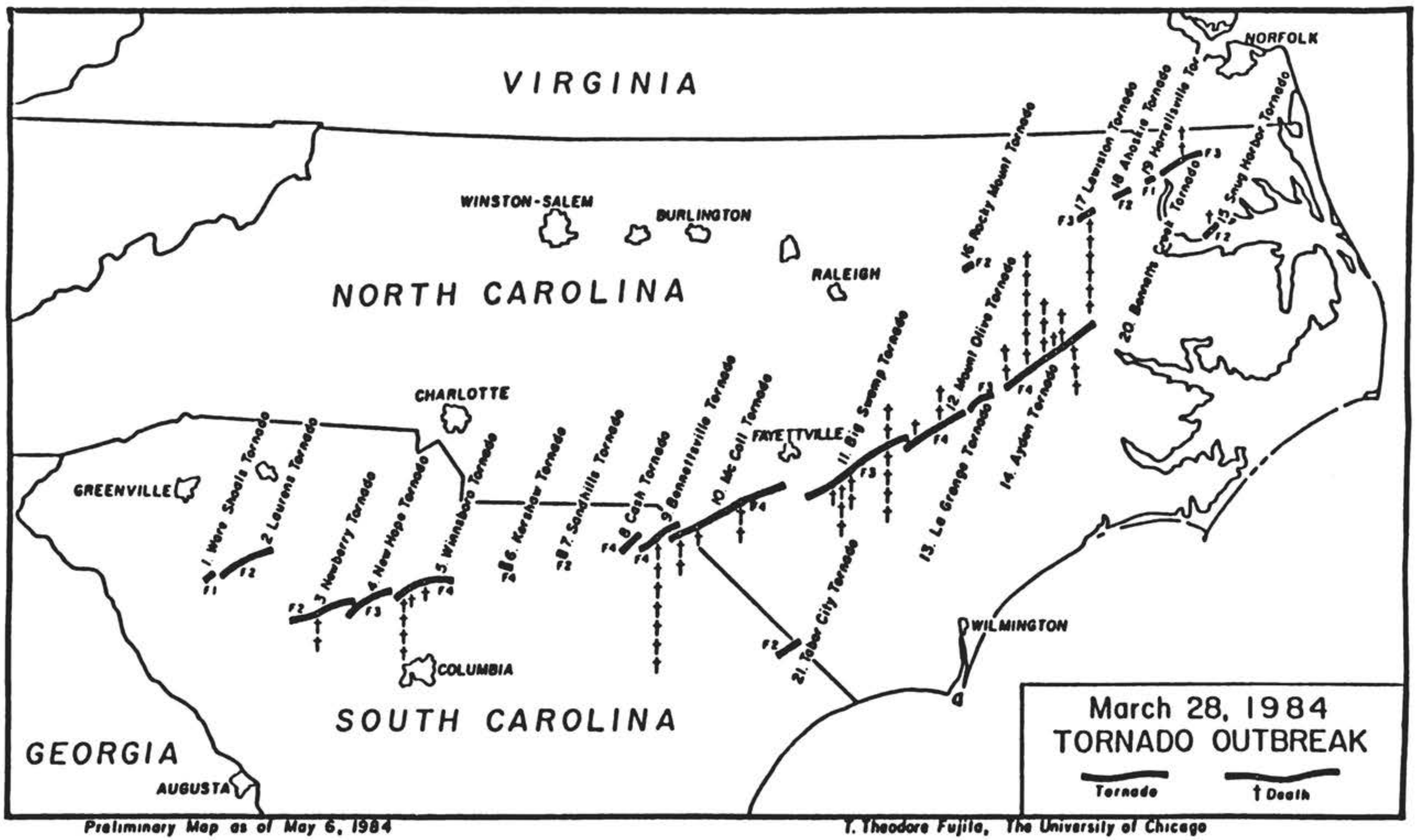


FIGURE 1 Tornado tracks and intensities. (Source: National Oceanic and Atmospheric Administration, 1984.)

In Newberry the tornado literally ran along Main Street, damaging buildings that ranged from modern commercial buildings to churches and assembly buildings over 100 years old.

In Winnsboro the tornado followed an arc around the town, striking domestic housing, steel-framed commercial and government buildings, a private school, a church, and several mobile homes.

In Bennettsville the tornado also skirted the town, severely damaging a shopping center, apartment complexes, and a variety of domestic dwellings.

The Newberry tornado was classified by Fujita as F2 and the Winnsboro and Bennettsville tornadoes as F4 (see Figure 1). On the Fujita scale an F2 tornado is associated with wind speeds of 113-157 mph and an F4 tornado is associated with speeds of 207-260 mph. An engineering analysis of one of the structures and comparison of the damage with that caused to other structures by hurricanes with known wind speeds suggested that, although the damage descriptions were appropriate, the wind speeds were probably much lower, perhaps less than 150 mph.

Fujita also reported that the mean path width of these tornadoes was large: 0.6 mile in the case of the first two, and 1.2 miles at Bennettsville. The wind speed within these tornadoes would therefore have varied considerably. As a consequence, this report discusses damage by type of building rather than by location.

As expected, mobile homes performed very poorly in the tornadoes, with nearly 40 percent of the storm-related deaths occurring in these structures. In some cases, mobile homes were completely destroyed while adjacent structures were virtually undamaged. The performance of domestic dwellings varied considerably depending on location and quality of construction. Public buildings constructed of unreinforced load-bearing masonry suffered extensive damage. Steel-framed public buildings performed much better, although cladding and secondary members were often severely damaged. A particularly disturbing failure was that of the Northwood Village Shopping Center in Bennettsville, which had a hybrid steel and masonry form of construction similar to that of hundreds of shopping centers in the region.

For wood-framed buildings, some simple improvements in construction techniques, such as securing rafters or roof trusses to the frames and the frames to the foundation, could significantly reduce the risk of collapse and loss of life in future severe storms. Good roof ties and the provision of vertical reinforcement in walls could have a similar effect for masonry structures.

There is no state-wide building code in South Carolina. If a jurisdiction adopts a building code, it must be one approved by the state, which currently means the Standard Building Code (Southern Building Code Congress International, 1982). However, most rural areas have not adopted a building code, and even when a code has been adopted, inadequate resources often limit its enforcement. This has had an adverse effect on the quality of construction in general and on the ability of structures to resist the natural hazards of tornadoes, hurricanes, and earthquakes to which the area is prone in particular. The state-wide enforcement of appropriate building regulations could greatly reduce damage and loss of life in future events of this type.

2

SINGLE-FAMILY DOMESTIC DWELLINGS AND MOBILE HOMES

The areas struck by the South Carolina tornadoes contain a variety of domestic housing, but the residents of many of the areas are relatively poor. The average per capita income in South Carolina as a whole is $8,039. In Marlboro County it is only $5,485. Thus, although the areas affected contain luxury homes, and some of these were damaged by tornadoes, much of the housing stock is that of low-income families, and mobile homes form a significant portion of that.

There are essentially two forms of conventional construction in South Carolina, wood-framed and load-bearing concrete block. The wood-framed houses are clad with wood siding or brick veneer. Basements exist in some dwellings in Newberry and Winnsboro but are rare in the Bennettsville area. Since all three towns are at least 80 miles from the sea, building codes contain no special requirements for roof clips or straps that might exist in a hurricane-prone coastal area. Masonry construction does not normally contain vertical reinforcement.

Examples were found of both masonry and framed construction in which the buildings had been completely destroyed, reduced either to a pile of rubble or widely scattered pieces of wood (Figures 2, 3, and 4).

In better constructed single-story buildings of frame construction, the damage was often restricted to the loss of roofs, windows, and portions of exterior walls (Figure 5). Internal corridors often remained sufficiently intact to provide shelter for the occupants (Figure 6). In other instances, basements provided a safe refuge even though the house above was completely destroyed. In Winnsboro, one fatality occurred when an unreinforced masonry wall of a house collapsed on an occupant. The shelter of a bed was sufficient to ensure the survival of others in the building.

Because the effect of tornadoes is very localized, it is difficult to compare forms of construction or different locations. However, the most severe damage--the complete removal of houses--took place on isolated open hilltops in the Winnsboro area, where the tornado appears to have jumped from hill to hill.

In the Bennettsville area, where the terrain is flat, the presence of a heavily wooded area around the houses appeared to have reduced the damage, although the area struck was of relatively expensive homes. As expected from aerodynamic considerations, steep-pitched roofs appeared to have performed better than shallow-pitched roofs.

In many instances, roofs failed either intact or in large sections, the toe-nailed connections having been inadequate in the case of frame construction and the anchors, if provided, not holding in the case of masonry construction.

As has been observed many times before, mobile homes performed extremely poorly (Figures 7 and 8). Nearly 40 percent of all the deaths in the storm occurred in mobile homes. Most of the mobile homes damaged were not fitted with tie down straps. In some instances they were completely destroyed while adjacent buildings of conventional construction, albeit rather poor, were virtually undamaged.

An isolated tornado touched down in Anderson County near the Georgia border and completely destroyed a mobile home that had been well tied down. In some places the ground anchors pulled out, but in other places the tie-down straps failed (Figures 9 and 10).

FIGURE 2 Modern frame house in Winnsboro.

FIGURE 3 Older frame house in Winnsboro.

FIGURE 4 Masonry house in Winnsboro.

FIGURE 5 Modern frame house with brick siding in Winnsboro.

FIGURE 6 Detail of interior corridor.

FIGURE 7 Mobile home in Winnsboro.

FIGURE 8 Mobile home in Winnsboro.

FIGURE 9 Mobile home in Anderson County.

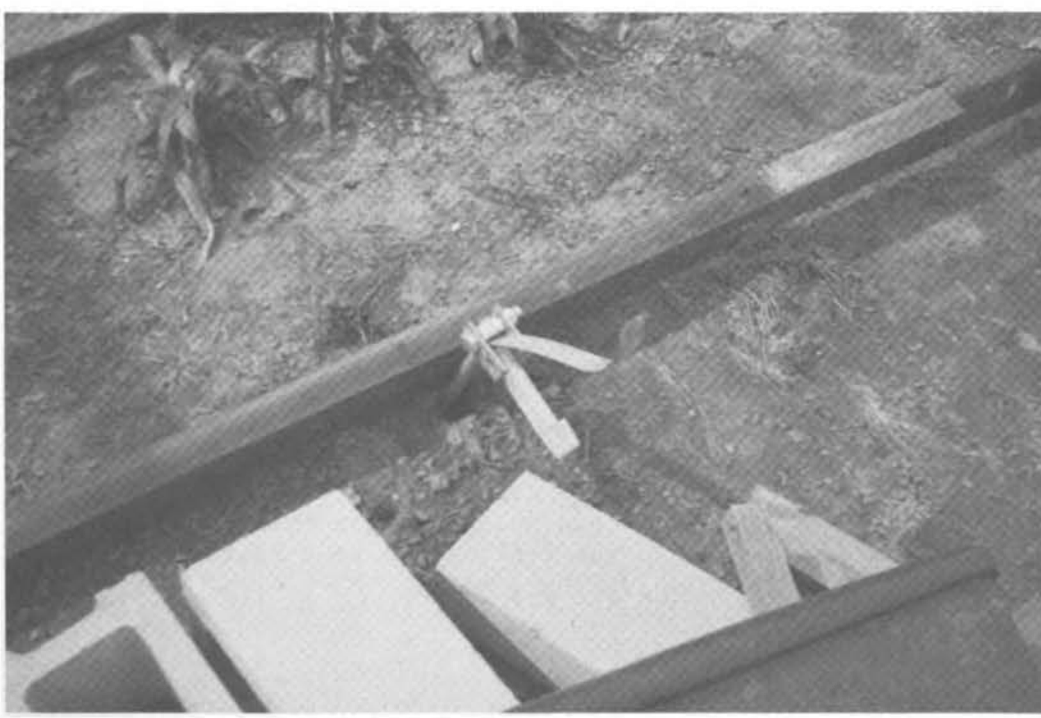


FIGURE 10 Failed tie-down strap.

3

MULTIPLE-FAMILY DOMESTIC DWELLINGS

Two apartment complexes were hit by the Bennettsville tornado. Neither was more than two stories high, and their form of construction was essentially the same as that of a framed single-family dwelling. Damage ranged from almost complete collapse of a structure to localized roof damage. Figure 11 shows the more heavily damaged complex. Figures 12 and 13 show typical damage in an adjacent complex.

Examination of the failed structures revealed examples of both good and bad construction that may have accounted for the differences in their ability to resist wind forces. The location of the tornado and changes in wind speed due to shelter must of course also be considered.

The more heavily damaged complex had brick veneer siding, which in some instances appeared to have been poorly tied to the framing (Figure 14). Corner bracing of plywood sheeting was provided in some buildings but was not continuous from floor to floor (Figure 15). In other buildings, diagonal ties were provided, with only styrofoam between the brick and the framing (Figure 16). Under the action of extreme winds, the brick veneer on the sides appeared to have been stripped from these buildings. Once that occurred, apart from the weight of the building, only the nails used to locate the framing, acting in withdrawal, were available to resist the overturning moment of the wind. In many instances this proved inadequate.

In the adjacent apartment complex, where damage was not as severe but where wind speeds may have been lower due to the path of the tornado, an example of better bracing was discovered. Here the metal ties and plywood overlapped at the corners (Figure 17). Exterior wood siding also appeared to have provided some continuity (Figure 18). The single-story rental office of the complex revealed, however, that, although the bottom plate of the frame had been bolted to the foundation, the rest of the framing had been inadequately attached to that plate (Figure 19). This building was carried away by the wind.

FIGURE 11 Hillcrest apartments in Bennettsville. (Photograph courtesy The State.)

FIGURE 12 Marlboro Court Apartments in Bennettsville.

FIGURE 13 Marlboro Court Apartments in Bennettsville.

FIGURE 14 Detail of Hillcrest Apartments showing unused masonry ties.

FIGURE 15 Detail of Hillcrest Apartments showing lack of continuity of plywood sheeting.

FIGURE 16 Detail of Hillcrest Apartments showing poor diagonal bracing.

FIGURE 17 Detail of Marlboro Court Apartments showing better bracing.

FIGURE 18 Detail of Marlboro Court Apartments showing continuity provided by wood siding.

FIGURE 19 Rental office of Marlboro Court Apartments.

4

PUBLIC BUILDINGS

A number of buildings to which the public had access were damaged by the South Carolina tornadoes. They can be classifed as load-bearing masonry, steel-framed, and a hybrid system of steel and masonry.

LOAD-BEARING MASONRY

On the whole, buildings constructed of unreinforced load-bearing masonry performed extremely poorly.

A dance academy housed in an old brick building in Newberry collapsed entirely (Figure 20). Fortunately, the occupants sought shelter under a stairway and survived with minor injuries. A short distance away an auto parts store consisting of 18-ft-high hollow masonry walls faced with brick and supporting heavy steel roof trusses collapsed, killing the manager. Figure 21 shows the structural arrangement after the debris had been cleared. Also in Newberry, St. Luke's Episcopal Church, which is listed in the register of historic monuments, suffered severe roof damage and the collapse of one load-bearing masonry wall.

In Winnsboro a number of retail stores and workshops made of unreinforced load-bearing concrete masonry units either collapsed entirely or suffered severe damage. An example is shown in Figure 22. One wing of a private school, the Richard Winn Academy, of similar construction also suffered severe damage, mostly from the wind but possibly also from school buses reportedly seen flying through the air during the tornado. The construction of this part of the school followed the normal practice of using unreinforced walls and short roof anchors. Had the tornado struck earlier, or had after-school activities not been canceled due to the bad weather, serious injury and loss of life might have resulted. There was no safe refuge area in this section of the school, and a lightweight annex building was completely carried away (Figures 23, 24, and 25).

A church also collapsed in Winnsboro (Figure 26). The method of construction again followed the common local practice regarding slenderness of the walls, the positioning of pilasters, and the connection of the roof to the walls. The complete inadequacy of such roof-wall connections in resisting uplift forces is clearly shown in Figure 27. The plane of weakness is simply transferred from the roof-wall connection to

FIGURE 20 Dance academy in Newberry.

FIGURE 21 Auto parts store in Newberry.

FIGURE 22 Masonry retail store in Winnsboro.

FIGURE 23 Richard Winn Academy in Winnsboro.

FIGURE 24 Classroom at Richard Winn Academy.

FIGURE 25 Classroom at Richard Winn Academy.

FIGURE 26 Church in Winnsboro.

FIGURE 27 Detail of church showing roof-wall connection.

the block-block connection in the row below. The uplift forces not balanced by the weight of the roof must be resisted by the tensile strength of the mortar. The walls of the church also collapsed easily when the resisting diaphragm of the roof was removed. The building was unoccupied at the time, but the consequences of a tornado striking this church during a service are obvious.

STEEL-FRAMED BUILDINGS

Buildings consisting of moment-resisting steel frames appear to have performed well. Although the cladding and secondary members such as purlins were often severely damaged, the buildings maintained their structural forms and would have provided safe refuge during the storm. Figures 28 and 29 show the Fairfield County Community Center in Winnsboro. Contrast the damage to this structure with the severe damage to the masonry building next to the center shown in Figure 22. Nearby, the gymnasium of the Richard Winn Academy also suffered damage only to its cladding (Figure 30).

Figures 31 and 32 show typical damage suffered by retail stores of metal construction, the first in Winnsboro, the second in Bennettsville.

HYBRID STEEL AND MASONRY CONSTRUCTION

One of the most disturbing examples of damage was the failure of the Northwood Village Shopping Center in Bennettsville. This structure was designed by a firm of architects and engineers and constructed in an area where the Standard Building Code had been adopted. Although no plans of the structure had been lodged with the Building Inspection Department, a set was obtained from another source, and an analysis was made of the building system. A wind tunnel study was also conducted on a model of the structure, as described later in this chapter.

During the course of the storm, a tornado over 1 mile in diameter appeared to have passed over the shopping center, producing a horizontal velocity gradient across the front of the building. The wind appeared to have blown primarily onto the front of the building, causing the most damage on the west end.

Figure 33 shows an overall plan of the shopping center. The structural system consisted essentially of a built-up roof system overlaying rigid insulation and a 1.5-in.-deep corrugated metal deck of 22 gage galvanized steel. The metal deck was spot welded to bar joists (metal truss rafters) that spanned between girders or between girders and masonry walls. Figure 34 shows a roof framing plan. The girders themselves were supported on 5-in.-diameter columns or by masonry walls. The front wall of the building consisted of glass or 4-in. brick with an 8-in. backup wall of hollow concrete masonry units. The side, rear, and partition walls consisted of 12-in. hollow concrete masonry units. A bond beam was provided on the front and rear walls, and the masonry units were filled with concrete for a few courses where girders or bar

FIGURE 28 Fairfield County Community Center in Winnsboro.

FIGURE 29 Detail of Fairfield County Community Center.

FIGURE 30 Gymnasium of Richard Winn Academy.

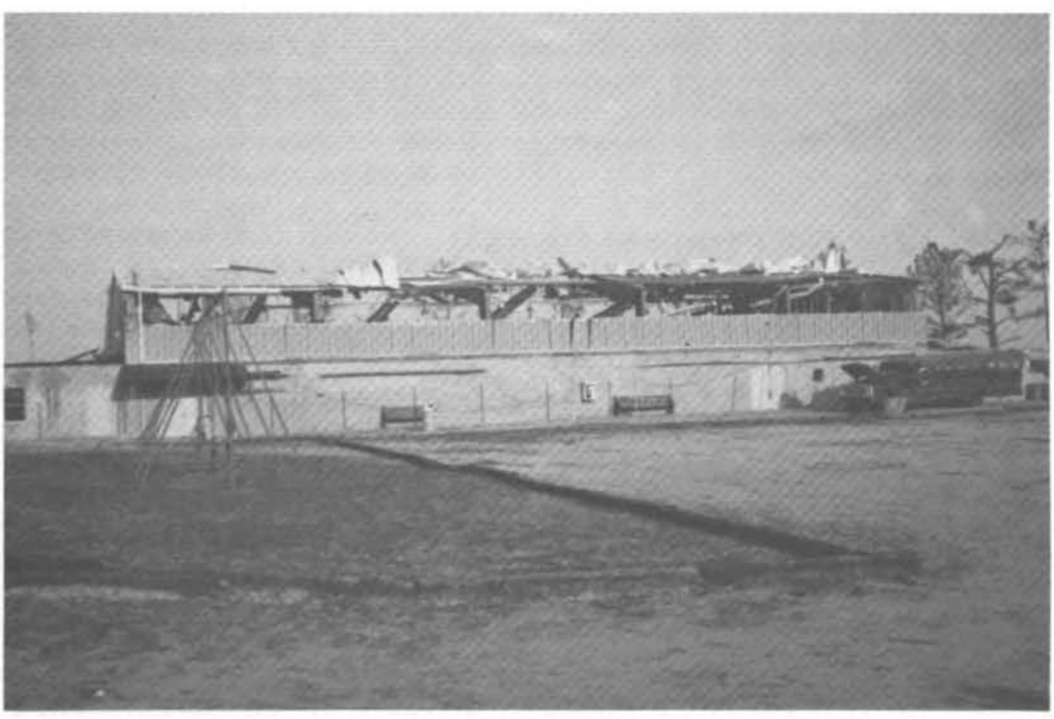


FIGURE 31 Steel-framed retail store in Winnsboro.

FIGURE 32 Steel-framed retail store in Bennettsville.

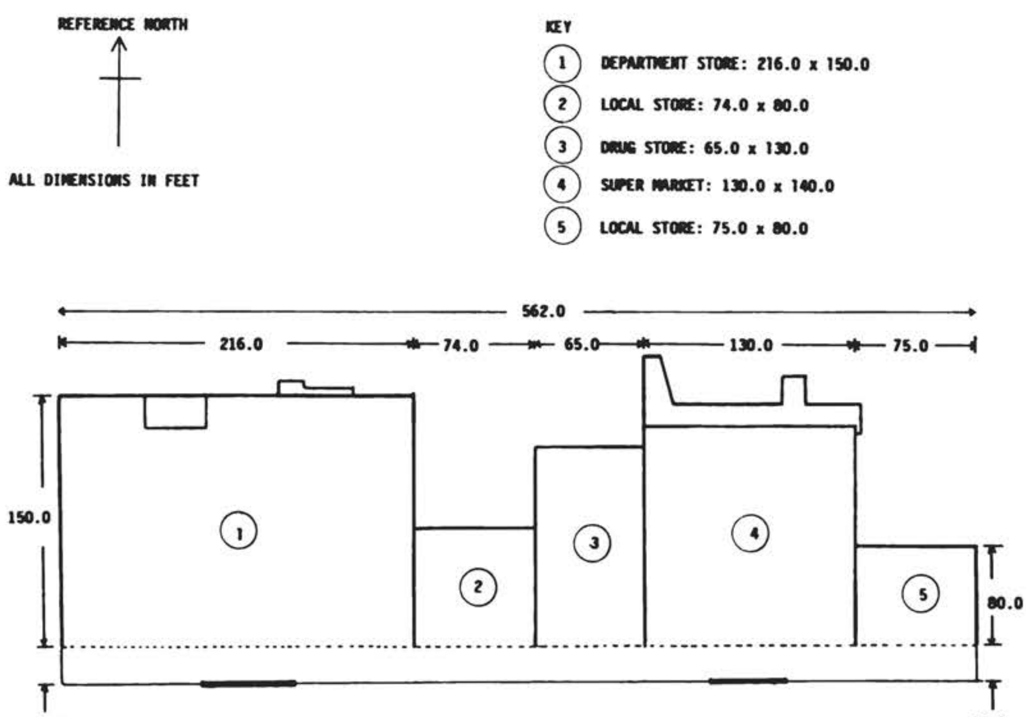


FIGURE 33 Plan of Northwood Village Shopping Center in Bennettsville.

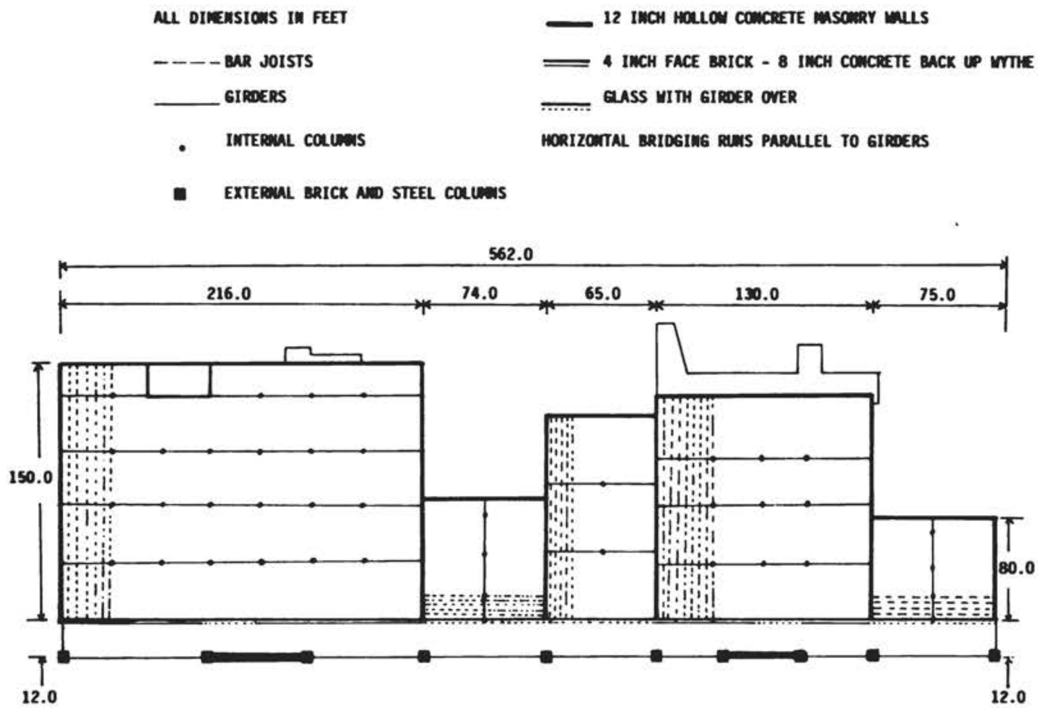


FIGURE 34 Framing plan of Northwood Village Shopping Center.

joists were supported by walls without bond beams. Control joints were provided in the walls at intervals of between 20 and 30 ft. Horizontal truss reinforcement was provided in alternate courses. No vertical reinforcement was provided. The walls ranged in height from 16 to 20 ft.

Figure 35 shows the general extent of the damage. The department store at the west end was reduced to a pile of twisted steel and masonry. The front wall had been blown in, the side and back walls had been blown out, and the roof framing system had collapsed.

Next to the department store, the small unoccupied local store lost all its supporting walls, but the center line of columns remained upright (Figure 36).

In the drug store the front and side walls collapsed, as did the front half of the roof (Figure 37). The girders had been joined at a point that would normally carry little moment. With the collapse of the side wall, this became a highly stressed area and the joint failed (Figure 38).

In the adjacent supermarket the front wall failed, and the resulting pressurization of the interior and the suction on the roof appeared to have lifted the first row of bar joists and wrapped them around the first line of girders (Figures 39 and 40).

The local hardware store at the extreme east end, not shown in Figure 35, suffered relatively minor damage. The front wall, consisting mainly of glass, failed, and some metal decking was removed from the roof, but the roof framing remained intact (Figure 41). This was also the only part of the shopping center where there was any trace of the

FIGURE 35 Aerial view of damage to Northwood Village Shopping Center. (Photograph courtesy The State.)

FIGURE 36 Surviving framing in local store.

FIGURE 37 Roof collapse in drug store.

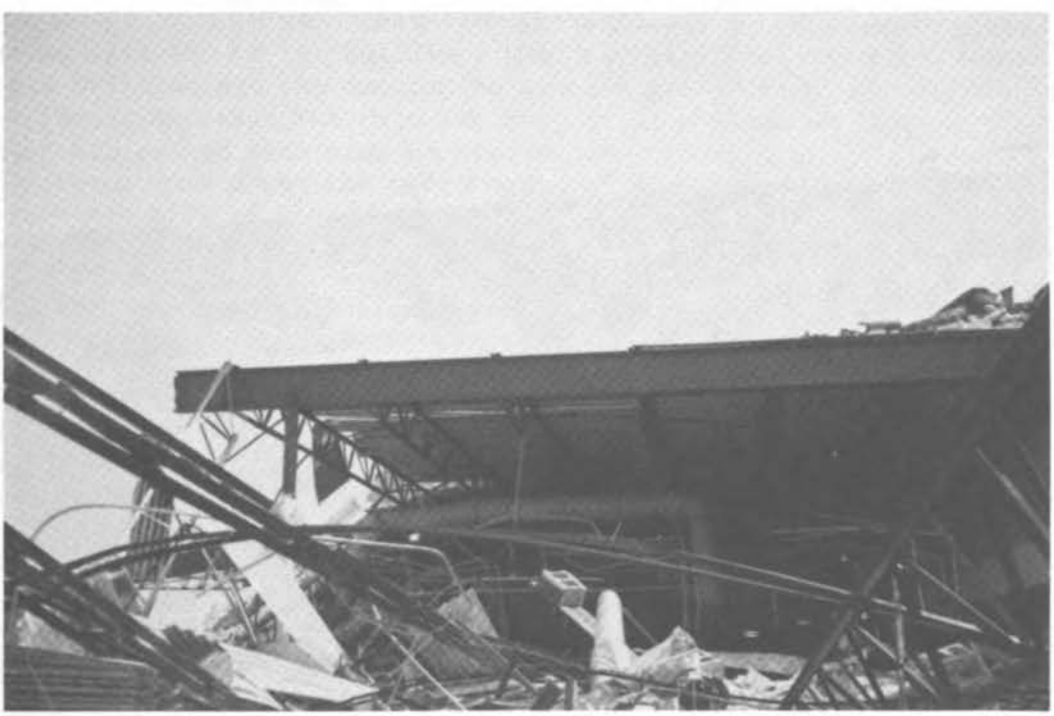


FIGURE 38 Failure of beam joint.

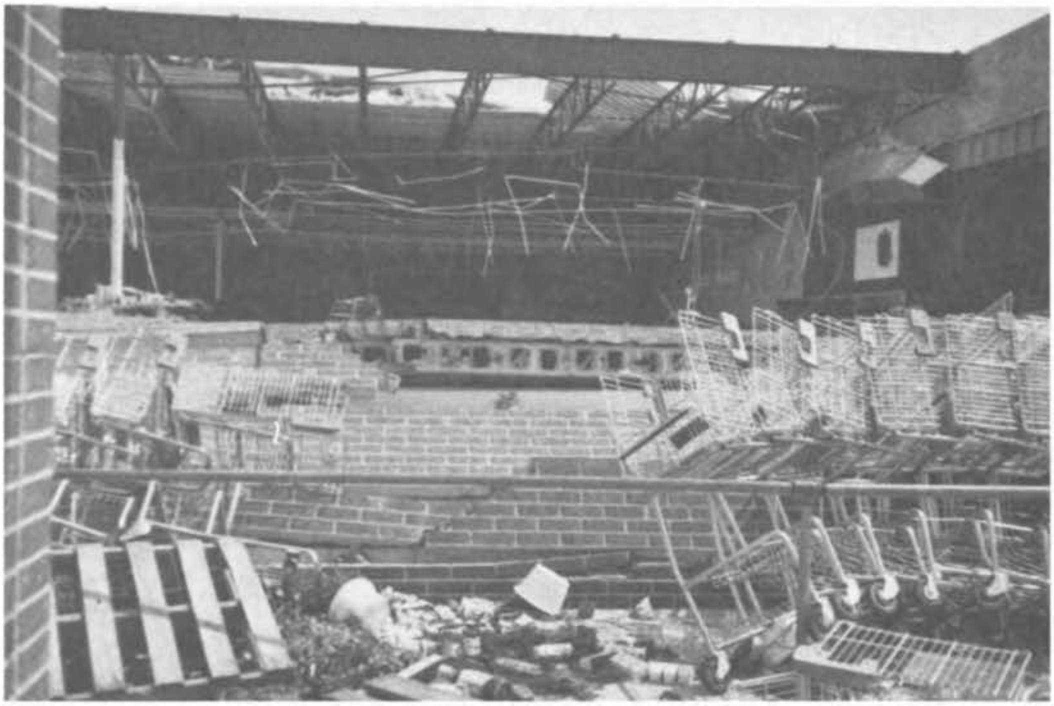


FIGURE 39 Failure of front wall in supermarket.

FIGURE 40 Failure of roof in supermarket.

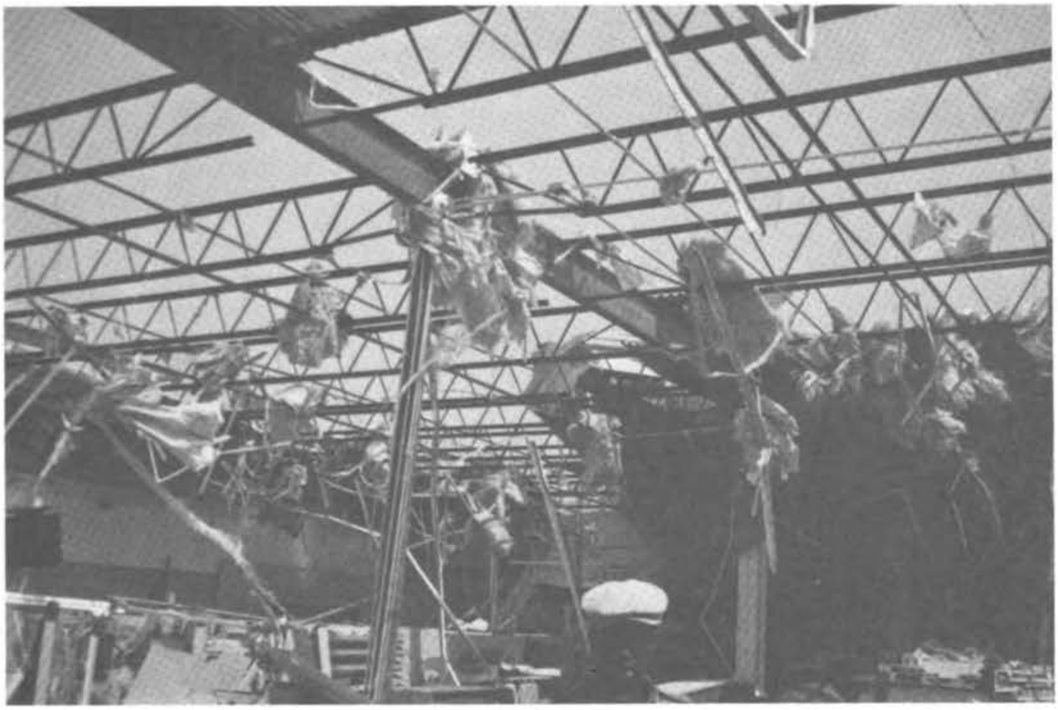


FIGURE 41 Surviving roof system in hardware store.

front canopy (Figure 42). Although the wind speeds were undoubtedly lower at this end of the building, the orientation of the roof members in this building and the unoccupied store differed from that of the three larger stores. The main girders ran from front to back in the smaller stores and from side to side in the larger ones.

To learn more about the performance of the structure, a detailed analysis was made of the department store (Desai, 1984). The results of this analysis are summarized below.

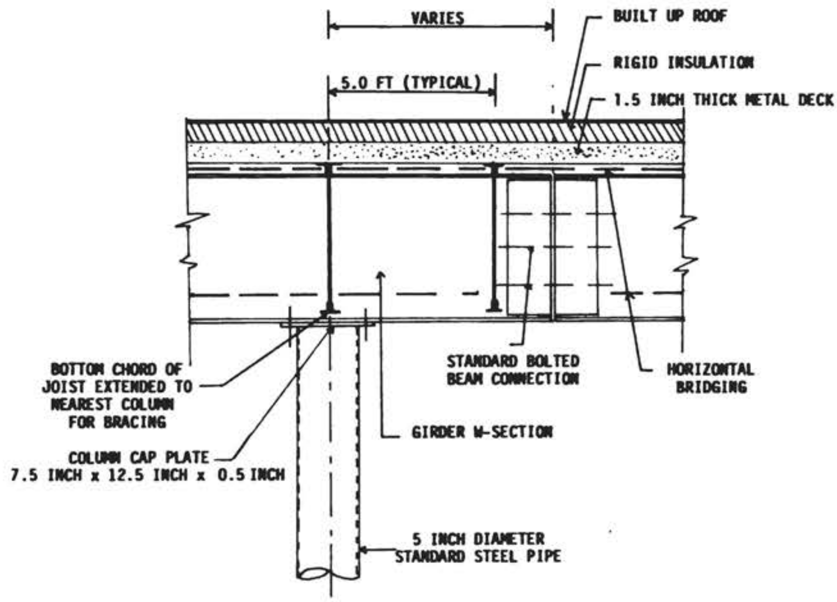
In checking the roof system, several points became evident. The bar joists were quite adequate for the design gravity loads provided the decking supplied lateral restraint to the compression flanges of the bar joists. The girders were capable of carrying the design gravity loads provided the bar joists supplied lateral restraint to the compression flange of the girders. These girders were apparently designed on the assumption of there being a pin connection between the girders and the columns. Figure 43 shows that this connection was so meager that this was a reasonable assumption. Indeed, the columns could only carry the required load if they were pin-ended and fully restrained against lateral movement. Although this restraint was initially provided by the bar joists and girders, it was ultimately provided by slender unreinforced masonry walls.

These masonry walls were also required to act in other ways. They had to carry vertical loads imposed by the bar joists and girders.

FIGURE 42 Surviving canopy at east end of the shopping center.

These loads would have been well distributed to the front and back walls, where bond beams were provided (Figures 44 and 45). On the side walls, where no bond beam existed, the load distribution would have been limited (Figure 46). In any case, the roof was extremely light and the vertical loads carried by the walls were small. This had a detrimental effect on the ability of the walls to perform another function, that of resisting the wind loads. In this respect, they had to be capable of transferring the load either directly to the ground or to the roof. Here the metal decking acting as a shear diaphragm would have transferred the loads to the side walls. Functioning as very deep shear walls, these walls would finally have transferred the rest of the wind load to the ground.

Despite the important role played by the walls, little attention appears to have been paid to their design. They were apparently sized to meet the absolute maximum ratio of unsupported distance to thickness permitted by the Standard Building Code in effect at the time of construction--18--with the roof being deemed to provide support. However, according to the code, if a roof is used to provide lateral support, the maximum horizontal distance between supports should not exceed 75 times the thickness of the wall, which in this instance would be 75 ft. The side wall of the department store stretched for 150 ft without support and contained several control joints. The front wall stretched for 216 ft.



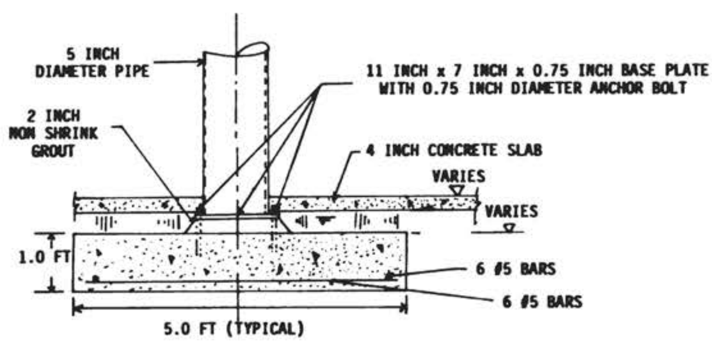


FIGURE 43 Typical beam-column connection.

The requirements concerning unsupported length are intended to avoid instability, but the walls should have been capable of resisting all the vertical and horizontal loads required by the code. An engineering analysis of the walls indicated that in all instances the critical combination of dead, live, and wind loads specified by the code exceeded the allowable strength of the wall. On the other hand, the tempered glass used in the front wall had a capacity far in excess of that required by the code. It is surprising, therefore, that eyewitnesses reported that the glass failed first, creating an opening sufficiently large that occupants and contents of the building were blown to the rear. Shortly after the glass failed, the roof sheeting was stripped off, followed almost immediately by the collapse of the walls and the roof framing system.

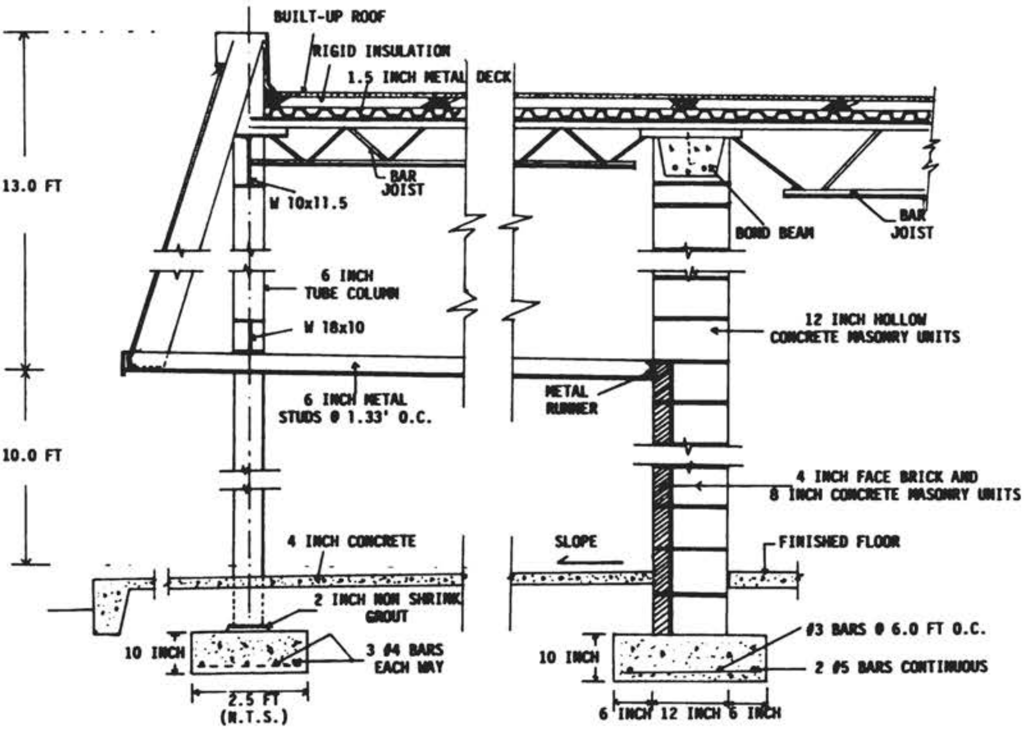


FIGURE 44 Detail of front wall (as designed).

Rescue operations had disturbed the collapsed front wall of the department store, but in other stores evidence was found that the glazing system, including the frame, had been stripped from the surrounding masonry. This indicated that failure may have been initiated by poor wall to window frame connections.

Another factor that may have contributed to the failure was the fact that the canopy had not been built as shown in Figure 44, but had been replaced by inferior wood framing as shown in Figure 42. Without the restraint and shelter provided by this canopy, the front wall of the department store possessed very little wind resistance.

To determine how the failure of certain components influenced the wind loads on the remaining components, a study was conducted in the structural engineering wind tunnel at Clemson University using a 1 to 288 scale model of the shopping center and a boundary layer appropriate for the local terrain (Wright, 1984). Table 1 presents the significant conclusions of this study, showing the wind velocities, assumed normal to the front face of the building, likely to cause failure of the components.

These speeds were determined from the mean net pressure coefficients obtained in the wind tunnel tests at the stage of damage indicated in the table. When the building was intact, no account was taken of a possible internal pressure in excess of the external ambient pressure.

TABLE 1 Wind Speeds (in Miles per Hour) Expected to Cause Failure of Components of the Department Store at the Northwood Village Shopping Center

Building Condition and Component	Assumed Factor of Safety Against Collapse 1	2	3
Building Undamaged			
Front wall with canopy	86 (79)	121 (112)	148 (137)
Front wall without canopy	89 (27)	126 (38)	154 (46)
Canopy	99 (--)	140 (--)	172 (--)
Side wall	147 (81)	207 (114)	254 (140)
Rear wall	150 (89)	213 (126)	261 (155)
Front third of roof	173 (--)	245 (--)	300 (--)
Rear two thirds of roof	131 (--)	186 (--)	227 (--)
Front Wall Collapsed			
Side wall	100 (55)	141 (78)	173 (95)
Rear wall	90 (54)	127 (76)	156 (93)
Front third of roof	76 (--)	107 (--)	131 (--)
Rear two thirds of roof	80 (--)	113 (--)	139 (--)
Roof Removed			
Side wall, top-supported	148 (82)	210 (116)	257 (142)
Side wall, free-standing	96 (57)	135 (80)	166 (98)
Rear wall, top-supported	74 (41)	105 (58)	127 (71)
Rear wall, free-standing	48 (28)	68 (40)	83 (49)

NOTE: The first figure in each entry refers to components designed to meet the pressures specified by the Standard Building Code with the factor of safety indicated. The second figure in parentheses is based on the allowable strength of the components as determined from the design drawings by Desai (1984).

One would normally expect a factor of safety against collapse of 3 in masonry construction and 2 in steel construction. With these factors of safety, components designed to meet the pressures specified in the Southern Standard Building Code should not have experienced serious damage until the wind speed reached nearly 150 mph. If at that stage the front wall had failed, failure of the roof sheeting would soon follow. This would probably have left the exterior side wall free standing, but the rear wall might still have received support from the bar joists. Irrespective of the type of support, the failure of the walls would be inevitable, and the subsequent collapse of the unrestrained girder and bar-joist system would be likely.

Regarding the failure of the actual structure, estimates could only

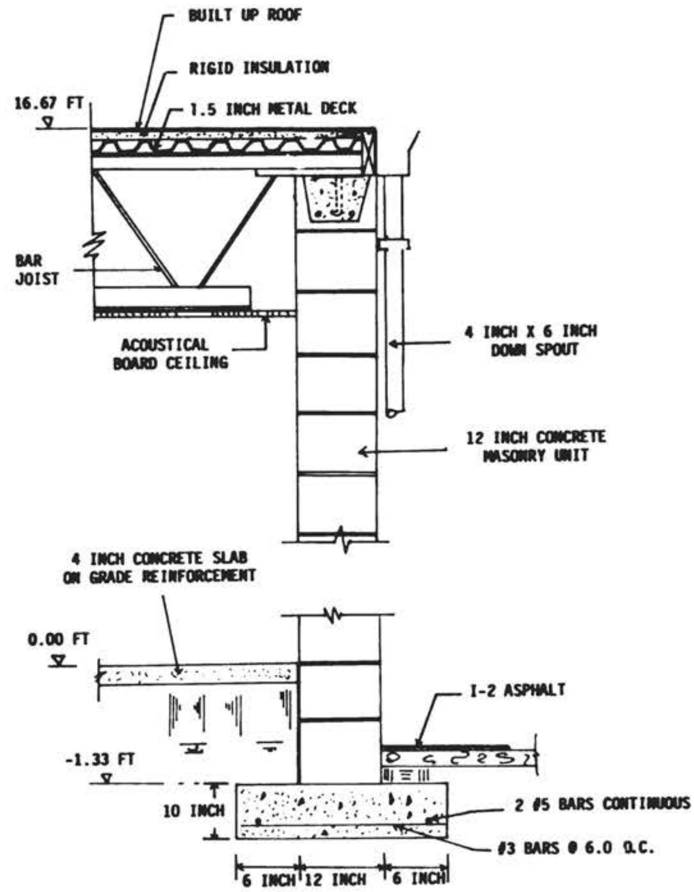


FIGURE 45 Detail of rear wall.

be made of the strength of the walls. No data were available concerning the ability of the roof sheeting to resist uplift forces or the wind resistance of the wood canopy. It is highly unlikely, however, that they would have met the code design requirements with a factor of safety of more than 2, and the roof sheeting is known to have failed before the walls.

Surprisingly, despite the inadequate design of the walls, if the canopy provided shelter and support to the front wall, and if the glazing system remained intact, the structure should not have been damaged until the wind reached nearly 140 mph. However, failure of the canopy or glazing system at any speed in excess of 100 mph would probably have caused the complete collapse of the structure. That is the basic weakness of this form of construction. In a conventional steel-framed structure, the failure of cladding elements or glazing will relieve the wind load but will not significantly reduce the capacity of the structural system to carry the loads. In this hybrid steel and masonry system, relief of the wind loads through failure of the roof is accompanied

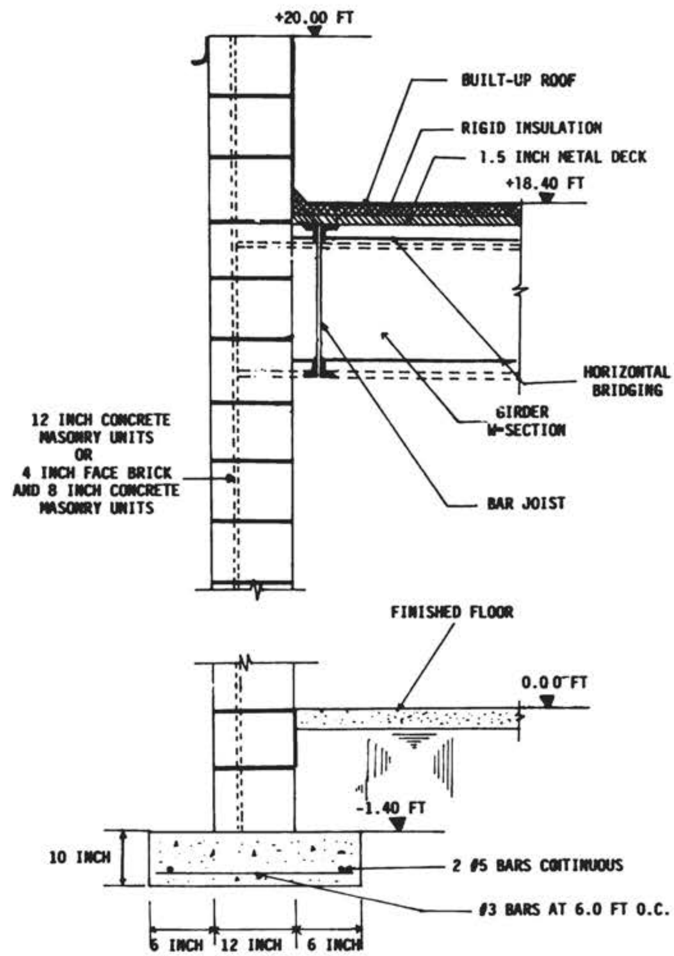


FIGURE 46 Detail of side wall.

by a loss of restraint to the walls and a consequent reduction in structural capacity far greater than the reduction in load. Since these walls also provide the lateral bracing for the vertical load-bearing system, the conditions for a progressive collapse are established as soon as the roof diaphragm fails. It is particularly disturbing that the diaphragm's integrity, and ultimately that of the whole structure, depends upon such minor details as the window frame to masonry connections, the design of the walkway canopy, and the welding of the metal decking to the bar joists.

5

GENERAL COMMENTS

It is generally assumed that it is not economical to design a structure that will suffer no damage if it is hit by a tornado except where damage to that structure would seriously endanger the population at large, as in the case of nuclear power plants. It is feasible and desirable, however, to design public buildings that will not collapse in a tornado and will thus provide a safe refuge for the public. This is particularly true in hurricane-prone areas, where such buildings may be used as emergency evacuation centers in weather conditions that often generate tornadoes.

The performance of the public, unreinforced masonry buildings examined in this survey was extremely poor. South Carolina has the highest earthquake risk of any state on the East Coast and is subject to hurricanes, so it is surprising that unreinforced masonry is used so extensively in the area when its poor performance in extreme winds and earthquakes has long been known.

Since the cost of reinforcing masonry is small compared with the overall construction costs and the returns are so great, there seems little excuse for the continued use of unreinforced masonry in places of public assembly.

The collapse of the Northwood Village Shopping Center and the subsequent testing and analysis highlighted serious shortcomings in its form of construction. Unfortunately, it is a form of construction used in hundreds of shopping centers in the region. The use of a moment-resisting frame would add relatively little to the total cost yet would increase the safety of such buildings significantly. The better performance of buildings constructed in this way clearly shows the advantage to be gained from this form of construction.

With regard to domestic dwellings, a number of simple improvements could be made to conventional construction that, although they would not prevent minor damage in the event of a tornado, would probably prevent collapse and loss of life. In September 1984 the author conducted a similar damage survey in North Carolina following Hurricane Diana (Mitchell et al., in press). North Carolina has a statewide residential building code (North Carolina Building Code Council, 1968) that contains specific requirements for improving the wind resistance of domestic dwellings. These requirements were developed following a series of devastating hurricanes in the 1950s. At present they apply only to coastal areas of the state. For timber-frame construction the code requires that rafters and roof trusses be secured to the framing by metal clips and that the frames be tied to the foundations. This may be achieved by 3/8-in.-diameter rods, not more than 8 ft apart, running from the top of the frames to the foundation. A similar requirement exists for masonry buildings.

In Hurricane Diana the wind speeds are thought to have been on the order of 100 mph. Buildings built to the building code appeared to have performed well. Many buildings predating the code lost all of their roofs, although the walls remained standing. Several examples of this type of failure were also found in the paths of the South Carolina tornadoes (Figure 47). These buildings would probably have suffered very little damage had they possessed the connections between roof and foundation required by the North Carolina Uniform Residential Building Code. Indeed, there was a strong similarity between the nature of the damage to poorly connected buildings caused by the South Carolina tornadoes and that caused by Hurricane Diana and Hurricane Alicia (which was of similar strength; see Savage et al., 1984)). This similarity suggests that the wind speeds in these tornadoes may not have been exceptionally high in many instances and that relatively minor changes in building techniques could significantly reduce the risk of damage and loss of life. Based on the experience in North Carolina, the addition of ties and clips adds relatively little to the overall cost of a building.

Unanchored mobile homes are clearly dangerous structures in tornadoes, but the performance of at least one mobile home in this survey suggests that while conventional tie-down techniques may give good protection in normal storms they do not render a mobile home a safe location in a tornado.

The nature of these tornadoes was very unusual with respect to their dimensions and rate of travel. The large diameter of the disturbances probably accounts for the nature of the damage observed. For example, at the Northwood Village Shopping Center all of the debris blew in one direction, which is consistent with the shopping center being to the east of the center of the tornado. Since the tornado was reportedly traveling in a northeasterly direction at about 60 mph, the highest wind speeds would have been to the right of its center, where the forward motion would increase the windspeed caused by the circulation. No serious structural damage was observed in the area where the debris was blown in a direction opposite to the forward motion of the tornado.

The path width was significant in other ways. Tornadoes in this area usually have path widths of a few hundred feet and are relatively slow moving. Had this been the case at Bennettsville, it would not have been appropriate to have studied the department store in a conventional wind tunnel. The width of the store, 200 ft, would have been a substantial proportion of the width of the tornado, and the wind regime would have differed appreciably from that modeled in the wind tunnel. Instead, the Bennettsville tornado was over 6,000 ft in diameter and possessed a high forward velocity. Under these circumstances there would be little variation of wind speed and direction across the building, and conditions should have been similar to those created in the wind tunnel.

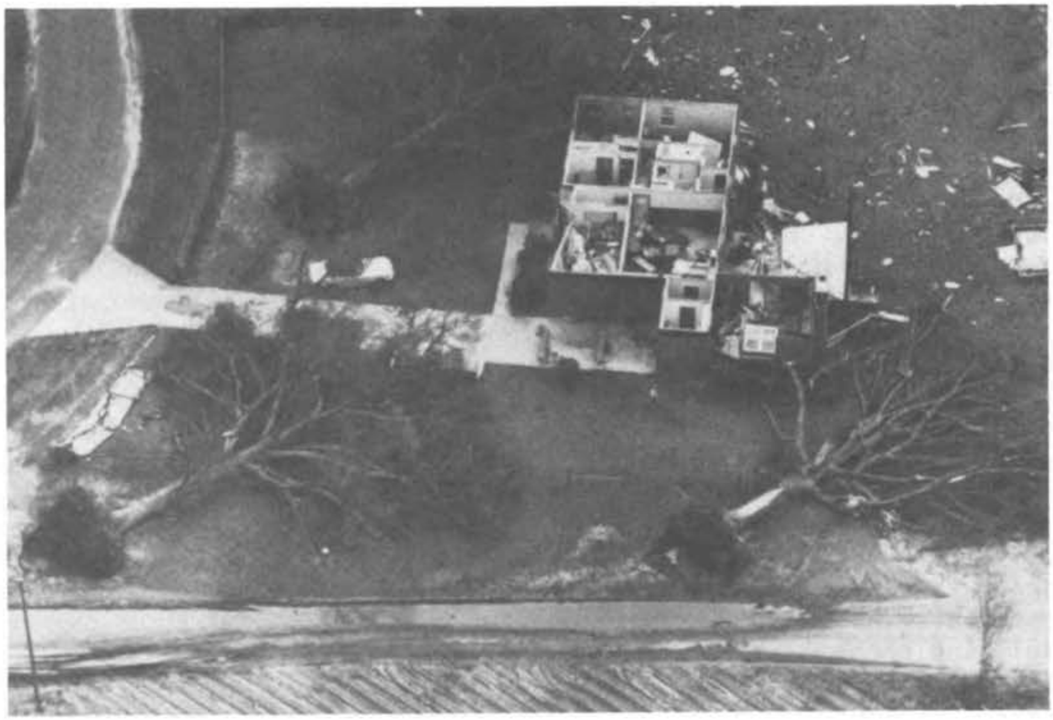


FIGURE 47 Typical roof failure caused by the South Carolina tornadoes. (Photograph courtesy The State.)

As a consequence of the unusually wide path widths, direct comparisons could also be made between the performance of buildings in this storm and buildings subjected to recent hurricanes. Care should be taken, however, in applying these conclusions to large buildings subjected to tornadoes with small diameters.

Although South Carolina is not in one of the well-known tornado areas, there have in fact been 33 killer tornadoes in the state since recordkeeping began in 1912. In one outbreak in 1924, 77 people were killed. All three towns in this survey had been hit by tornadoes before. In many respects the state appears to have been well prepared to cope with tornadoes. The National Weather Service had established a system of weather spotters and for several years had held tornado awareness weeks prior to each tornado season. One was held just four weeks before this outbreak. Some towns had siren systems to warn of tornadoes, and schools conducted regular tornado drills. When the outbreak occurred, the National Weather Service issued timely advisories and warnings. The emergency services appeared to have functioned well, and the utility companies restored services in a reasonable amount of time. Further information on the response of the National Weather Service and the community to the storm can be found in National Oceanic and Atmospheric Administraion (1984).

The Building Officials Association of South Carolina has made repeated efforts to get the state legislature to mandate the adoption of building codes and the certification of inspectors. Unfortunately, the state government has maintained a laissez-faire attitude toward building control. This, combined with the relatively low income level and generally gentle climate, has tended to encourage the construction of unsuitable buildings. Except on the coast, windy days are a rarity. The relatively high design wind speeds are due to the passage of earlier hurricanes and severe thunderstorms. At the time of these tornadoes, there had not been a severe hurricane in the area for more than 20 years, and nobody had been killed in a tornado for over 10 years. In the absence of strong building control, methods of construction have apparently developed that are unable to withstand the extreme wind conditions that occasionally occur in the area.

Unlike hurricanes, tornadoes rarely give people enough time to choose where they will take shelter, and places of public assembly must be designed so that they do not collapse when hit by a tornado. The public must be made aware that buildings can resist tornadoes, and the building profession must be made aware of how this can be done.

Better trained engineers and building inspectors, and perhaps a change in the attitudes of insurers and building financers, could bring about the necessary improvements in public buildings. Foremost among these should be the discontinuation of the use of unreinforced masonry in a structural system.

For domestic dwellings and other nonengineered low-rise structures, local or state governments must adopt a more positive attitude toward the adoption and enforcement of specific and easily understood building code requirements concerning wind resistance. This process should be greatly facilitated by the recent introduction of a standard for walls in hurricane-force winds (Southern Building Code Congress International, 1984). Designs meeting the requirements of this standard are deemed to satisfy the wind loading provisions of the Standard Building Code for walls, nominally 8 ft high, constructed of wood stud, brick, or concrete masonry units. The standard is intended to be used in areas where the design wind speed, based on a 100-year recurrence interval, exceeds 80 mph. This includes all of South Carolina. Use of this standard would significantly improve the ability of future low-rise structures to resist this type of event.

For added protection of the occupants, the practice of constructing basements should be encouraged where conditions permit. Where underground construction is difficult, the provision of a reinforced interior room could be an acceptable alternative.

6

POSTSCRIPT

Five months after the tornadoes of March 28, 1984, in South Carolina, the area was visited again. The dance academy and auto parts store in Newberry has not been rebuilt, but nearly all of the other damage in the town had been repaired.

In Winnsboro several houses were still in ruins, although many had been repaired. Surprisingly, the steel-framed Fairfield County Community Center had been dismantled and was being rebuilt from scratch. This was also true of the gymnasium of the Richard Winn Academy. Indeed, the school had been completely demolished, including an undamaged wing. The new construction was of unreinforced masonry. A much expanded church was being built on the site of the one demolished in the storm. Again the construction was of unreinforced masonry.

The greatest economic effect of the tornadoes was probably felt in Bennettsville. No attempt had been made to rebuild the Northwood Village Shopping Center. The only activity on the site was in a temporary building erected by the drug store. The community had therefore been deprived of a major shopping facility for a considerable period of time. There were reports that even if the shopping center were rebuilt, the department store, part of a major chain, would not reopen. The apartment buildings damaged had been rebuilt, and more had been added to the area. Most private homes had also been repaired.

When Hurricane Diana made landfall on September 13, 1984, Bennettsville and Marlboro County came under a tornado advisory. This created considerable alarm among some residents. At their request, four local high schools were opened as emergency shelters, and over 100 people took shelter. Fortunately, no tornadoes were reported.

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APPENDIX:

BIOGRAPHICAL SKETCH OF THE AUTHOR

Peter R. Sparks is Associate Professor of Civil Engineering and Engineering Mechanics at Clemson University. He received a B.Sc. degree in Civil Engineering from Bristol University in 1968 and a Ph.D. in Structural Engineering from London University in 1974. From 1968 to 1977 he was a Scientific Officer at the United Kingdom Building Research Establishment, where he was responsible for research on the behavior of concrete under fluctuating loads and on the experimental determination of dynamic characteristics of large structures. In 1977 he moved to the Department of Engineering Science and Mechanics at Virginia Polytechnic Institute and State University, first as a Visiting Professor and then as Associate Professor. His research there was concerned mainly with the determination of wind loads on structures both from wind tunnel tests and full-scale measurements. He took up his present position in 1982. His current areas of interest are structural performance under wind and earthquake loads, structural system identification from vibration measurements, and the development of structural systems to resist natural hazards.

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