The Load PATH

A roof's chain of strength is only as good as its weakest link.

by Richard C. Schroter

Most wind damage to roofing occurs at relatively low wind speeds. Insurance industry statistics for the last 11 years show that while hurricane and tornado winds produce almost 75 percent of the dollar losses paid out by insurers, these storms represent only 27 percent of roof damage claims. Architects, engineers, and contractors outside traditional high-wind areas also need to pay heed to the lessons of hurricanes Andrew, Hugo, and Iniki. All projects require the same attention to detail regardless of the magnitude of the wind loads.

Quality assurance in the load path—from roof panel to ground—is the key to sound structural performance of all roofing systems. Since several different materials and systems are usually involved in the load path, coordinating design and installation is mandatory. Like a chain, the system is only as good as its weakest link. ◆ Until recently, there was no universally accepted test for evaluating the durability of connections using spaced fasteners to anchor thin sheet or membranes. Specific procedures are necessary because of local stresses and varying

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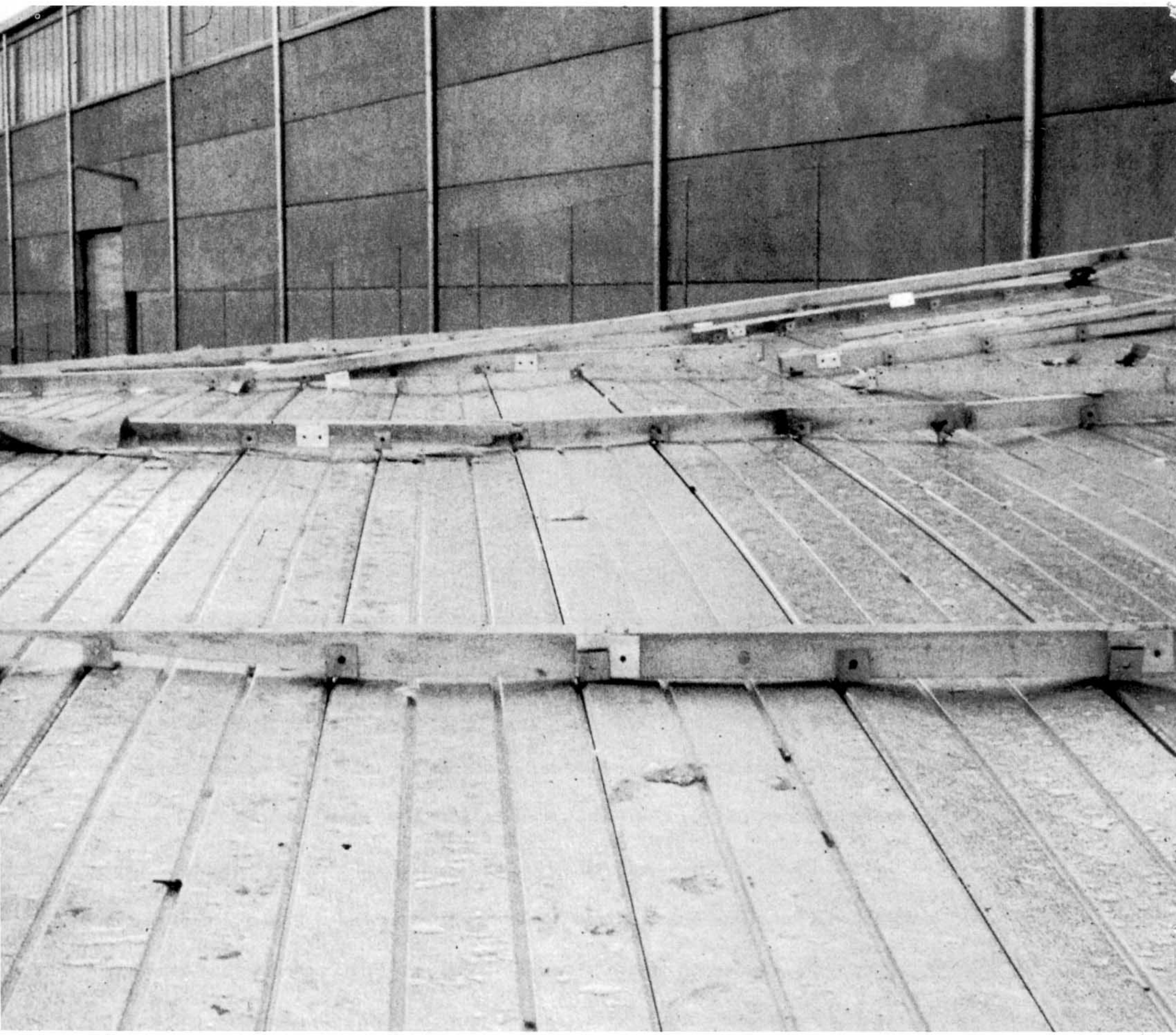

material behavior as the assembly changes shape from air pressure For metal roofing, that gap was filled earlier this year when ASTM test procedure E1592 was released.

In the case of metal roofing, the load path may involve panels, clips, clip anchor screws, purlins, purlin fasteners, the secondary structure, structural fasteners, the primary structure and its connections, and the foundation. Successive links of the structural chain are usually clearly defined, factory-made parts whose quality is not dependant on field weather conditions as are other materials. Loads on individual components can be calculated and compared with allowable loads. However, understanding how the parts work together and whether there are secondary stresses as a result of eccentric loads or prying forces is important.

Photo 1.

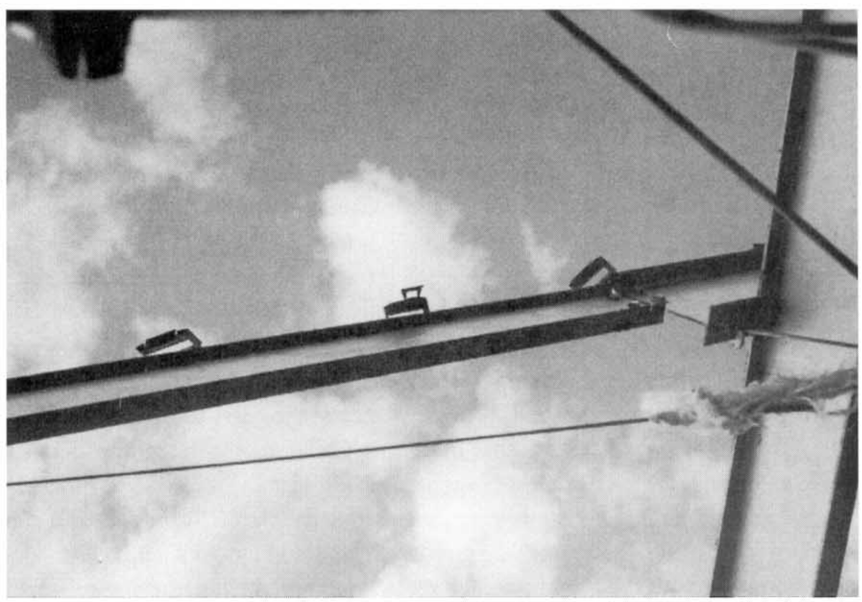

Photo 2.

Putting It All Together

The engineering process can be complicated if several subcontractors and types of materials are involved and if portions of the engineering design are carried out by different entities. Building system manufacturers stress the value of having a single source for engineering design responsibility. In exchange for this convenience, A/Es will probably lose control of some aesthetic items and, unless they are specific in setting the project design conditions, possibly some of the performance criteria.

For example, do not express the design criteria in terms of "to meet the local code" or in terms of wind speed alone. Both of these leave the exposure factor up to the bidder. There can be different interpretations of the applicable wind speed. It is better to express design loads in terms of pounds per square foot (Pascals).

In reroofing, the existing structure cannot be assumed adequate for support of the new work. Where the contract documents do not require a specific span capability, bidders may propose more efficient overall construction based on systems with different span capabilities than originally considered. Contract documents should allow for additional engineering and reinforcement of the existing structure. The design can be based on support spacings verified by the project engineer, with the cost of engineering

Photo 3.

and reinforcement of any other system to be included in the submittal.

Another concern is the load path revision from the original system to the new roof. Distributed loads concentrated at different locations can be disastrous when reroofing includes adding a secondary framing system for slope. Even without additional slope, the mere fact that a typical metal roofing system spans between widely spaced supports can change the way the structure beneath carries the load.

The engineer must verify that there is an adequate "platform" to support and anchor the new roof, ensuring that the load path of the retrofit system is compatible with the existing structure.

Causes

Structural failure of metal roofing is primarily caused by wind uplift. Each member and its connections are under load. Seams in metal roofing are often locked together so that panels stay joined after individual clips or rows of clips have broken. Once a local failure occurs during high uplift, adjacent connections must support ever increasing areas, allowing the failure to progress at much lower pressures.

Inadequate shear transfer by the roof of wind loads on one wall to the crosswise perpendicular walls also can cause damage. Because many metal roof systems are designed to expand and contract relative to the structure, they are limited in their capacity to act as diaphragms. Alternate framing such as diagonal tie rods or wind trusses may be needed to carry these loads.

Collapse of metal roofing under snow loads usually results from inadequate anticipation of loading in the design. The cladding, roof clips, and connections are seldom a principal factor in positive load failures. However, sliding clips do not provide as much lateral support for the top flanges of purlins as do fasteners through the cladding. If this is overlooked, purlins may buckle at mid-span from positive load.

The Load Path

Panels. Deformation from air pressure alters the section properties and structural characteristics of metal panels. Deformation from positive pressure tends to increase section properties while negative pressure initially reduces bending strength and can have an adverse affect on the clip connection. In the 1982 edition of *Specifications for Aluminum Structures*, the aluminum industry recognized this behavior, calling for air-pressure testing of all systems secured with clips. This standard lacks a detailed explanation of appropriate test procedure, however. It merely requires that air bags or tape "not bridge any joints or elements that will tend to spread laterally under load."

Many seams are vulnerable to spreading and then separating from the clips. For years, a variety of tests have been used, some more realistic than others. ASTM E1592 is designed for all types of sheet metal panels.

In *Photos 1* and *2*, taken in Dade County after hurricane Andrew, the clip-to-panel connection of a concealed-fastener SSR system proved to be the weakest link. *Photo 1* shows the walls and frame battered but still secure, with roof panel clips and tabs in place. Only the cladding has been blown away. Clips and tabs are clearly evident on the purlins in these photos.

Photo 3 shows a similar situation in Detroit, Michigan. The one-piece clips show little distress after the roof cladding failed at the gable end and just peeled away.

Photos 4 and *5* illustrate opposite sides of the same end of a project along the Alaskan coast, where high wind gusts are common. Most of the roof cladding on one slope blew away, making analysis of the failure difficult.

On the opposite slope, the roof is largely intact. One seam is ruptured for

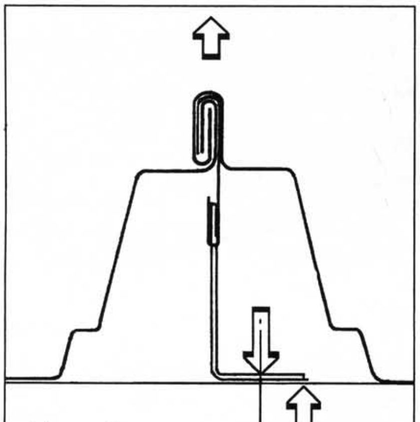

Figure A.

Photo 4.

Photo 5.

about 4.5 m (15 ft) near the middle of the 12 m (40 ft) long panel along the windward gable end, well within the high uplift pressure zone. Note that the eave and ridge ends are still intact. These ends are more resistant to this type of failure. The end closure conditions tend to flatten the panels and prevent the upward bowing of the pan that allows the ribs to spread apart. This suggests that the failure of the opposite slope probably originated at a seam.

The type of failures illustrated in these five photos will not show up in computations or in tests of insufficient length panels. As *Photo 5* suggests, end effects can extend 2.4 m or 3 m (8 ft or 10 ft) from an end condition. In ASTM E1592, the minimum specimen length for panels having both ends stiffened is 7.3 m (24 ft).

Of the major insurance listing agencies, Factory Mutual Research is the first to develop the capability to test specimens up to 7.3 m long. The UL 580 test specimens are 10 ft (3.05 m) long.

Clips. Anchors are designed to resist uplift yet allow the panels to slide relative to the structure to accommodate thermal expansion and contraction. Evaluation of the strength of the attachment to the panel requires a test of the assembly as described above. However, strength of the clip itself can be best evaluated in individual pull tests. Multiple setups must be used to simulate the actual load path of the installed system throughout extremes of thermal expansion and contraction.

Crosswise and lengthwise leverage can occur with two-piece clips because the hook can travel several inches. With thin sheet metal parts, test assemblies must also represent the minimum fastener heads to be used to ensure that deformation or tearing around the screw heads does not limit the capacity.

Clip fasteners. The tensile load on the clip fastener is usually higher than the uplift on the clip itself. A clip anchor screw is seldom in line with the center line of the engagement with the panel. An offset of only 20 mm (¾ in.) is enough to double the initial tension on the fasteners. *Figure A* shows how a rigid angle clip base doubles the tension in the screw compared to the uplift force from the roof.

Photo 6, page 72, shows local distortion of screw holes in purlins from clips rotating from the off-center load. Torque tests of the fasteners showed that thread failure had already started. The screws were on the verge of failure after the system was exposed to slightly less than the design pressure.

This view was taken from static air-pressure testing of components used in an installation where failed fasteners were found throughout the roof. Tests confirmed that clip leverage could double the anticipated fastener load. With two-part clips there are similar prying forces from hook or tab displacement relative to the centerline of the purlin as the roof experiences temperature extremes.

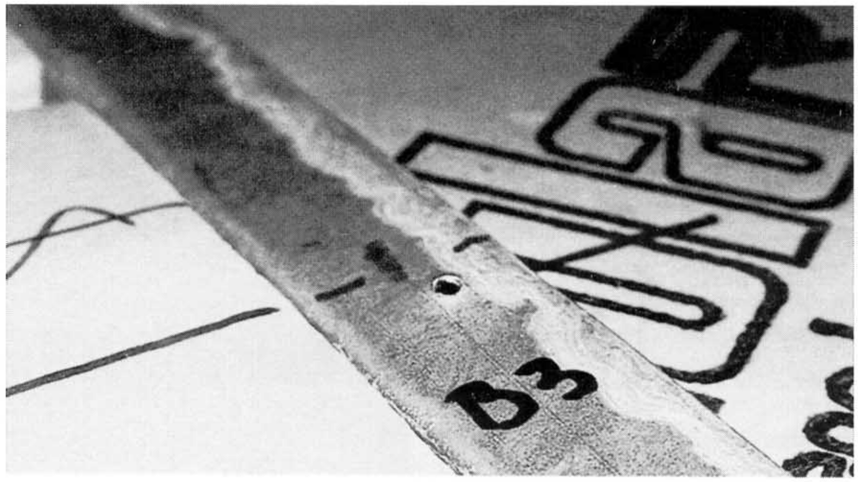

Photo 6.


Photo 7.

The usual safety factor for steel connections is 2.5. With the allowable one-third stress increase for wind loads, it becomes 1.875. If a design overlooks leverage, the safety factor decreases to 0.94. If the design also overlooks the variations in thickness and tensile strength of mill standards, the safety factor can be even less.

Standard industry tests for fastener pullout from various materials are usually available from manufacturers. Vendor data represents average ultimate values and should be adjusted for actual thickness and strength of the test specimen as well as by a suitable safety factor.

The American Iron and Steel Institute's (AISI) *Cold-Formed Steel Design Manual* safety factors are based on the actual thickness of the delivered material being within 95 percent of the thickness used in calculations. This may not be enough, however. Material adjustments are needed because cold-formed steel thickness can vary up to 15 percent for 16 gauge and 22 percent for 24 gauge. Thickness variations are even higher for galvanized steel sheet.

Actually, it is time for the construction industry to stop using gauge numbers for steel thickness. Current mill practice is that material is ordered and delivered based on decimal thickness. Since production tolerances are significantly closer than they were when the standard gauge ranges were established, the opportunity for misinterpretation or abuse is high.

Product manufacturers order material to the low side of the tolerance. Until the entire industry uses decimal thicknesses, engineers should design for minimum thickness whenever a gauge number is used.

When using fastener manufacturers' pullout data, request information on the actual tensile strength and thickness of the material used in the tests for comparison with what is expected for the project.

Screws are usually stronger than the members receiving the point. Pullout from wood typically varies with the size of any pilot hole, length of embedment, and with the density (species group, not structural grade) of the lumber used. Pullout from sheet metal varies with the size of the pilot hole, thickness, and the yield strength of the metal. Self-drilling screws intended for metal can be very weak in wood, and those intended for thick steel are weak in thinner sheet.

The capacity of field-driven fasteners can be degraded by improper driving tools. Impact wrenches, which can easily strip threads in wood and light-gauge metals, are not acceptable for use with most cladding work.

Purlins. The bending capacity of purlins to resist uplift and down loads is usually not a source of building failure if members are laterally braced to resist twisting or buckling of the compression flange. Purlins must be engineered to resist secondary lateral forces parallel to the top flange, such as the slope component of gravity or snow and forces from friction between clips and panels during temperature changes.

Purlin anchorage. When purlins are made of wood, connections may be overlooked in the design or assigned an inadequate factor of safety. In *Photo 7*, page 72, purlin anchors were two 10d nails. Spaced in a grid at 1.3 m (4 ft 3½ in.) in each direction, they supported 1.71 m^2 (18.4 ft^2). Nails are not recommended for use in pullout. Based on design values in the *National Design Specification for Wood Construction for Hem-Fir*, the allowable uplift would be less than 0.2 kPa (4 psf). Since the ultimate capacity is less than 1.0 kPa (20 psf), it is understandable that the project did not survive its first winter.

The photo on page 66 illustrates another case of inadequate purlin anchorage on a new roof. The entire metal roof with clips attached to wood purlins lies upside down next to the building. Clearly, insufficient attention was paid to the attachment of the wood to the underlying structure. Where different materials and trades are involved, in portions of the load path, the A/E must anticipate that different subcontractors will be used. Care must be taken during the submittal process in evaluating any changes involving connection details.

Photo 8.

Photo 9.

Reroofing has its own considerations, as the damage to an Alaskan hangar shown in *Photos 8* and *9* illustrates. During an energy-saving retrofit, two layers of insulation and a new metal roof were installed over an existing 40-year-old, 2 x 6 tongue-and-groove wood deck. The original corrugated metal cladding was secured on close centers to the deck. The structural design was primarily based on snow loads. Wind uplift was not a major consideration. The actual stresses were spread more or less evenly to the deck and then to the steel purlins below. The structure is uniform throughout. At the time of the original design, no code provision existed for higher loads at perimeters or corners.

The retrofit consisted of two layers of rigid insulation between wood members at right angles to each other to produce staggered joints. This type of assembly avoids through-cracks that would compromise the insulation's effectiveness. Purlins in the top layer run crosswise to the slope, providing clip anchorage at 1.3 m centers. These are secured to sleepers that run parallel to the slope and are secured to the deck with hat-shaped straps at 0.6 m (2 ft) centers. While maintaining the snow load capacity, this configuration effectively reduced the uplift capacity by a factor of four because the sleeper anchors omit six of the eight planks in each section of deck.

In two storms approximately one year apart, about 6 m (20 ft) wide portions of the cladding ripped away from the corners of the structure, pulling out a pair of 2 x 6s every 1.2 m (4 ft) or so. The missing deck sections clearly illustrate where the load path was weakest. The location of the failures also supports the importance of accounting for higher uplift in corner zones.

An example of a weakness further down the structural chain occurred in a project near Stuttgart, Germany (see *Photos 10* and *11*). The building's roof panels and clips were secured to timber purlins over a corrugated metal deck. The complex load path was thoroughly engineered. An air pressure-certified metal roof system was secured to wood purlins attached to metal decking over concrete beams. A wind gust blew off part of the roof, removing the roof covering, purlins, insulation, and metal deck. In some cases, these layers are still attached to precast concrete beams approximately 0.4 m (16 in.) deep by 0.15 m (6 in.) wide weighing about 220 kg/m (150 lbs per lineal foot).

The failure appears to have initiated at either the connection of the corrugated deck to the concrete beams or of those beams to the primary structural support. It was estimated that the actual force required to produce this damage exceeded the design standards by a factor of five.

No supplier likes to talk about wind damage, so investigation results are published only after catastrophes. Unfortunately, lessons from high-wind storms are not given enough attention. ◆

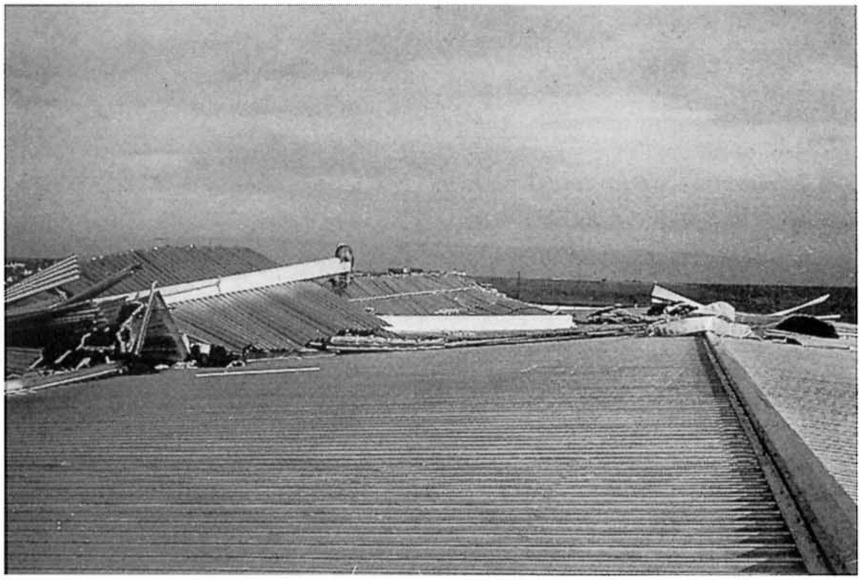

Photo 10.

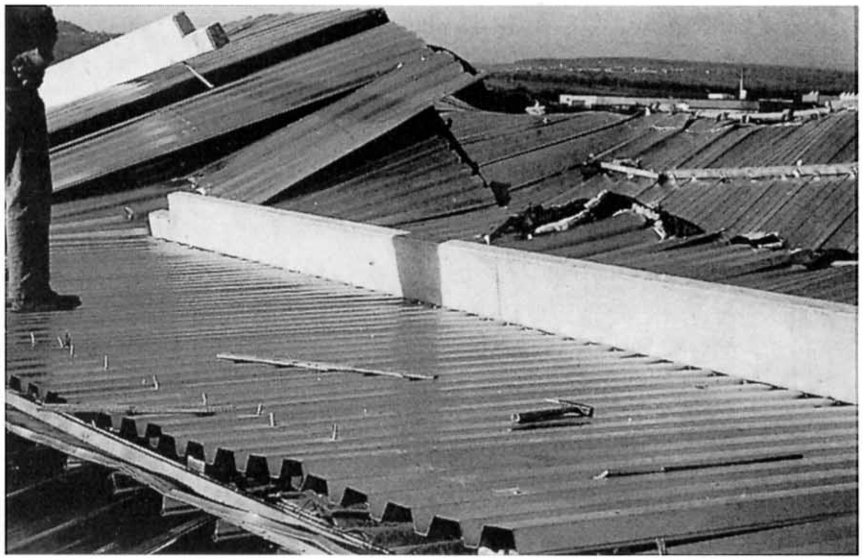

Photo 11.

References

ASTM E1592 Standard Test Method for Structural Performance of Sheet Metal Roof and Siding Sytems by Uniform Static Air Pressure Difference. American Society for Testing and Materials, 1916 Race Street, Philadelphia, Pennsylvania 19103.

Cold-Formed Steel Design Manual American Iron and Steel Institute, 1101 17th Street, N.W., Washington, D.C. 10036-4700.

Design Specifications for Stress Grade Lumber and its Fastenings. American Forest and Paper Products Association. 1111 19th Street N.W., Suite 800, Washington, D.C. 20036.

Specifications for Aluminum Structures. Aluminum Association, 900 19th Street N.W., Washington, D.C. 10006.

Technical Note E830B Fastener Loads for Plywood. American Plywood Association, P.O. Box 11700, Tacoma, Washington 98411.

UL 580 Tests for Wind-Uplift Resistance of Roof Assemblies. UL, Inc., 333 Pfingsten Road, Northbrook, Illinois 60062.

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