Hail Events and Structural Decking

– A CASE STUDY –

By Dziugas Reneckis, PhD, PE, MIStructE; and Remo Capolino, PE, RRC

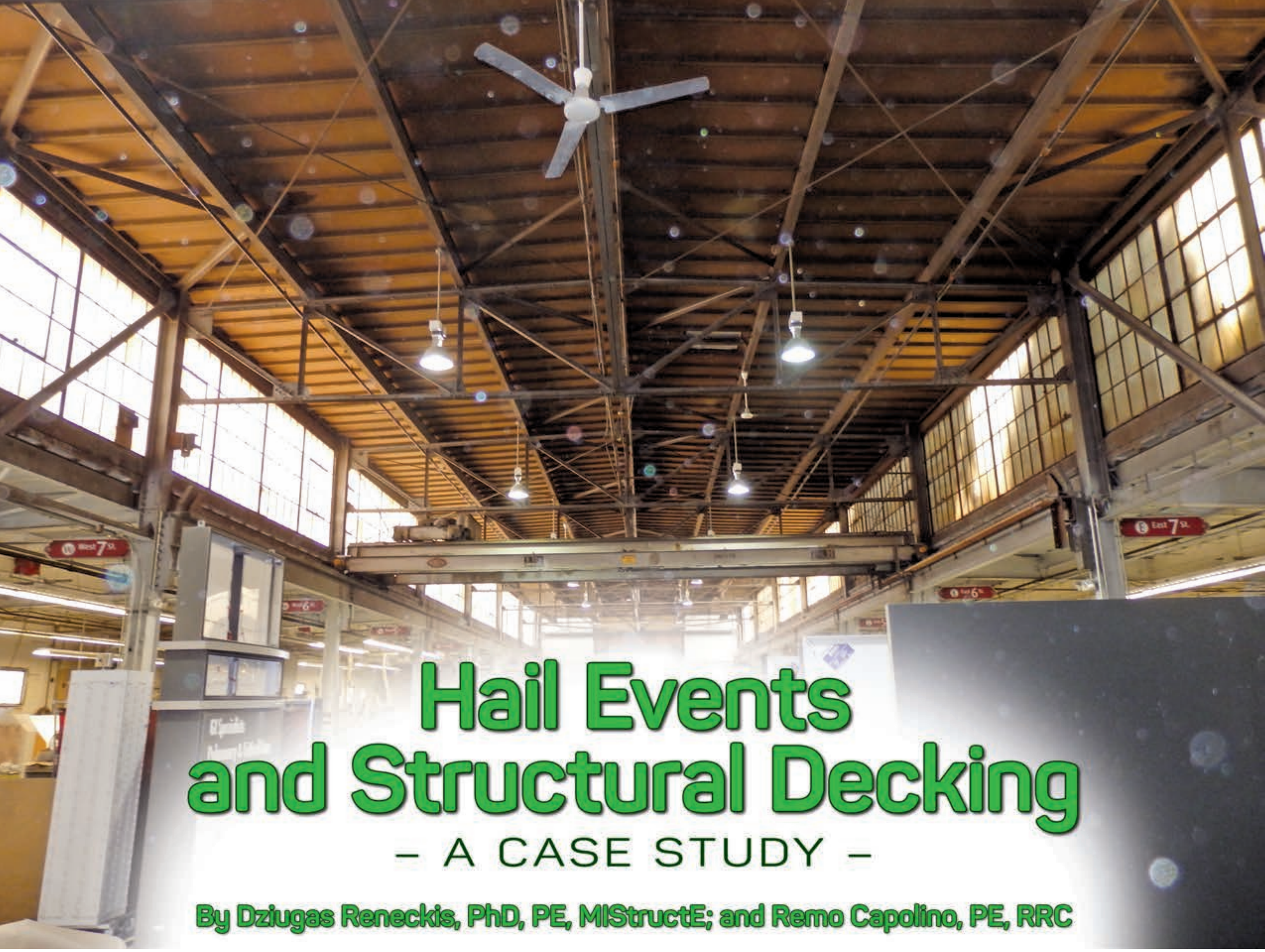


Figure 2 – Interior of building showing low bay and high bay with clerestory windows.

Reading, Pennsylvania, experienced two hailstorms on May 22, 2014 (*Figure 1*). These hail events caused significant and widespread damage to property around the area. Damage was reported to vehicles, sheet metal, and windows, as well as roofs and rooftop equipment.

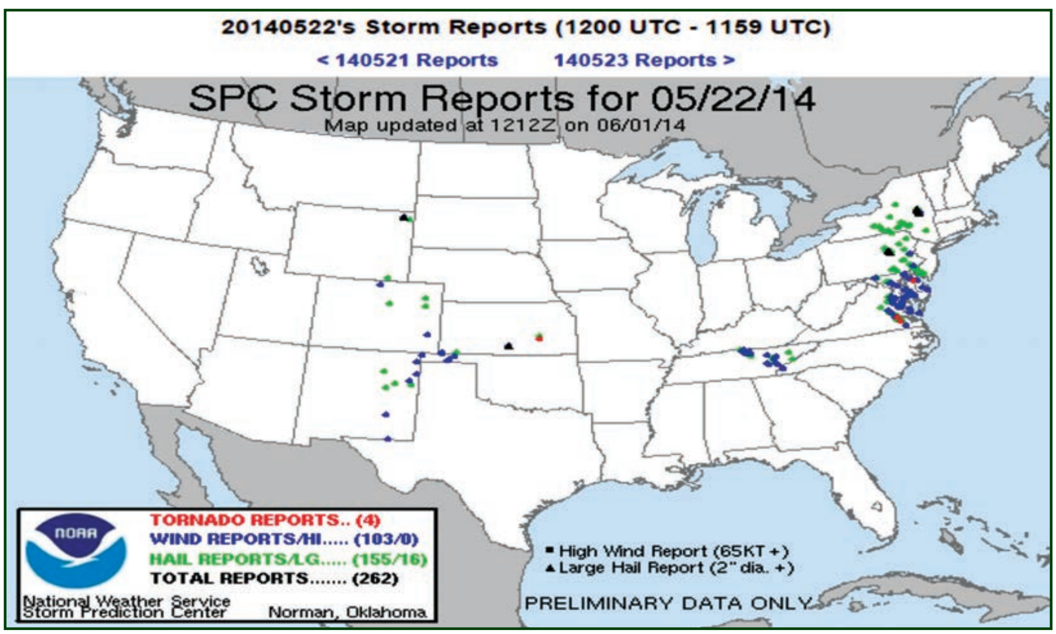


Figure 1 – National Weather Service storm report for May 22, 2014.

ASTM, FM, and UL all provide standards for roofing system hail impact resistance that take into account hail size and speeds that are or can be converted into the force the hailstones impart to the substrate when impacted. The standards typically stop there and simply replicate conditions and forces, allowing professionals

Figure 3 – Dented exhaust hood.

Figure 4 – Damage to A/C fins.

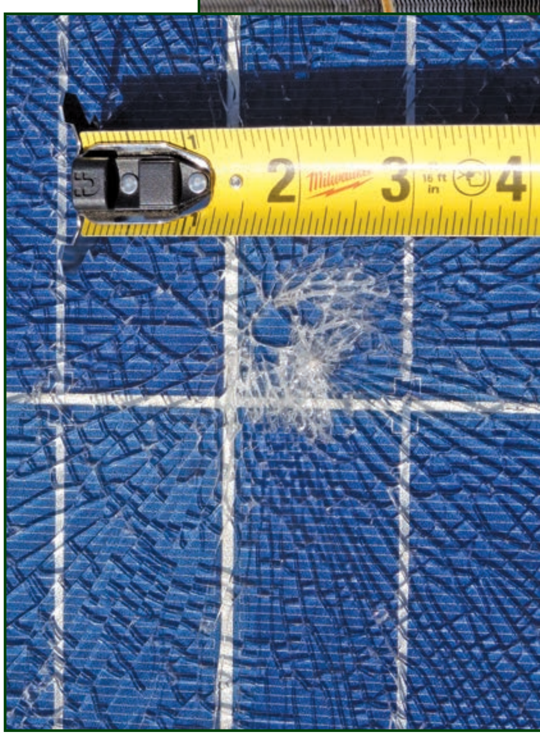


Figure 5 – Damaged solar panel.

to subject new roofing products and assemblies to these conditions and record their performance. This is fairly well understood in the roofing industry, but what about the structural deck below the roofing? What effect does a hailstorm have on the framing of the building?

The Reading hailstorms brought on a spate of insurance claims in which the claimants and their licensed structural engineers asserted that the hail hitting the roofing had caused damage not only to the roofing, but had also caused structural damage to the decking below the roofs. We will be presenting a case study of a single building that was impacted by the hailstorm and the results of our investigation.

The building in question is a fabrication warehouse and is comprised of an original building with several different structures that had been added on over the years. The claim of damage to decking was limited to the original building and, as such, this article will be similarly limited to this portion of the building.

The original building is a steel-framed structure with riveted steel trusses and beams. It is approximately 225 feet long

Figure 6 – Overview of roof assembly at low-bay area.

Figure 7 – Precast concrete channel slab planks at the roof deck.

with multi-wythe brick masonry mass walls at both ends. A section taken across the long axis of the building shows it to be a classic "low-bay/high-bay" structure, approximately 125 feet across, where train cars were directed through the high bay and unloaded into the low-bay areas on either side. The central third of the building has a low-sloped roof approximately 35 feet above a finished floor, while the two low bays have a low-sloped roof approximately 25 feet above the floor. In *Figure 2*, you will see the interior of the building, including the overhead rolling crane used to unload railcars.

The investigation began with the roof assembly. There were all the classic and obvious signs of a hail event: dents, deflections, and shattering of rooftop accessories such as exhaust fan hoods (*Figure 3*), A/C fins (*Figure 4*), K-style gutters, and—on this building—solar panels (*Figure 5*). There was also claimed damage to the low-sloped, fully adhered EPDM roofing on the newer buildings, but that is beyond the scope of this case study.

The roofing over the original low-bay/high-bay building is a loose-laid and ballasted EPDM installed over 3-in.-thick isocyanurate (iso) insulation that was laid directly over the original slag-covered coal tar pitch roof membrane (*Figure 6*). The

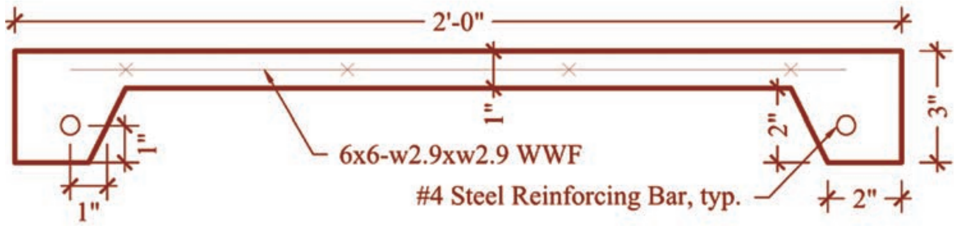


Figure 8 – Design of concrete planks.

gravel ballast was relatively heavy, approximated at 20 pounds per square foot, but included a large amount of fines.

The roof decking below the EPDM and coal tar pitch is precast concrete channel slabs that are gravity-set with retaining clips, spanning approximately seven feet between steel purlins (*Figure 7*). These precast planks are common throughout the roofing industry—not because they are typically installed today—but rather because they were often installed prior to World War II and they are so durable that if the building remains, so, too, do the precast concrete planks. The concrete planks on this building are two feet wide by seven feet long and are one inch thick at all areas except the two long edges where the concrete was thickened to three inches thick. This effectively gives the planks the shape of a blunted "C." The planks in question were weighed and measured and were determined to have been made with lightweight structural concrete with #4 reinforcing steel bars located at the thickened edges (*Figure 8*).

The condition that was cited and claimed as damage to the concrete planks was the loss of cover concrete at the underside of the #4 steel reinforcing bar. This area of spalled concrete was pervasive throughout the building; however, it was not uniformly distributed. Also, the exposed steel and concrete of the plank were observed to be in very different conditions. Some areas of the building had an interior renovation prior to the storm. At these locations, the underside of the planks were painted white and netting was installed under the planks. When the netting was inspected after the storms, large sections of concrete cover were found in the netting, the underside of the concrete plank had many areas of unpainted concrete, and corroded reinforcing steel was exposed (*Figure 9*).

The date of the interior renovation and painting allowed us to determine that the spalled concrete occurred after the renovation; however, we could not determine if it existed prior to the storm. Other areas of the deck were observed, and we found spalled concrete and the exposed #4 reinforcing steel, but these areas were painted white, indicating they were in this condition prior to the hail events (*Figure 10*). With these conflicting observations, it could not be determined with a reasonable degree of engineering certainty that the hail did or did not cause the spalling of the concrete plank. As a result, further investigation was required.

Research into hail and its impact upon the framing of a building revealed virtually nothing. No definitive method to calculate the load transfer from the hail to the structure was found. The reason this is difficult is because the hail event creates an impact load on the structure. Impact loads are different from steady state or dead loads because they can be of a very high magnitude but are of such a short duration that typical structural analysis methods simply do not apply. As a result, we outlined and completed a dynamic hail impact load analysis for the roof planks.

To be conservative and also to greatly simplify the analysis, it was assumed that the hail impact loads were transmitted directly to the concrete planks. That is to say that the gravel ballast and the 3-in.-thick iso insulation had no cushioning effect on the load imparted to the concrete plank. This is, of course, a wild oversimplification, because it is obvious that as a hailstone strikes a ballasted roof, it can break apart and will move some of the existing ballast by imparting some of the hail energy to the ballast. Additionally, the hailstone will give up more of its energy by tearing the paper facer and causing an indentation in the iso insulation. We chose to ignore these losses of energy and assumed a worst-case scenario as when there was no insulation or ballast between the hailstone and the structural deck.

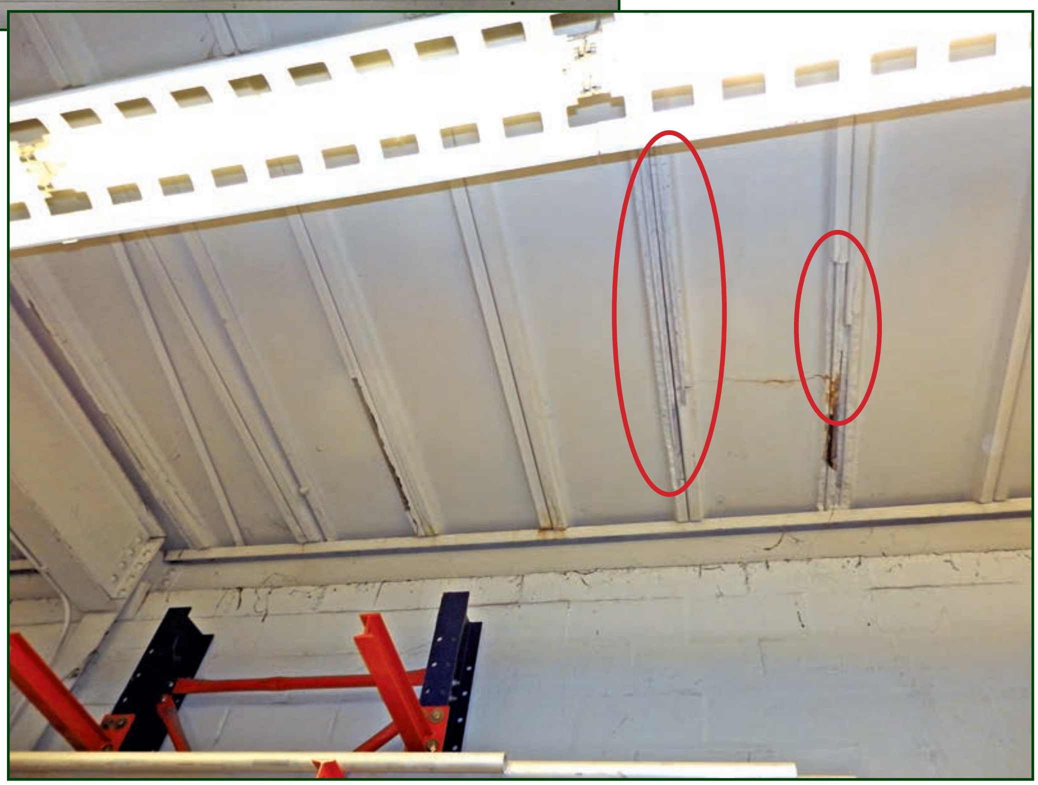
IMPACT LOAD ANALYSIS

Plank Analysis

The first step was to determine the structural capacity of the planks. Since there are still manufacturers of this product, we were able to obtain published data related to the

Figure 9 – Spalled concrete caught in netting with exposed unpainted concrete and reinforcing steel.

Figure 10 – Spalled concrete and exposed reinforcing steel with paint from interior renovation completed prior to storm.



dimensions, as well as the strength of this product. We also calculated the strength of the plank in accordance with ACI 318-11, *Building Code Requirements for Structural Concrete*. Loading combinations per ASCE 7-10, *Mini-mum Design Loads for Buildings and Other Structures*, were applied, and it was determined that the planks can support approximately 45 pounds per square foot of live load.

Load Calculation and Duration

Based upon weather data, FM classifications, and a desire to be conservative, a "severe" impact model was selected; therefore, the load was assumed to be no greater than that represented by the FM Class I-SH Model (Crenshaw and Koontz, 2000 and 2002) and is the equivalent impact energy of a 1.75-in. diameter steel ball dropped from 17 ft., 9 in. The impact force of this missile was calculated to be 84 pounds with a load duration of 10 milliseconds and was assumed to be uniform throughout its applied duration. This uniform load application is again conservative, as it will, of course, start at zero, increase to 84 pounds, and then decrease to zero—all within the calculated 10 milliseconds.

Frequency and Location of Load

Once the load of a single hailstone was calculated, a statistical analysis was performed of the number and location of individual hail strikes—and thereby loads—that occur on a single plank over a specific time interval. A randomly applied hail impact load can be represented by a statistical model of random occurrences over a time period, or a Poisson process (Ang and Tang, 1975):

$$p_x(t) = \frac{(\nu t)^x}{x!} e^{-\nu t}.$$

The inputs of storm duration from hail-tracking data (hail fell for approximately 15 minutes), and the number of hail impacts per square foot observed at multiple test cuts taken at the roof (average of 58 per sq. ft.) were used, and it was calculated that on a single plank, on average, one hail-

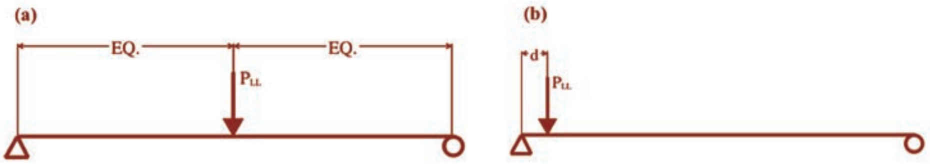


Figure 11 – Case 1 (a): concentrated impact force at mid-span; and Case 2 (b): concentrated impact force near the support.

stone landed every 1.1 seconds. Based upon this, the mean occurrence rate of impact loads is approximately 0.90 hailstones per second. Therefore, the probability that a single roof plank was subjected to more than one impact over a hail strike loading duration of 0.01 seconds (10 milliseconds) is 0.004%, and is calculated by:

$$P(N > 1) = 1 - [P(N = 0) + P(N = 1)],$$

$$= 1 - e^{-0.9(0.01)}\left[1 + \frac{0.9(0.01)}{1}\right],$$

$$= 1 - 0.99996 = 4.0x10^{-5}.$$

A probability of 0.3% is generally considered to be the threshold for practical certainty in statistical analysis for engineering design per ASTM E122, *Standard Practice for Calculating Sample Size to Estimate, With Specified Precision, the Average for a Characteristic of a Lot or Process*. Therefore, it was estimated that any single concrete plank may have been subjected to upwards of four hail strikes over a duration of 1.1 seconds (the average time between hail strikes), based upon the low probability of occurrence of 0.3%.

Due to the calculated duration of loading from a single hail strike, and the statistical improbability that two strikes will occur simultaneously, we chose to assume the next most conservative loading scenario of four hail strikes occurring—one immediately after another—over a duration of approximately 1 second. Two cases were assumed: one where all four of these hail strikes occurred at the center span of the plank, and a second case where all loading was applied near the support. These two loading scenarios will impart the maximum moment and shear forces to the plank, respectively. See *Figure 11*. Again, this loading scenario was done so as to be as conservative as possible by calculating the worst-case scenario, no matter how unlikely it is to occur.

The concrete planks were modeled using CSI SAP2000 structural analysis software with 84-pound loads applied at 0.01-second intervals. Based on the above statistical analysis, four impact forces were applied at intervals equal to the period of vibration of the planks (which we calculated to be equal to approximately 0.06 seconds), resulting in a cumulative application of impacts, adding to the amplitude of the precast concrete plank response (*Figure 12*). This approach is considered to be somewhat conservative because during the hailstorm event, it is likely that half of the hailstones landed on the roof surface in sync, while the other half landed out of synch, which thereby reduced the amplitude of the dynamic response of the planks.

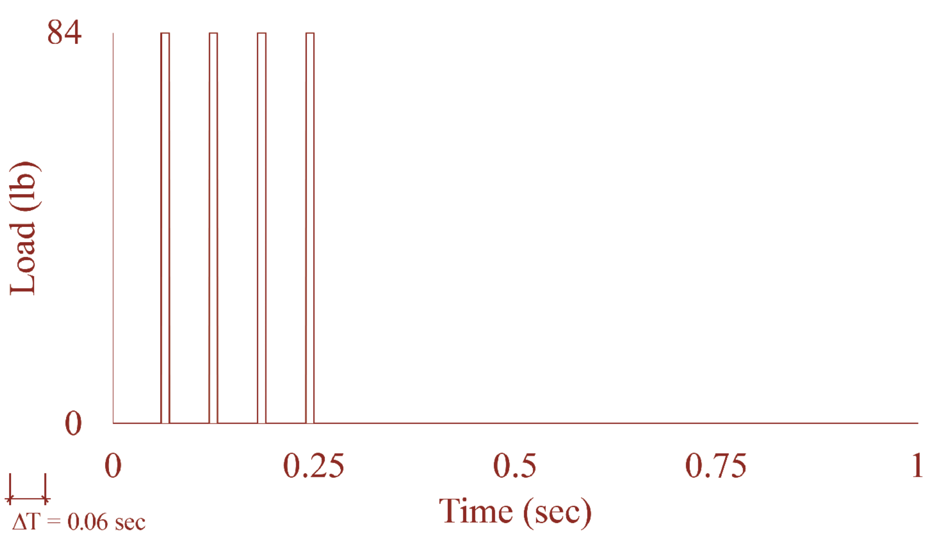


Figure 12 – Idealized impact force history.

Plank Reaction to Applied Loading

When the calculated loads were applied in accordance with the calculated timing, the resultant deflections, shear force, and bending moments were determined and are plotted in *Figures 13* through *15*. All calculated loads were marginal when compared to the plank structural capacities that were initially calculated.

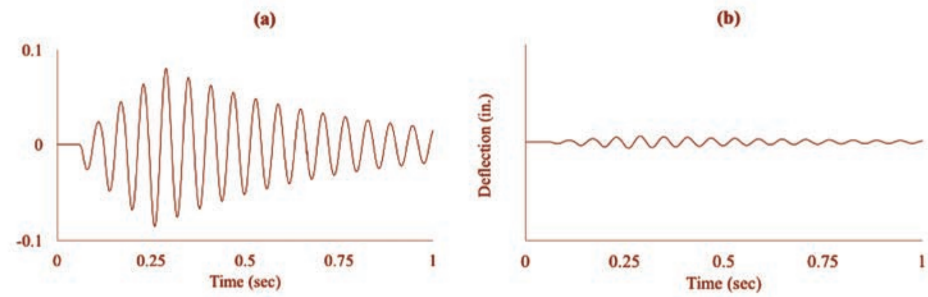


Figure 13 – Deflection results for Case 1 (a) and Case 2 (b).

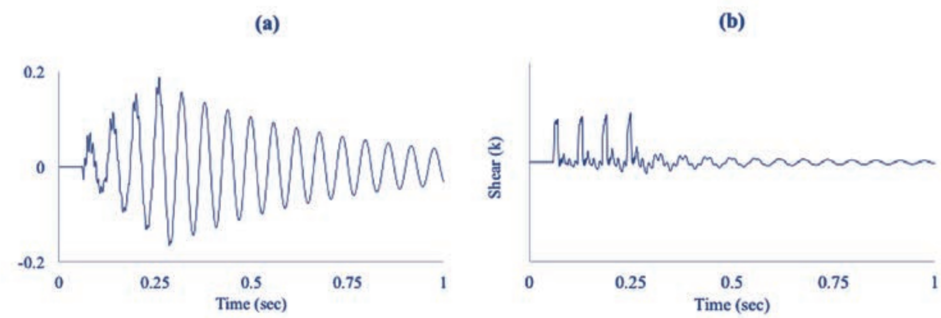


Figure 14. Shear force results for (a) Case 1 and (b) Case 2.

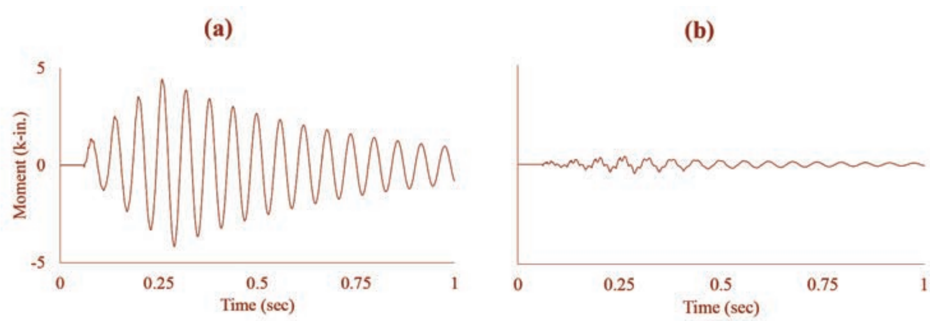


Figure 15 – Bending moment results for Case 1 (a) and Case 2 (b).

Analysis Case	Load Combination	Resultant Bending Moment		Resultant Shear	
		Moment (k-in)	Utilization[2]	Shear (k)	Utilization[2]
Case 1: Impulse Forces at Mid-span	Governing Design Combination 1.2D + 1.6L	7.08	U = 0.40	0.299	U = 0.61
	Estimated Failure Load 1.0D + 1.0L	4.43	U = 0.25	0.187	U = 0.32
Case 2: Impulse Forces Near the Support	Governing Design Combination 1.2D + 1.6L	7.80	U = 0.15	0.158	U = 0.42
	Estimated Failure Load 1.0D + 1.0L	4.92	U = 0.11	0.099	U = 0.23

[1]Estimated shear and moment capacities for the precast concrete planks:
Nominal (without factors) $V_n = 1.0$ k $M_n = 28.7$ k-in.
Factored ($\phi = 0.9$ for moment and $\phi = 0.75$ for shear) $\phi V_n = 0.75$ k $\phi M_n = 25.8$ k-in.

[2]U = Utilization Ratio = Ratio of combined applied force to strength:
$[1.2D + 1.6L]/\ \phi(M_n \text{ or } V_n)$ For LRFD factored design load combinations (includes strength reduction factor $\phi = 0.9$ for moment and $\phi = 0.75$ for shear)
$[1.0D + 1.0L]/(M_n \text{ or } V_n)$ For estimated unfactored load components (does NOT include strength reduction factor)
Values of U≤1.0 indicate satisfactory element capacity and behavior, while values of U>1.0 indicate overstress in the elements. Hence, the greater the value of U, the more likely element failure will occur.

Table 1 – Structural review of precast concrete planks.

As can be seen in *Table 1*, the worst-case scenario is when the planks' shear service capacity is 61% utilized when the plank is loaded at center span.

The fact that the shear capacity of the planks will be reached/exceeded by exterior loading before the moment capacity of the planks can be reached is enlightening. Shear and moment failures of concrete planks result in very different types of failures. "Classic" moment failures of concrete planks are when the concrete at center span deflects downward and develops cracks on the bottom surface and can result in loss of reinforcing steel cover as was seen at the site. However, since these planks are limited in strength by shear, excessive loading failure should be evidenced by shear failures. Classic shear failures are ones where the concrete plank fails at or immediately adjacent to the support and are very different from the conditions observed at the site (*Figure 16*).

Since the allowable/usable moment and shear capacities were always 40% or more than the applied loading, assuming all worst-case scenarios, it was concluded that the loading due to hail did not cause the damage to the concrete planks.

Once the structural analysis proved it was not the applied loading that led to the damage, it was fairly simple to show that long-term corrosion of the reinforcing steel caused rust-jacking that caused the concrete to spall and expose the steel. This causation of the damage was corroborated by the fact that many areas of spalled concrete were painted white (indicating that

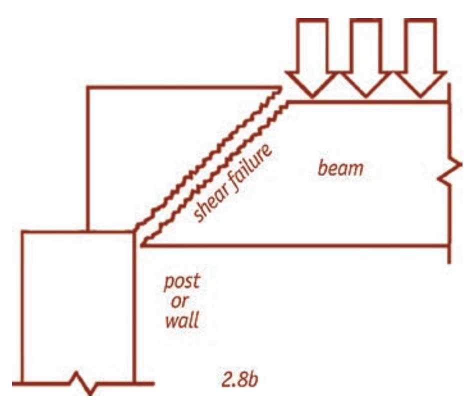


Figure 16 – Location of a shear failure is at support, not mid-span of structural member. Graphic courtesy of http://boulderlibrary.net/timber-framing-for-the-rest-of-us-rob-roy/shear-and-shear-failure.html.

the spalling of the concrete cover over the reinforcing bars occurred prior to the hail event), as well as the lack of any damage to the ballasted EPDM roofing above these allegedly damaged concrete planks.

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Dziugas Reneckis

Since joining Wiss, Janney, Elstner & Associates (WJE) in 2014, Dziugas Reneckis has performed field investigations, condition surveys, construction period services, and special inspections involving steel frame, masonry, wood, and reinforced concrete structures. Additionally, he has provided structural services associated with the renovation of building façades and interiors. Before joining WJE, he concentrated on the design of special structures in glass and steel, as well as façade engineering. Reneckis completed his undergraduate and graduate studies in civil engineering at the University of Illinois at Urbana-Champaign.

Remo R. Capolino

Remo R. Capolino, a principal with WJE, grew up in a family-owned specialty roofing contracting business, and graduated from the University of Connecticut with a BS in civil engineering. After more than 15 years in contracting and leadership roles with the Association of General Contractors (AGC), Northeast Roofing Contractors Association (NERCA), and the National Roofing Contractors Association (NRCA), he turned to consulting. He is a member of the RCI Interface *Peer Review Board.*

PA SCHOOL DISTRICT REJECTS COOPERATIVE PURCHASING AGREEMENT ROOF REPLACEMENT; SAVES $800,000

The Schuylkill County Area Vocational Technical School Board in Mar Lin, Pennsylvania, hired a construction manager to replace the roofs at the Schuylkill Technology Centers in Mar Lin and Frackville recently at a projected cost of $1,710,000—approximately $800,000 less than would have been spent had it gone through with a state contract Master Intergovernmental Cooperative Purchasing Agreement arrangement with The Garland Co.

The board had been planning to hire Garland, a roofing manufacturer it had worked with numerous times, to do the project for $2.519 million. But board member David Frew suggested the roof committee investigate alternatives to the no-bid Garland deal. Frew wrote in an e-mail to the *Republican Herald* that Garland was "the manufacturer and the installer and the designer, and it was specified just for their product." As a result, the district had "expected that they would come in a few percentage points lower" than the "not-to-exceed" number of $2.519 million established through the U.S. Communities cooperative purchasing deal. This was not the case.

"The state contract procurement, while convenient, may not offer the most competitive approach to this type of work. Our experience with other school districts is that better pricing can be obtained from the marketplace via competitive bids vs. the state contract," Frew wrote.

Instead of signing the contract, the committee researched options and hired Performance Construction Services as the district's construction manager. Performance suggested an EPDM synthetic rubber roofing membrane roof with a 30-year warranty, and estimated the two projects should come in closer to $1,710,000. The project will be bid and will include architecture, engineering, and roof consulting fees to observe the installations.