with roller expansion bearings so that the structure could expand laterally in response to temperature. Because the existing wrought iron anchor bolts were too short to accommodate the fitted expansion bearings used in the new design, the wrought iron anchor bolts were spliced to steel anchor bolt extensions by means of threaded, steel collar coupling assemblies. (Grimm 1901) The collar coupling assemblies were encapsulated by large diameter circular washers, and were consequently obstructed from view prior to the collapse. (Figure 1)

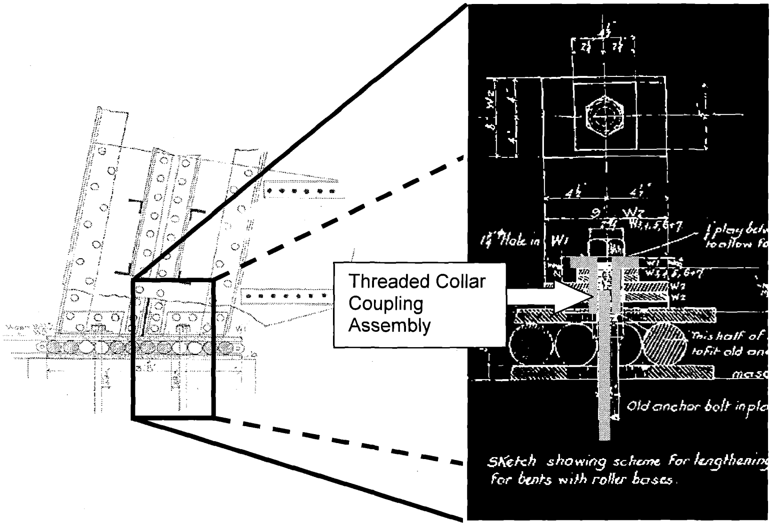


Figure 1 – Wrought Iron Anchor Bolt Extension via Threaded Collar Coupling Assembly

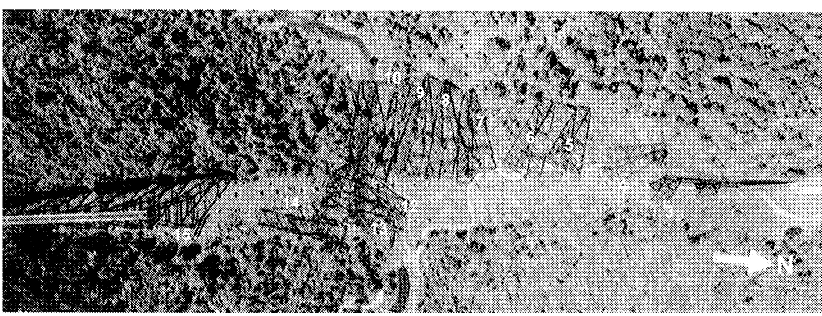
As the 20th century progressed, nearby coal resources were depleted, and rail traffic diminished. The viaduct remained in service but activity was light. Finally, in 1959 the railroad company sold the structure for scrap to the owner of a private salvage company. Realizing its value as a historic resource, the owner chose not to dismantle the structure, but instead, sold it in 1963 to the Commonwealth of Pennsylvania, who then established at the site a state park that featured the bridge as its centerpiece. Listed in the National Park Services National Register of Historic Places in 1977 and designated a civil engineering landmark by ASCE in 1982, the viaduct was used by a private railroad concessionaire from 1980 through early 2002. That year, Pennsylvania Department of Conservation and Natural Resources closed the structure after a routine inspection uncovered severe deterioration in observable structural elements of the towers. In 2003, repair work that focused solely on the restoration of these visibly deteriorated tower elements began and was progressing at the time of the collapse.

Board of Inquiry of Investigation

The Commonwealth of Pennsylvania has established guideline procedures, in Pennsylvania Department of Transportation Publication 220 dated July 2001, which govern the investigation of a catastrophic collapse. The intention of these procedures is to provide a thorough forensic and analytical investigation of a catastrophic collapse by a team of specialists, who are entirely independent from any ongoing design, maintenance, construction, or rehabilitation activities associated with the structure. Within the framework of these guideline procedures, the following methodology was established for conducting the forensic investigation.

Methodology of Investigation

Strict protocols were established to support the field and analytical phases of the investigation. These protocols were established within 24 hours of the incident and set in motion a rapid chain of events. Within one week of the collapse, [1] a team of specialists, skilled in forensics, meteorology, and fracture interpretation were assembled and placed under contract to the Pennsylvania Department of Conservation and Environmental Resources, [2] all arrangements were made to conduct a one day field investigation including all logistical support necessary to freely transport all investigative personnel over the rugged terrain, and [3] high resolution aerial photography of the 1200 m (4000 ft) wind impacted area was obtained by the Commonwealth of Pennsylvania, Department of Transportation. (Photograph 2)



Photograph 2 – High Resolution Aerial Photograph – tower numbers indicated

The field investigation phase relied upon the assembly of a team comprising a wide range of engineering and scientific disciplines, who conducted a single day examination of the site. The team was delegated with specific investigative inspection responsibilities to complete the investigation in a single day as required by the Commonwealth's guidelines. The site was cordoned and the debris field was treated as evidence which was permitted to be observed but otherwise was to remain untouched, with the exception of select material samples taken off site for laboratory examination. The team was subdivided into three squads which were individually tasked with (a) the examination of the 1200 m (4000 ft) roughly circular wind impacted area of the park surrounding the structure, to assess the gross scale meteorological implications, (b) the examination of the 320 m (1250 ft) debris field on a span by span basis, to systematically record all damaged elements of the structure, observe and record obvious forensic markers and develop initial hypotheses regarding the sequence of collapse, and (c) the examination and selection of limited fractured samples for further laboratory investigation, to support the follow-on analytical phase of the investigation. Additionally, eyewitness testimony, was recorded from all park maintenance personnel and all construction laborers who were present on the site during the storm event and subsequent collapse. The eyewitness testimony was taken with the understanding that accounts likely were emotionally charged and were taken from individuals without scientific backgrounds.

Physical and mechanical properties as well as chemical composition and metallurgical microstructure were determined through laboratory investigation for the fractured wrought iron anchor bolts, the fractured steel collar coupling assemblies and the steel anchor bolt extensions.

Subsequent analytical investigation included back calculation of wind velocity necessary to induce a rotational failure of a tower element, based on visual observations evidenced at the site during the investigative phase. The back calculation relied on the strength of connection between substructure and superstructure determined from laboratory testing and utilized published wind coefficients (ASCE-7). The back calculation additionally considered inertial effects. In addition, the analytical investigation relied on careful examination of the high resolution aerial photography as well as eye-witness testimony to synthesize a complete and consistent explanation of the initiation and sequence of the collapse. The analytical investigation concluded with the preparation of a comprehensive report accompanied by animated computer renderings demonstrating the sequence of collapse. Upon presentation to responsible government agencies, the complete *Board of Inquiry* report and animations were made available to the general public on the Pennsylvania Department of Conservation and Natural Resources' web site. (http://www.dcnr.state.pa.us/info/kinzuabridgereport/kinzua.html)

Findings

In the course of a forensic investigation, there are many indicators, that when taken collectively, decode the event. (Leech et al. 2004) At the Kinzua site, the following four distinct forensic markers were apparent:

Order markers – including the ordering of materials clustered within a debris field. The inversion of clustered materials within a debris field allows the reconstruction of the direct order of collapse. Order markers were apparent during the field inspection enabling the determination of the precise order of tower collapse.

Directional markers – including both the direction of fallen trees and collapsed towers. (Figure 2) Directional markers were evident through high resolution photography and revealed wind directed in two orthogonal directions (initially from the east, then from the south). The observation of these markers confirmed the sequence of collapse as well as the contribution of both "vortex leading edge" (easterly) winds accompanied by "inflow" (southerly) winds occurring in rapid sequence in this extreme weather event.

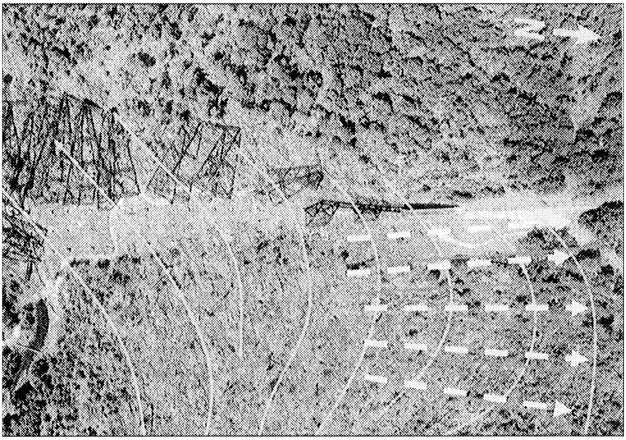


Figure 2 – Depiction of Wind Streamlines (Directional Markers) with "leading edge" wind streamlines – directed westward; and "inflow" wind streamlines – directed northward

Separation markers – including all evidences of "clean" breaks. (Figure 3) During the field investigative phase, it was apparent to the investigative team that the initiation of failure occurred at boundary between superstructure (of trestle bent configuration) and substructure, most likely at the specific boundary of the 1882 (original construction) and the 1901 (reconstruction).

At this boundary, the original wrought iron anchor bolts and masonry were preserved in the reconstruction. Fractures were observed within certain wrought iron anchor bolts (1882 construction) and a majority of expansion bearing, steel collar coupling assemblies (1901 construction), that provided connectivity between the original masonry and reconstructed superstructure. Consistently "clean breaks" were observed at the interface of the 1882 and 1901 construction. The separation markers were observed during the field inspection. The separation markers evidenced distinctive separation of the superstructure from its roller bearing assembly at the base of the superstructure for all eleven collapsed towers.

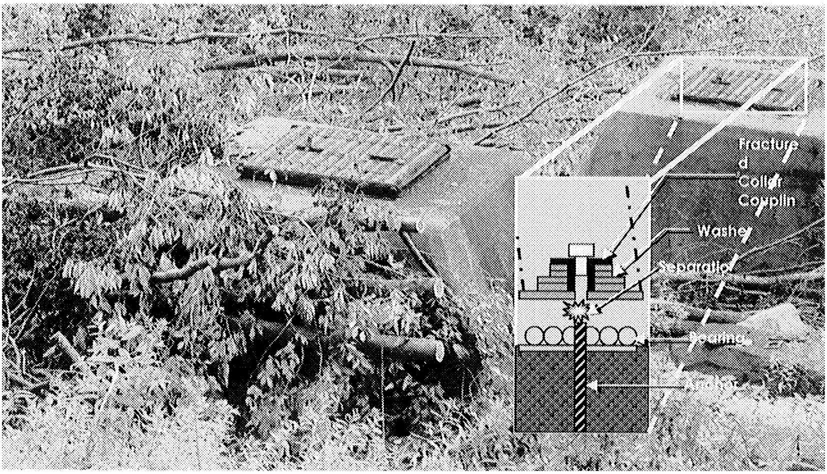


Figure 3 – Illustration of separation failure at substructure/superstructure interface

Fracture markers – including steel evidence of consistent patterns of small sub-critical fractures within members. (Photograph 3) Consistently, small sub-critical fractures were observed in the steel collar couplings connecting the 1882 construction and the 1901 construction on the windward side of the towers. Subsequent fractographic examination using a light microscope and microscopic examination using a scanning electron microscope, revealed evidence of long term fatigue crack propagation. The visual appearance of the fractures was characteristic of fatigue fracture with a flat, smooth surface without any evidence of plastic deformation in the fracture region. Microscopically, the fracture morphology also appeared fatigue-like with smooth featureless regions, typically found in high sulfur steels. (Photograph 4) No evidence of fatigue beach marks or striations were observed in these regions, however, the absence of evidence of brittle fracture by a cleavage mechanism or ductile fracture mechanism by dimple fracture suggested that the flat regions were most likely the result of stable crack propagation by a fatigue mechanism.

Based on the four forensic markers and subsequent analysis, the *Board of Inquiry* investigation concluded that failure initiated at the "weak-link" of the system – the anchor bolt system on the eastern faces, which was initially installed in the 1882 construction and subsequently modified in the 1901 construction. The 1901 construction provided a collar coupling assembly which included a series of washers that surrounded the anchor bolts and couplings. Consequently, the through wall-cracking of the collar couplings was hidden from view during routine condition inspections.

Photograph 3 – Delineating Through-Wall Cracking

The *Board of Inquiry* investigation concluded that the circumstance of a nearly north-south structure alignment and fractures within the collar couplings at the eastern tower legs resulted in a structure which was specifically vulnerable to winds from the east but not otherwise vulnerable to prevailing westerly winds. (Leech et al. 2003)

All eleven collapsed towers were fitted in the 1901 reconstruction with expansion bearings secured to the existing masonry via (1882) wrought iron bolts and (1902) collar coupling assemblies. Based on site observation that for the majority of these locations, three out of four of the anchor bolt assemblies at each tower leg displayed complete separation of the superstructure from the substructure at the collar coupling connection, the immediate failure at the expansion bearings may be characterized as a separation failure. As this separation failure resulted in a rotational failure of each tower about the fixed (or opposite) tower bearings, a back calculation of the force effects necessary to overturn the structure became the basis for prediction of the limiting wind velocity. (Figure 4) This back calculation derived uplift capacity based on the fracture of one anchor bolt out of four (with no strength attributed to the remaining three anchor bolts within a typical four bolt tower location) and utilized limit strengths derived from specimen samples. This back calculation subsequently derived an applied wind velocity of 42 m/s (94 mph) accompanied by a wind pressure of 1.9 kPa (39 psf) and recognized two distinct modes of failure of the anchor bolt system.

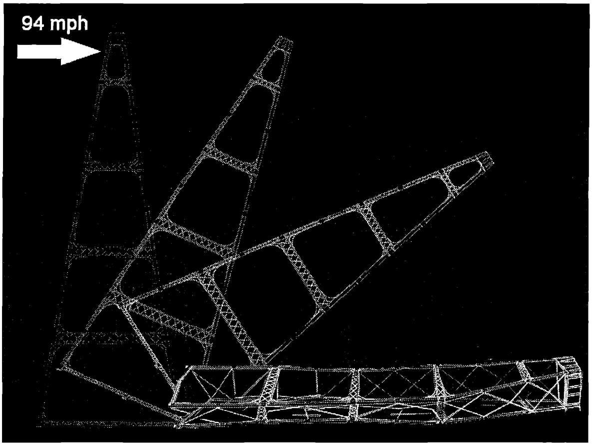
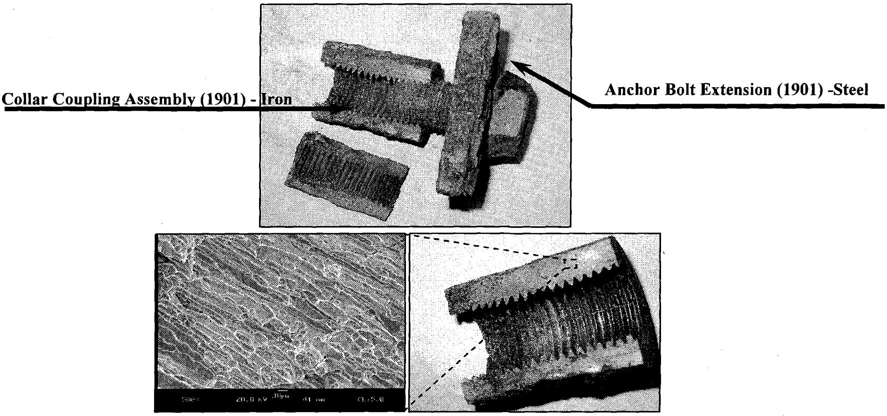


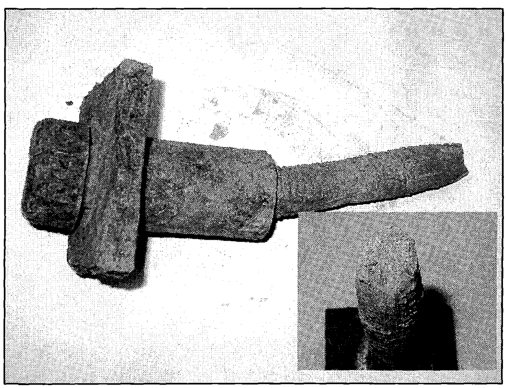
Figure 4 – Illustration of Rotational Failure Mechanism

Failure Mode 1 – Coupling Failure at the Boundary of 1882 and 1901 Construction – Expansion Bearing Anchor Bolt Collar Coupling. This mode accounts for approximately 3/4 of observed separation failures. All collar couplings observed at the site exhibited a radial cracking pattern, along with multiple longitudinal "splits" within the anchorages. The equiangular "splits" completely penetrated the collar couplings. The collar couplings found throughout the debris field exhibited similar fracture indications. Coupling failures showed evidence of fatigue fracture with secondary fractures occurring by overload presumably during the collapse. (Photograph 4) The *Board of Inquiry* investigation concluded that the couplings, which separated from the bearing assemblies and which were strewn within the debris field, experienced long term fatigue crack propagation prior to the time of the collapse incident. Because the cracks propagated through the entire thickness of the coupling, these collar couplings were judged to be ineffective for the transmission of uplift forces to the substructure. (Kaufmann and Connor 2003)

Mode 2 – Ductile Failure within the existing 1882 anchor bolts – Expansion Anchor Bolts. (Photograph 5) This mode accounts for approximately 1/4 of observed failures. Fractographic examination of fractured original 1882 anchor bolts showed that the fracture resulted from tensile overload and was a fully ductile fracture. (Kauffmann and Connor 2003) The estimated tensile capacity of a single (1882), 31.75 mm (1-1/4 in) anchor bolt at failure, considering a 20% corrosion loss, was determined to be 13 kN (30 tons) based on Brinnel Hardness evaluation. Based on the observed, 3:1 ratio of collar coupling failure to ductile anchor bolt failures, an uplift capacity of 13 kN (30 tons) was attributed to each tower. This capacity established a lower bound, critical wind speed of 42 m/s (94 mph), which was sufficient to initiate failure. (Leech et al. 2003) The failure was sudden and catastrophic.



Photograph 4 – Laboratory Investigation of Through-Wall Cracking
(demonstrating long term fatigue crack propagation)



Photograph 5 – Wrought Iron Anchor Bolt – Ductile Fracture

The collapse of eleven supporting towers and twenty-three of the forty-one structure spans was rapid and proceeded in three distinct and separate episodes as illustrated on the accompanying figures. All girders and towers between towers 3 and 15 collapsed. Separation of the structure into three distinct collapsing segments is attributable to the arrangement of the wind locks within the girder system and to the nature of the wind event. (Figure 5) The 1901 design introduced expansion joints (and accompanying wind locks) at irregular locations within the structure. The collapse of the structure in three distinct episodes was controlled by the location of these wind locks.

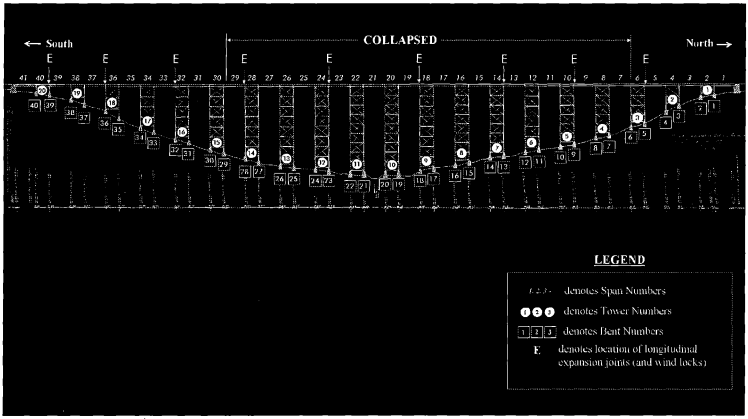
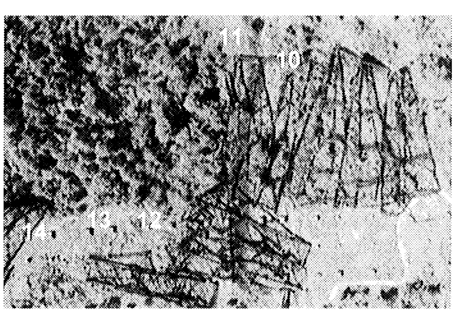


Figure 5 – Structure Elevation View Demonstrating Wind Lock Location

The following occurred in Episode 1 in the sequence indicated. (Photograph 6)

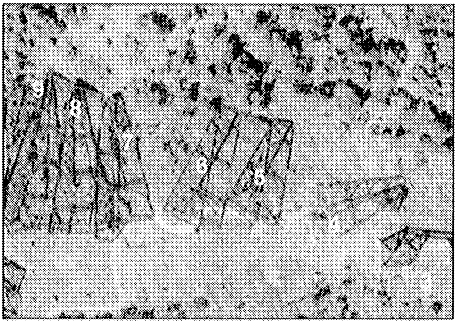
1. Tornado touched down – easterly (or "vortex leading edge") winds grew rapidly – local wind speeds (from the east) exceeded 40 m/s (90 mph) – as wind speeds grew, the towers oscillated laterally in response to their natural frequency.
2. "Separation" failures occurred within the "expansion" anchor bolt system of Towers 10, 11, 12, 13 and 14.
3. Rotational failure accompanied by collapse of Towers 10 and 11 and adjoining spans occurred.
4. Towers 12, 13 and 14 initially become airborne and "jumped" a small distance north and westward. Towers 12, 13 & 14 momentarily came to rest in the upright position on the ground, not initially collapsing. The rails and wooden decking for a brief period of time remained affixed to several spans and held the three towers in a vertical position, initially preventing immediate catastrophic collapse.



Photograph 6 - Debris Field – Episode 1 – tower numbers indicated

The following occurred in Episode 2 in the sequence indicated. (Photograph 7)

1. Tornado moved northward – easterly ("vortex leading edge") winds grew rapidly – local wind speeds (from the east) exceeded 40 m/s (90 mph). As wind speeds grew, all towers oscillated laterally in response to their respective natural lateral frequencies.
2. Wooden decking and rails (spans 1 – 18) separated from the structure.
3. "Separation" failure occurred in sequence within the "expansion" bearings of Towers 9, 8, 7, 6, 5 and 4.
4. In sequence, rotational failure of Tower 9 occurred, shortly, followed by rotational failure of Towers 8, 7, 6, and 5. Collapse was progressive from South to North.
5. Tower 4 was initially restrained by Tower 3 and the connecting girder span. However, after elongation of the girder span's connection to Tower 3, rapid collapse and clockwise twist of the tower occurred. Tower 3, although standing, was visibly distorted.



Photograph 7 - Debris Field – Episode 2 – tower numbers indicated

The following occurred in Episode 3 in the sequence indicated. (Photograph 8)
1. Tornado moved northward – rapid and confined southerly ("inflow") winds attack from the south.
2. The wooden decking and rails, momentarily connected to Towers 12, 13 and 14 during Episode 1, separated from the structure.
3. Towers 12, 13 and 14 and adjoining girder spans, having separated from the bearings during Episode 1, twisted and subsequently collapsed in a southerly direction.
4. The final remaining girder span, momentarily affixed to Tower 15, oscillated laterally several times at the Tower 15 connection, eventually separating and rotating upside down before impact. The rails remained attached and "hung" from Tower 15.