FORCON INTERNATIONAL - NC, LTD.
1730 Mt. Vernon Road, Suite H, Atlanta, GA 30338
770/390-0980 • 770/390-0981 FAX • www.forcon.com
Firm License #: C-1617

May 23, 2022

Lucie Peoples, Attorney
McAngus Goudelock & Courie PLLC
10 Brook Street, Suite 200
Asheville, NC 28803

RE:	City of Hickory (Plaintiff) vs. Neill Grading and Construction Company, Inc., Dane Construction, Inc., and Western Wood Structures, Inc. Preliminary Report – **Attorney Work Product**
CASE #:	22-CVS-804 (North Carolina, Catawba County, Superior Court)
FORCON CASE #:	NC22003-1

Dear Attorney Peoples:

As Counsel for Neill Grading and Construction Company, Inc. (Neill), you have requested that I (Dara Thomas, P.E., a FORCON International Corporation (FORCON) Professional Forensic Structural Engineer), provide services as an Expert and prepare a Preliminary Report on this matter. **The purpose of the report is to express my opinions, to-date, about the catastrophic failure of the Iconic Arches at the City Walk Project, Hickory, NC.**

BACKGROUND INFORMATION

The Iconic Arches over the Pedestrian Bridge (also known as the Rudy Wright Bridge) portion of the Project were completed and opened to the public in approximately December of 2021. The Arches were constructed of GluLam wood components, steel plates, bolts and guy wires, which were positioned from the apex of the Arch to concrete bases on the sides of the bridge. The Pedestrian Bridge is located between the NCDOT Main Avenue Bridge, and a railroad bridge known as Norfolk Southern bridge. All three bridges cross over NC Highway 127. On the next page is a photo of the Bridges and Arches sometime after completion, but prior to the collapse. The photo was obtained from the *Hickory Daily Record* article dated February 18, 2022 and was taken by Robert C. Reed.

In the early morning hours of February 18, 2022, the Iconic Arches collapsed to the north side and came to rest on the Main Avenue Bridge. Some debris fell between the Pedestrian Bridge and the Main Avenue Bridge, onto NC Hwy 127 below. On the next page is a photo taken after the collapse. This photo was received from Neill.

The completed Iconic Arches and adjacent bridges (Credit: Robert C. Reed)

The collapsed Arches and all three bridges. (Credit: from Neill's files – image DJI_0039)

SUMMARY OF OBSERVED CONDITIONS AT SITE INSPECTION (Physical Evidence)

I performed a site inspection on February 22, 2022, in order to view, inspect and document the collapsed remains of the Iconic Arches at the City Walk project in Hickory, NC. In attendance with me at the inspection were: Mike Wilson (VP-GM of Neill), Todd Shoebridge (Neill), Jake Lantz (Construction Manager, Freeze & Nichols), Steve Miller (City of Hickory Engineer) and Will Hamblin (City of Hickory Engineer). This investigation is limited to the Iconic Arches. No investigation of the structural stability of the bridges was conducted by Forcon.

During the non-destructive site investigation, the property was visually inspected and photographed, and field notes documenting pertinent information were prepared. Go-Pro videos were also taken of the remains. Some of these photographs are included as Photographs in this report. Photographs and Go-Pro videos taken during the inspection, which are not included in this report, are retained in the project files for potential future use, and will be readily available during the Discovery phase of this matter.

Photo 1 shows a view of the collapsed arches, approaching from the west end of Main Avenue bridge. The site was secured against pedestrians entering the area.

Photos 2-11 are images of the multiple base plate connections of the "legs" of the arches. Each connection has four ¾" diameter bolts through a base plate, into the concrete foundation. In all locations the base plate and anchor bolts were either deformed or completely pulled away from the foundation.

Photos 12-22 are images of the 7/8" diameter guy cables that were anchored to the apex of the arch space frame. There were two cables on each side of the arch structure. These cables were anchored at the bottom to concrete *bents* which support the pedestrian bridge. At the apex of the arch, four cables were affixed to two steel brackets which were connected to the arch assembly. The steel bracket at the bottom of the lower arch is of interest because it was discovered during the inspection that the steel bracket (known as S15 in the Western Wood shop drawings) had pulled apart. It broke at a welded seam. Part of the bracket S15 was still attached to the bottom of the apex of the arch and part of S15 was lying on the Main Avenue bridge. Also, the cable still attached to the part lying on the bridge was nearly pulled completely apart from tensile forces.

Photos 23-28 focus on the connection assembly at the apex of the arches.

Photo 29 shows a view of the failure of a kerf plate at a connection node.

Photo 30 documents the collateral damage to the concrete guardrail of the Main Avenue bridge. Part of the Arches crushed the guardrail.

Photo 31 shows some debris of the arch which fell to NC Hwy 127 below. Some debris fell between the pedestrian bridge and the Main Avenue bridge.

DOCUMENTS REVIEWED

As part of my investigation, I reviewed the following documents. This list is not exhaustive nor exclusive, as Discovery is ongoing.

1. Civil Action Complaint: City of Hickory v. Neill Grading and Construction Company, Inc., Dane Construction, Inc. and Western Wood Structures, Inc. Case # 22-CVS-804.
2. CONTRACTS 01-13, EB-5750 City Walk Contract/Bid Documents (also contains the Specifications for the project).
3. Three security camera videos of the collapse, received from Neill. Files are labeled as: 2022-02-18_08-39-13-806_1; FPS arch video; ORDKE7863. Below is an image clipped from one of the videos. The moment of the collapse appears to have started about seven minutes after midnight on February 18, 2022 (if the time and date stamps were set properly on the security system).

The lower red arrow is pointing to the node where the arch began to pivot to the north. This is node 4 or 22 of the computer analysis model, depending on which side one is viewing. The upper arrow points to the apex (top) of the arch space frame – which is falling over to the north (left in this photo).

4. Certificate of Liability Insurance from Dane to Neill (2-24-2021).
5. Certificate of Liability Insurance for City of Hickory (2-25-2021).
6. The Dane Subcontract with Neill (AIA Document A401-2017) dated July 12, 2019, and Exhibits A-I for the Subcontract.
7. Images labeled DJI_0013 through DJI_0039, image DJI_0700, image DJI_0724 and images IMG_0299 through IMG_0316 received from Neill.
8. EB-5750 City Walk Plans (398 pages) dated February 28, 2019, on the cover page, and produced by Wood Environment & Infrastructure Solutions, Inc. (WEIS), with various sections signed and sealed by Brian E. Johnson, PE on March 6, 2019 and James Studer, PE on March 7, 2019 and H. H. Deeb, PE on March 6, 2019. The Pedestrian Bridge pages were signed and sealed by Mr. Deeb (sheets S01-01 through S01-25). Only the shape, look, and dimensions of the Arches were shown in the WEIS plans. No design was offered on these sheets of plans for the Arches. The engineering for the design was to *be completed by the fabricator*, and this was clearly indicated on the plans.
9. A 68-page document with Transmittal Cover Letter #13, dated February 26, 2020, from Ben Wilson of Dane Construction to Mike Wilson of Neill, containing the engineering design calculations and the Shop Drawings for the Iconic Arch, which were prepared by Western Wood Structures, Inc. (Western Wood). These calculations and Shop Drawings were prepared by or under the supervision of Paul C. Gilham, PE. Mr. Gilham signed and sealed all documents on December 20, 2019, and then revised unknown items and dated the document December 30, 2019.
10. A 21-page document with Transmittal Cover Letter #13 from Ben Wilson of Dane Construction to Mike Wilson of Neill, containing Shop Drawings for the Iconic Arch, dated February 26, 2020. These Shop Drawings had “accepted as noted” stamps on them from NC Dept. of Transportation, dated April 6, 2020.
11. News article by *Hickory Daily Record* dated February 18, 2022, by Walt Unks and Robert C. Reed.
12. The website of www.csiamerica.com (Computers and Structures, Inc.), the creators of the SAP2000 computer program utilized by Mr. Paul C. Gilham of Western Wood. Reviewed various user help videos which explains use, input, and results of the program.
13. CSi Analysis Reference Manual (569 pages), obtained from the “csiamerica” website.
14. The website of www.westernwoodstructures.com. This is the company which designed and fabricated the Iconic Arches.
15. StormIntel Weather Guidance report (obtained by Forcon) for February 17 and 18, 2022.
16. CoreLogic Weather Report (obtained by Forcon) for Jan. 1, 2009, through March 29, 2022.
17. WeatherUnderground Weather Report for February 18, 2022. (Obtained by Forcon)

FORCON INTERNATIONAL

18. Letter from the North Carolina Board of Examiners for Engineers and Surveyors dated February 22, 2022, signed by Mr. Andrew L. Ritter, addressed to Mr. Warren Wood, City of Hickory City Manager. This letter requested that the Board be notified if the investigation of the collapse revealed *gross negligence, incompetence, or misconduct by any of its licensees.*

As most of the cited reviewed documents are readily available to all parties, they are not attached to this report. However, the Weather Reports from StormIntel and CoreLogic are attached as Exhibit 1. The Weather Underground data is available and is free on the website www.weatherunderground.com, so a printout is not included. The other cited websites and the cited User Manual for the SAP2000 software are available to the public, online.

WEATHER CONDITIONS

The following sources were utilized to determine weather conditions at the time of the collapse; the address used in the queries was the Fire Station across the street, as the bridge does not have an address:

1. StormIntelligence Weather Guidance for February 17 and 18, 2022 revealed that there were no instances of high winds at the location on this date. This source defines high winds as being greater than or equal to 50 mph. They do not report on actual wind speeds below 50 mph. The report did, however, verify that there were no tornadoes or microbursts in the immediate vicinity of the Arches.
2. CoreLogic Weather Verification Services for the period of January 1, 2009, through March 29, 2022 revealed that there were no wind gusts above 40 mph on the date of the collapse. This report contains all wind events of 58 mph or greater within 3 miles of the location. Wind speeds being reported within the report represent 3-second wind gusts at 10 meters, starting at 40 mph and increasing in 1 mph increments. The last high winds experienced at this location occurred on August 31, 2021, so the completed structure had not had high wind loads on it prior to the collapse.
3. Weather Underground website revealed that the sustained wind speed at approximately midnight between February 17 and February 18, 2022, was 18 mph and there were wind gusts of 33 mph. Winds were from the south, which makes sense as the Arches collapsed toward the north.

ENGINEERING EVALUATION

The scope of this preliminary report is to try to ascertain the probable cause(s) of the collapse and to endeavor to discover the party or parties whose actions may have contributed to the collapse. This evaluation is based upon evidence produced to date; the investigation will be ongoing. If further evidence is found during the Discovery phase of this suit to alter Forcon's opinion, it will be presented in an updated report. At your request and since time is of the essence, Forcon agrees that it is important to share the professional opinions of Dara Thomas, PE at this juncture, rather than finish all Discovery before writing a report. Thus, Forcon is issuing a Preliminary Report.

Review of the documents began with the Contracts 01 through 13. These documents contain the responsibilities of the parties (such as Neill) and also the Specifications and Design Criteria for the pedestrian bridge and the Iconic Arches.

In Contract 2, page 27, section (A) it states:
The Contractor shall guarantee materials and workmanship against latent and patent defects arising from faulty materials, faulty workmanship, or negligence for a period of twelve months following the date of final acceptance of the work for maintenance and shall replace such defective materials and workmanship without cost to City of Hickory, NC. The Contractor will not be responsible for damage due to faulty design, normal wear and tear, for negligence on the part of City of Hickory, NC, and/or for use in excess of the design.

And on page 28:
This guarantee provision shall be invoked only for major components of work in which the Contractor would be *wholly responsible* for under the terms of the contract.

With regard to Neill, it is my understanding that Neill was not *wholly responsible* for the design or construction of the Iconic Arches. Rather, Western Wood was responsible for the Design, materials, and fabrication of the Arch components. This responsibility was clear in both the Specifications and on the Project Plans prepared by WEIS. Likewise, Neill was not *wholly responsible* for the erection of the components. This was performed by others.

Based upon my visual inspection and my review of the documents, plans and specifications, I have seen no evidence that the Arches were assembled/erected improperly. Therefore, I find no fault with Neill or any erection subcontractors downstream from Neill concerning the erection/construction. Rather, it was built according to the signed and seal shop drawings by Western Wood. Finding no construction defect with the assembly, my focus switched to the design of the Arches by Western Wood.

There are numerous facets to the structural design of a complex arched space frame, such as the Iconic Arches project. Each one will be discussed separately, in detail.

THE DESIGN CRITERIA

The Design Criteria specified in Specifications and Plans are enumerated in regular font below. Forcon's response is in **bold font.**

1. In Contract 7, Section 34 80 00, Item 1.4 (B) it says: Heavy Timber Arches shall be designed to meet the minimum load requirements of NCDOT Standard Specifications, AASHTO Guide Specifications for the design of pedestrian bridges and ASCE-7 whichever is most critical.

   There is no evidence that the arch space frame design was checked for all of those load requirements. Only the AASHTO LRFD was cited in the design calculations and the computer modeling of the space frame.

FORCON INTERNATIONAL

2. In Item 1.4 (E)(4) of that section it states: Wind Load: All bridges and heavy timber arches shall be designed for a minimum wind load of 35 pounds per square foot (psf). The wind is calculated on the entire vertical surface of the bridge as if fully enclosed. The wind shall be applied horizontally at right angles to the longitudinal axis of the structure.

   Since the wind loads are calculated within the SAP2000 finite element analysis program, we cannot confirm what the resultant design wind pressure was for the solid elements, but we do know that at least for the hand calculations for the guy wires (which were the sole wind-resisting components), Mr. Gilham stated a design wind pressure of 32.16 psf, which is lower than the required 35 psf minimum stated in the Contract Documents/Specs. We also know that the input for loads in the SAP2000 software only references the AASHTO design loads. There is no evidence that the NCDOT Standard Specs or the ASCE-7 standard were considered.

   Furthermore, rather than modeling the wind load against the side of the double arches as if it had the entire vertical surface area between the upper and lower arch as a solid surface, Mr. Gilham only modeled the upper arch in the SAP2000 software. Therefore, only the 24" tall vertical surface of the upper arch was loaded with horizontal wind forces. One cannot assume that by observation the lower arch would experience the same wind load. This approach would not be the same as if the entire double arch system had been modeled together as a space frame. And it certainly does not produce the proper result of modeling as if the space between the two arches was a solid vertical surface, as required in the Specifications. The modeling choices by Mr. Gilham will be discussed further in a later section.

3. In Item 1.4 (E) (9), which applies to both the pedestrian bridge and the timber arches: The horizontal deflection due to lateral wind loads shall not exceed 1/500 of the span length under the Design Service wind load.

   In the stated design criteria in the calculations by Western Wood, the horizontal deflection criteria stated for wind loads against the face of the arch space frame is 1/120 of the span length.

   Deflection criteria of 1/500 of the span length (L) results in a numerical value in inches. The approximate allowable horizontal deflection for L/500 is +/- 4.9". The approximate value of L/120, per Western Wood, is +/- 20.5". It must be understood that these limits are the result of full design loads of a hurricane event, which did not happen on the night of the collapse. But one can see that the Western Wood design would allow for catastrophic deflection under full load. This is because the dead load of the timber arches being 20.5" from the center of gravity would produce a P-Delta effect which would likely buckle and fail the space frame. This is because the enormous dead weight being that far from the center of gravity would produce a moment (twisting) force that would be additive to the anticipated wind pressure. This will be discussed further in a later section.

4. On Sheet S01-01 of the Project Plans prepared by WEIS it states: Wood Arch foundation connection to be designed and engineered by vendor/fabricator (meaning Western Wood) to meet Specification requirements as provided in Contract Documents. In Contract 7, Item #11 it states: Unless specified otherwise, the bridge manufacturer shall determine

FORCON INTERNATIONAL

the number, diameter, minimum grade, and finish of all anchor bolts. The anchor bolts shall be designed to *resist all horizontal* and uplift forces to be transferred by the superstructure to the supporting foundations.

Foundation connections include the base plates and anchor bolts of the "legs" of the arches into the large concrete bulkheads and also includes the attachment of the guy wire anchors into the concrete bents below the pedestrian bridge. The approach used by Western Wood for both of these anchor points will be discussed as part of the discussion on structural analysis.

5. **Regarding the cable connector steel bracket S15 (shown in the Western Wood shop drawings), which pulled apart at a welded joint, I could not locate any calculations of the required weld size or length. There is no indication that the welds were designed at all for the reaction forces.**

DISCUSSION OF STRUCTURAL ENGINEERING MODELING APPROACH

The Engineer-of-Record (EOR) at Western Wood used the finite element analysis computer program known as the SAP2000. He also employed what he called a proprietary program called SAPSTRESS to design some (not all) of the steel connections. We have no way to inspect or learn about the capabilities of SAPSTRESS, but there are copious amounts of information on the SAP2000 tool. I found that there are four levels of the SAP2000 with varying price points. The BASIC costs $2,000; PLUS costs $5,000; Advanced costs $8,000; Ultimate costs $12,000. This is for the current version 24. The version used by Western Wood is stated to be version 21.0.2. It is not known which level that equates to. Comparison between the two versions is not available online at this point in time. Therefore, I called the manufacturer of the SAP2000 software. The technical support person I spoke with said that the versions would have the same capabilities. The only things that change with the updated versions are items driven by Code updates. The analysis capability should be the same.

As previously mentioned, the EOR from Western Wood made the choice to only model the upper arch. Some of the inherent flaws created by doing this were previously mentioned in the Design Criteria discussion above. But there are other things set in motion by doing this. Additionally, the EOR modeled the two (not four) cables as solid rod elements, rather than cables. This allows the cable to be assigned not just a tension load on the windward side, but a compression load on the leeward side. This creates a false (imaginery) load resistance in the leeward cable. It is unknown why this choice was made, when the SAP2000 program does have the ability to include cables in the analysis.

To proceed further, we must understand some basic concepts. There are six *degrees of freedom* for all points (nodes) in space. And *elements* are the solid components between nodes. In our case the GluLam timbers and the guy wires are the elements. Every joint/connection, base plate and guy wire attachment is a node in the Arch Space Frame. All loads and masses applied to the elements are transferred to the joints. Joints are the primary locations in the structure at which the displacements are known or are to be determined. Every joint/node is capable of moving/displacing in the x, y, and z axis. The x axis is generally in the direction of the framing member of the structure. In other words, the x axis is along the length of the arch timbers. The y axis is perpendicular to the x axis, so at 90 degrees horizontal from the length of the arch timbers. The z axis is up and down (vertical) from the node. So those are the first three degrees of freedom. The other three degrees of freedom are described as *rotation about those three axes*. Rotation is similar to twisting about the node. Each degree of freedom in the structural model must be one of the following types:

- Active (the displacement is computed during the analysis)
- Restrained (the displacement is specified, and the corresponding reaction is computed during the analysis)
- Constrained (the displacement is determined from the displacements at other degrees of freedom)
- Null (the displacement does not affect the structure and is ignored by the analysis)
- Unavailable (the displacement has been explicitly excluded from the analysis)

Here is a diagram of the six degrees of freedom (snipped from the CSi Analysis Reference Manual):

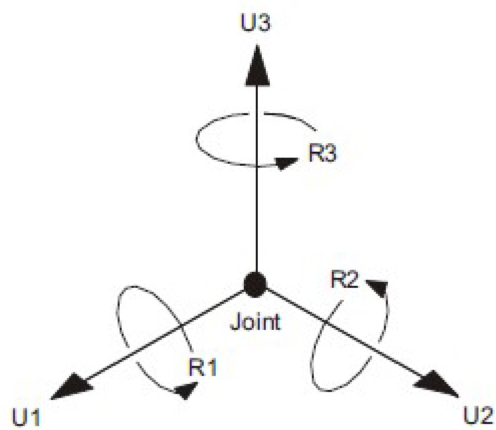


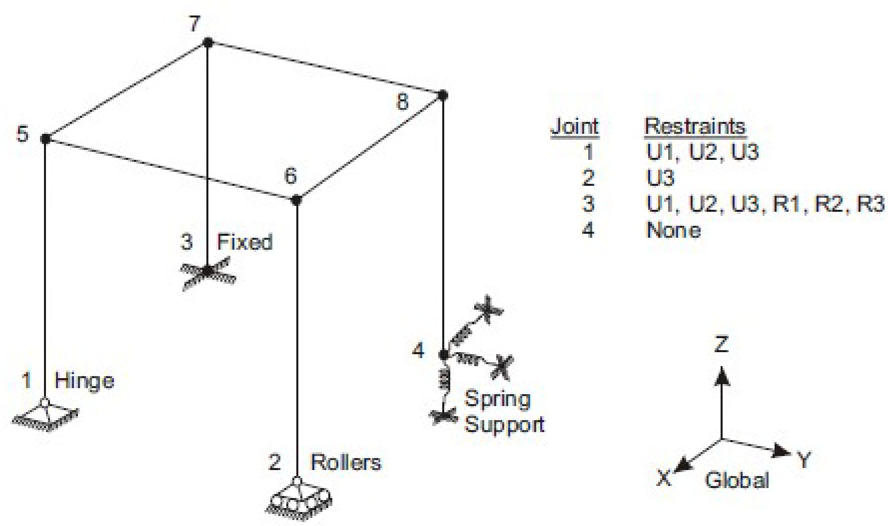
Figure 5
The Six Displacement Degrees of Freedom in the Joint Local Coordinate System

When designing a free-standing space frame such as the Iconic Arches, one must develop a game plan as to how to guarantee stability and resistance to such things as horizontal wind load. Another thing to consider and guard against is lateral torsional buckling out of plane and the P-Delta effect. These things are important to account for, as a structure such as this does not have any inherent *lateral bracing* like a regular building does. In a building, there are roof levels and floor levels that can provide bracing of the main wind resisting components against horizontal wind forces. There are also walls that can be designed as *shear walls* to resist lateral wind loads and lateral torsional buckling of framing members. In our case, there is no inherent bracing

FORCON INTERNATIONAL

available. So, *bracing* must be created by using certain elements to resist wind load and to prevent lateral torsional buckling and the P-Delta effect.

An obvious choice to utilize available components for bracing is to have designed all eight base plate – foundation connections to resist lateral load. In order to do this, the joints at these locations would have to intentionally be designed in the computer model as a *fixed connection* rather than a *pinned connection.* A fixed connection would prevent displacement in all six degrees of freedom. Whereas the Western Wood computer model allowed rotation about the base plate nodes (marked the three rotational degrees of freedom as *unrestrained instead of restrained).* To do this, a moment-connection would have to be accomplished. This would likely mean a heavy-duty steel sleeve around the GluLam timbers with multiple large diameter anchor bolts into the concrete. The connection at the base plate nodes would have to eliminate not only movement side-to-side but eliminate any rotation of the structure around the connection. We know that the base plate connections were not fixed connections because of the choices made in the computer program. Following is a diagram depicting a fixed node vs. a pinned (hinged) node:



3-D Frame Structure

As we can see from the diagram, in order to create a *fixed* node (node 3 in the diagram) in the frame analysis computer model, we must restrain all six degrees of freedom. And we can see in the calculations from Western Wood that the EOR of the timber structure only restrained the x, y, and z movements (U1, U2, U3). The rotation degrees of freedom (R1, R2, R3) were not restrained in the computer model by Western Wood. This means that the eight base plates were allowed to freely pivot/rotate once the arches began to fall over. They offered no assistance in resisting the collapse.

Next, we must consider buckling and P-Delta effect decisions in the design process. Lateral torsional buckling is when a framing member starts to roll *out of plane* under loading, because

FORCON INTERNATIONAL

the side of the member is not braced laterally at a uniform distance. Picture a long slender wood roof truss with a load on the top chord. If the truss is not braced with plywood sheathing and with rows of lateral bracing, it can easily start to buckle out of plane and roll over and fail catastrophically. A technical definition of the P-Delta effect is … *a second order effect experienced by any structure when subjected to lateral loads like wind and is originated by an additional destabilizing moment generated due to the gravity load acting on laterally deflected member further displacing it*. As previously mentioned on page 8, the enormous dead weight of the timbers would create a large additional *moment* (rotation) force if the arch was allowed to deflect horizontally too much. We know that the wind gusts were on the order of only 33 mph at the time of the collapse. We know that the sustained wind speed was 18 mph. So even though those wind speeds are nowhere near the hurricane wind loads that could be encountered, it may have been enough to displace all that weight to the north and cause rotation forces to kick in. This makes perfect sense when we consider that the nodes at joints 4 and 22 showed the first sign of rotation out of plane in the security videos. Those joints just would not be able to resist the additional moment applied (rotation force to the north), because they were not designed for forces from the P-Delta effect. Below is a diagram snipped from the Western Wood computer model, to assist the reader in understanding the node locations. Also, note that only the upper arch was modeled, not the entire structure.

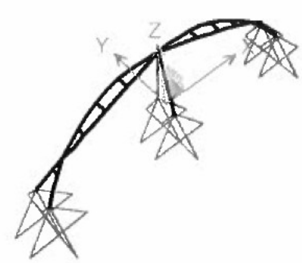


Figure 1: Finite element model

1.1. Joint coordinates

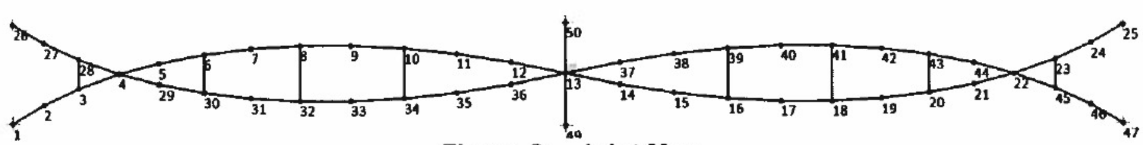


Figure 2: Joint Map

There is evidence to suggest that the guy cables were over-stressed, as one of them was nearly pulled apart due to tension. But before it snapped completely, the connector Mark S15 came apart at the welds. This cable situation most likely also suffered due to the P-Delta effect. The dead weight causing additional twisting load to the north likely overcame the cable and connection design and it failed. Once that happened, lateral forces were then transferred to nodes 4 and 22. Once those connections began to come apart, the twisting load to the north was transferred to the base plates, which were not designed to resist such twisting moment. We know this because they were defined in the computer model analysis as *pinned connections*.

FORCON INTERNATIONAL

The reason that nodes 4 and 22 and then the base plate nodes failed is because they were not designed to resist the complete wind force on the structure. The only wind-resistant components were the cables.

As mentioned earlier, there were no calculations provided for welds of the cable attachment piece. Even if they had been designed properly for loads occurring, with the entire structure having no node displacement allowed in the SAP2000 program, if the P-Delta effect started to influence stresses in the structure, the program would not have included those effects. In essence P-Delta effects were ignored. There is no way any of the structure, as designed, could resist the additional loads due to the dead weight shifting out of plane. The SAP2000 program can be run allowing displacement of nodes and allowing the P-Delta effect to play out. There is no evidence in the calculations that the computer model was run for the P-Delta effect.

Additionally, the model created in the program by Western Wood only modeled the upper arch. It did not model the entire space frame, as one. Even though the upper and lower arch essentially only touched each other at one place (the apex where the cables interact), the structure certainly acted as one structure during the failure, so they should have been analyzed together. So even if the P-Delta effect had been opted for in the computer output for the single arch, it would have resulted in invalid results.

Forcon has not performed a full design of the Arches using SAP2000 or any other tool. That is outside the scope of this assignment. But there are enough errors found and poor design choices made for the author to state her professional opinion that the collapse was due to inadequate structural design.

CONCLUSIONS

Based upon my site observations and investigation, documents reviewed, education, experience, and engineering evaluation, with consideration of the American Society of Testing Materials (ASTM) Standards E2713-18 and E678-07(Reapproved 2013), it is my professional opinion that:

- **The Iconic Arches collapsed due to some, or all of the Western Wood design flaws identified.**
- **Forcon will state for transparency purposes that Forcon has filed a Complaint with the North Carolina Board of Examiners for Engineers and Surveyors (NCBELS) against the Western Wood EOR, Mr. Paul C. Gilham, PE (NC PE # 024262). This Complaint was filed with NCBELS on or about April 15, 2022. NCBELS will be performing their own private and confidential investigation of this matter.**
- **There is no evidence to suggest that Neill Grading and Construction Company, Inc. performed any work, or failed to perform any work, in a manner that caused or contributed to the Iconic Arches collapse.**

FORCON INTERNATIONAL

LIMITATIONS

The contents of this Preliminary Report are intended for the use of **McAngus Goudelock & Courie PLLC** and its designated representatives and are not intended for any other purpose. FORCON assumes no liability for the misuse of this information by others. The observations, comments, conclusions, analysis, and opinions expressed herein are based upon the results and interpretations of the testing and/or data collection activities performed at the time of the inspection, and the best information provided to me, and have been provided to a reasonable degree of engineering certainty. FORCON reserves the right to update/revise the observations, opinions and/or the recommendation in this report should conditions change, or additional information become available.

A digital signature has been used to sign and seal this document and to electronically transmit this document. If not digitally signed and sealed by the licensed professional named and shown as its author, the findings, conclusions, and opinions cannot be relied upon; as such, the document has been provided for information purposes only. Printed copies of this document are not considered signed and sealed and the signature must be verified on electronic copies.

FORCON appreciates this opportunity to have assisted **McAngus Goudelock & Courie PLLC** with this investigation. We trust that this information fulfills the requested scope of services in this matter, and if we can be of further support, please do not hesitate to contact us.

Regards,

NORTH CAROLINA
PROFESSIONAL
SEAL
28783
ENGINEER
DARA THOMAS

Dara Thomas

Digitally signed by Dara Thomas
Date: 2022.05.23 13:59:36 -04'00'

Dara Thomas, P.E.
Professional Engineer
North Carolina PE License Number 028783

FORCON INTERNATIONAL

Exhibit 1
Weather Reports

FORCON INTERNATIONAL

WeatherGuidance
StormIntelligence®

Claim/Reference #:	C00312609	
Insured/Carrier Name:	Neill Grading	
Claimed Date of Loss:	N/A	
Search Period:	02/17/2022 - 02/18/2022	
Insured Location:	19 Second Street Drive NE, Hickory, NC, 28601	
Latitude: 35.7328231	**Longitude:** -81.3348275	**County/Parish:** CATAWBA

We have reviewed radar data, ground reports and other data for the stated period and location. As a result, we have identified the following verifiable instances of winds >= 50 mph at that location:

•We were unable to identify any verifiable instances of high winds at the insured location during the requested search period.

Other Requested Information:

We examined ground-based storm reports as well as weather radar and other data and could not verify the occurrence of a tornado at or near the insured location during the requested search period.

No microbursts occurred in the immediate vicinity of the insured location during the requested search period.

I hereby attest that the above information is true and correct to the best of my knowledge and professional opinion as indicated by my signature below on this 21st day of March, 2022.

Robert C. White • President/Sr. Forensic Weather Analyst • www.weatherguidance.com • (512) 504-3151 x5

FORCON INTERNATIONAL

CoreLogic

Weather Verification Services

Wind Verification Report

Claim or Reference #	C00312609
Insured/Property Owner	Neill Constr
Address	19 2nd St Dr NE Hickory, NC 28601
Coordinates	Latitude 35.732823, Longitude -81.334828
Date Range	Jan 01, 2009 to Mar 29, 2022
Report Generated	March 30th, 2022 at 17:27:01 UTC

Storm Events

	Estimated Maximum Windspeed			
Date	**At Location**	**Within 1 Miles**	**Within 3 Miles**	**Within 10 Miles**
Aug 31, 2021	58 MPH	58 MPH	58 MPH	58 MPH
Aug 17, 2021	57 MPH	63 MPH	66 MPH	67 MPH
Aug 14, 2021	51 MPH	53 MPH	59 MPH	67 MPH
Jul 26, 2021	60 MPH	61 MPH	62 MPH	62 MPH
Aug 2, 2020	52 MPH	57 MPH	63 MPH	63 MPH
Jul 31, 2020	64 MPH	65 MPH	67 MPH	67 MPH
Jul 18, 2020	64 MPH	64 MPH	65 MPH	65 MPH
Jun 21, 2020	52 MPH	53 MPH	60 MPH	61 MPH
Apr 12, 2020	59 MPH	64 MPH	65 MPH	65 MPH
Feb 6, 2020	50 MPH	53 MPH	58 MPH	64 MPH
Jan 10, 2020	46 MPH	51 MPH	59 MPH	63 MPH
Aug 22, 2019	59 MPH	61 MPH	63 MPH	63 MPH
Aug 21, 2019	66 MPH	67 MPH	68 MPH	70 MPH
Aug 13, 2019	58 MPH	60 MPH	63 MPH	65 MPH
Aug 7, 2019	58 MPH	59 MPH	62 MPH	62 MPH
Jul 11, 2019	64 MPH	66 MPH	66 MPH	68 MPH

FORCON INTERNATIONAL

	Estimated Maximum Windspeed			
Date	**At Location**	**Within 1 Miles**	**Within 3 Miles**	**Within 10 Miles**
Apr 26, 2019	66 MPH	67 MPH	67 MPH	67 MPH
Oct 11, 2018	54 MPH	58 MPH	62 MPH	63 MPH
Sep 27, 2018	53 MPH	55 MPH	59 MPH	60 MPH
Aug 31, 2018	54 MPH	57 MPH	61 MPH	64 MPH
Aug 6, 2018	58 MPH	60 MPH	62 MPH	63 MPH
Jun 25, 2018	65 MPH	66 MPH	66 MPH	66 MPH
Oct 23, 2017	68 MPH	70 MPH	74 MPH	74 MPH
Jul 27, 2017	50 MPH	51 MPH	58 MPH	61 MPH
Jul 23, 2017	60 MPH	61 MPH	62 MPH	66 MPH
Jul 17, 2017	53 MPH	56 MPH	61 MPH	63 MPH
Jul 15, 2017	63 MPH	65 MPH	67 MPH	68 MPH
Jul 5, 2017	65 MPH	66 MPH	67 MPH	69 MPH
Jun 13, 2017	64 MPH	67 MPH	71 MPH	71 MPH
Apr 3, 2017	57 MPH	60 MPH	62 MPH	62 MPH
Mar 1, 2017	64 MPH	65 MPH	66 MPH	66 MPH
Feb 9, 2017	67 MPH	68 MPH	69 MPH	69 MPH
Nov 30, 2016	61 MPH	62 MPH	64 MPH	64 MPH
Jul 16, 2016	65 MPH	66 MPH	67 MPH	67 MPH
Jul 8, 2016	58 MPH	61 MPH	63 MPH	68 MPH
Jul 5, 2016	60 MPH	62 MPH	66 MPH	66 MPH
Jun 29, 2016	55 MPH	59 MPH	64 MPH	66 MPH
Feb 24, 2016	66 MPH	67 MPH	67 MPH	67 MPH
Aug 14, 2015	58 MPH	58 MPH	58 MPH	60 MPH
Jul 21, 2015	46 MPH	53 MPH	59 MPH	66 MPH
Jul 13, 2015	57 MPH	60 MPH	61 MPH	61 MPH
Jun 24, 2015	57 MPH	61 MPH	63 MPH	63 MPH
Jun 22, 2015	60 MPH	62 MPH	63 MPH	63 MPH
Feb 14, 2015	56 MPH	57 MPH	60 MPH	67 MPH
Sep 6, 2014	62 MPH	65 MPH	66 MPH	66 MPH
Jul 24, 2014	57 MPH	59 MPH	62 MPH	69 MPH
Jun 20, 2014	55 MPH	58 MPH	60 MPH	60 MPH
Jun 19, 2014	58 MPH	60 MPH	62 MPH	66 MPH
Jun 8, 2014	50 MPH	55 MPH	60 MPH	61 MPH
May 27, 2014	59 MPH	61 MPH	63 MPH	63 MPH
Dec 17, 2013	61 MPH	62 MPH	64 MPH	64 MPH

FORCON INTERNATIONAL

	Estimated Maximum Windspeed			
Date	At Location	Within 1 Miles	Within 3 Miles	Within 10 Miles
Sep 1, 2013	62 MPH	63 MPH	64 MPH	64 MPH
Aug 12, 2013	62 MPH	62 MPH	62 MPH	62 MPH
Jul 17, 2013	59 MPH	60 MPH	64 MPH	65 MPH
Jul 9, 2013	61 MPH	62 MPH	63 MPH	64 MPH
Jun 26, 2013	58 MPH	61 MPH	65 MPH	70 MPH
Jun 13, 2013	62 MPH	64 MPH	67 MPH	67 MPH
Sep 1, 2012	57 MPH	59 MPH	62 MPH	63 MPH
Aug 1, 2012	60 MPH	62 MPH	64 MPH	66 MPH
Jul 5, 2012	51 MPH	54 MPH	63 MPH	64 MPH
Jul 1, 2012	61 MPH	61 MPH	62 MPH	63 MPH
Jan 11, 2012	61 MPH	62 MPH	66 MPH	67 MPH
Sep 2, 2011	60 MPH	61 MPH	61 MPH	61 MPH
Aug 19, 2011	71 MPH	73 MPH	74 MPH	74 MPH
Aug 14, 2011	61 MPH	62 MPH	62 MPH	63 MPH
Aug 13, 2011	57 MPH	58 MPH	59 MPH	64 MPH
Jul 7, 2011	66 MPH	66 MPH	67 MPH	67 MPH
Jul 4, 2011	61 MPH	63 MPH	65 MPH	67 MPH
Jun 28, 2011	60 MPH	62 MPH	63 MPH	63 MPH
Jun 22, 2011	66 MPH	68 MPH	69 MPH	69 MPH
Jun 18, 2011	65 MPH	66 MPH	66 MPH	66 MPH
Jun 11, 2011	62 MPH	64 MPH	65 MPH	65 MPH
Jun 9, 2011	67 MPH	68 MPH	68 MPH	68 MPH
May 22, 2011	61 MPH	62 MPH	63 MPH	67 MPH
May 3, 2011	60 MPH	64 MPH	66 MPH	66 MPH
Feb 28, 2011	60 MPH	64 MPH	68 MPH	73 MPH
Aug 5, 2010	62 MPH	64 MPH	66 MPH	68 MPH
Jul 18, 2010	49 MPH	52 MPH	59 MPH	65 MPH
Jun 24, 2010	54 MPH	55 MPH	58 MPH	62 MPH
Jun 15, 2010	70 MPH	71 MPH	71 MPH	71 MPH
May 28, 2010	55 MPH	57 MPH	59 MPH	59 MPH
May 16, 2010	53 MPH	58 MPH	63 MPH	67 MPH
Aug 5, 2009	61 MPH	63 MPH	65 MPH	67 MPH
May 9, 2009	61 MPH	62 MPH	62 MPH	62 MPH

• Wind dates begin at 6am CST on the indicated day and end at 6am CST the following day.
• Dash "--" indicates 58 MPH or higher wind was detected within 3 miles, but winds at location were less than 40 MPH.

FORCON INTERNATIONAL

- Report contains all wind events where winds of 58 MPH or greater were detected within 3 miles of the location.
- Wind speeds being reported within this report represent 3-second wind gusts at 10 meters; starting at 40 MPH and increasing in 1 MPH increments.
- This report contains wind events between Jan 01, 2009 and Mar 29, 2022.

FORCON INTERNATIONAL

Important Legal Notice and Disclaimer

THIS REPORT AND THE CORRESPONDING DATA IS PROVIDED FOR INFORMATIONAL PURPOSES ONLY AND ONLY FOR INTERNAL USE BY THE INITIAL PURCHASER/END-USER RECIPIENT HEREOF. NOTWITHSTANDING THE FOREGOING, SUCH PURCHASER/END-USER MAY SHARE THIS REPORT WITH THE OWNER OF THE PROPERTY THAT IS THE SUBJECT OF THE REPORT. CORELOGIC, INC. AND/OR ITS SUBSIDIARIES OR AFFILIATES (“CORELOGIC”) DO NOT ASSUME, AND EXPRESSLY DISCLAIM LIABILITY FOR ANY LOSS OR DAMAGE RESULTING FROM THE USE OF, OR ANY DECISIONS BASED ON OR IN RELIANCE ON, THIS REPORT AND THE CORRESPONDING DATA

THIS REPORT AND THE CORRESPONDING DATA ARE FURNISHED "AS IS", WITHOUT WARRANTY OF ANY KIND, EXPRESS OR IMPLIED, INCLUDING BUT NOT LIMITED TO, ANY WARRANTIES OF MERCHANTABILITY, ACCURACY, COMPLETENESS, NON-INFRINGEMENT, OR FITNESS FOR A PARTICULAR PURPOSE (EVEN IF THAT PURPOSE IS KNOWN TO CORELOGIC). SOME STATES DO NOT ALLOW THE EXCLUSION OF IMPLIED WARRANTIES, SO THE ABOVE EXCLUSIONS MAY NOT APPLY TO THE USER. IN THAT EVENT, ANY IMPLIED WARRANTIES ARE LIMITED IN DURATION TO NINETY (90) DAYS FROM THE DATE OF THE REPORT AND TO THE DOLLAR AMOUNT OF THE REPORT.

THE CONTENTS OF THIS REPORT ARE PROTECTED BY APPLICABLE COPYRIGHT AND OTHER INTELLECTUAL PROPERTY LAWS AND MAY INCLUDE PROPRIETARY OR OTHER CONTENT OF THIRD PERSONS. NO PERMISSION IS GRANTED TO COPY, DISTRIBUTE, MODIFY, POST OR FRAME ANY TEXT, GRAPHICS, OR USER INTERFACE DESIGN OR LOGOS INCLUDED IN THIS REPORT. ALL USERS OF THIS REPORT AGREE TO HOLD HARMLESS CORELOGIC AND ITS AFFILIATES, EQUITY HOLDERS, DIRECTORS, OFFICERS, EMPLOYEES, CONTRACTORS, AGENTS, REPRESENTATIVES AND SUPPLIERS OF THIRD PARTY CONTENT FROM ANY USE OR MISUSE, MISAPPLICATION, ALTERATION OR UNAUTHORIZED DISCLOSURE OF THIS REPORT.

THIS NOTICE AND DISCLAIMER IS INTENDED TO FUNCTION IN HARMONY WITH THE CORELOGIC TERMS OF USE AVAILABLE ON THE WEATHER VERIFICATION SERVICES WEBSITE. IN THE EVENT OF ANY CONFLICT BETWEEN THE TWO DOCUMENTS, THE CORELOGIC TERMS OF USE SHALL GOVERN.

Scan to Verify Authenticity

The original contents of this report are stored online in the original state provided by CoreLogic. Scan this QR Code and it will take you to a link to view the data and verify its authenticity.

FORCON INTERNATIONAL

Photograph 1) – Overview of collapse taken from west end of Main Avenue bridge.

Photograph 2) – Northwest foundations for the arch base connections.

FORCON INTERNATIONAL

Photograph 3) – Closer view of the NW base connection of the upper arch.

Photograph 4) – Closer view of Photo 3.

Photograph 5) – Grout below base plate at NW base connection is 3" thick. There were also steel bearing plates within. Each base plate had four ¾" diameter anchor bolts.

Photograph 6) – The steel base connection.

Photograph 7) – The lower arch base connection at the east end. The steel "shoe" pulled off of the base plate. The GluLam member splintered.

Photograph 8) – The upper arch base connection at the east end. The steel "shoe" pulled off of the base plate and base plate yielded.

FORCON INTERNATIONAL

Photograph 9) – Closer view of Photo 8.

Photograph 10) – View of opposite face of Photo 8.

FORCON INTERNATIONAL

Photograph 11) – Southwest base connection. Anchor bolts pulled out.

Photograph 12) – Example connection of the double guy wires on the north face.

FORCON INTERNATIONAL

Photograph 13) – Cable connector on the left appears to be part of Steel Weld Assembly Mark S15. The one on the right is an intact steel cable bracket.

Photograph 14) – Circled in red is part of Mark S15. Welds failed and this part detached from the part shown in Photo 13.

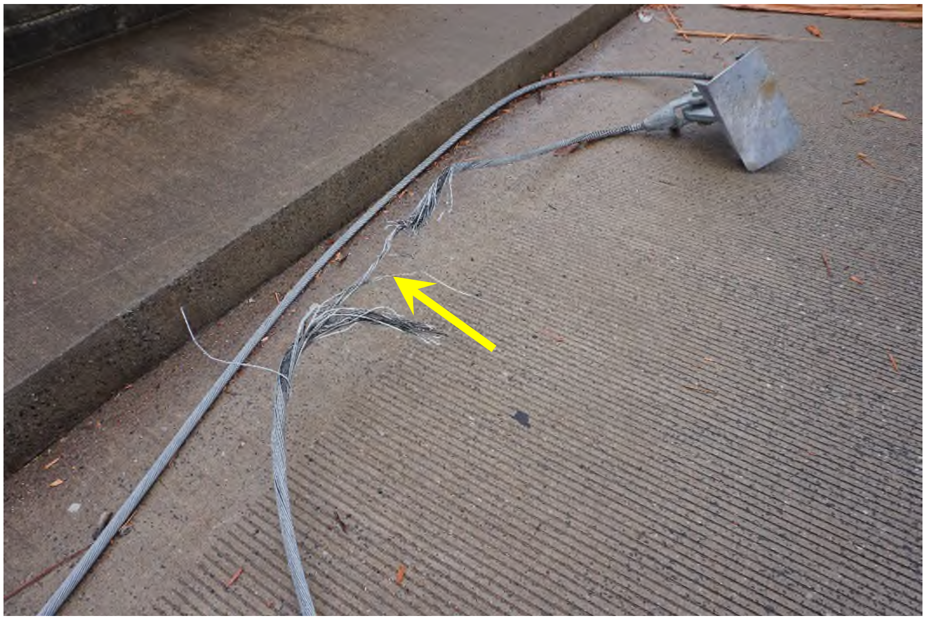
Photograph 15) – View of the intact steel cable bracket, which partially detached from the connection.

Photograph 16) –The intact cable bracket.

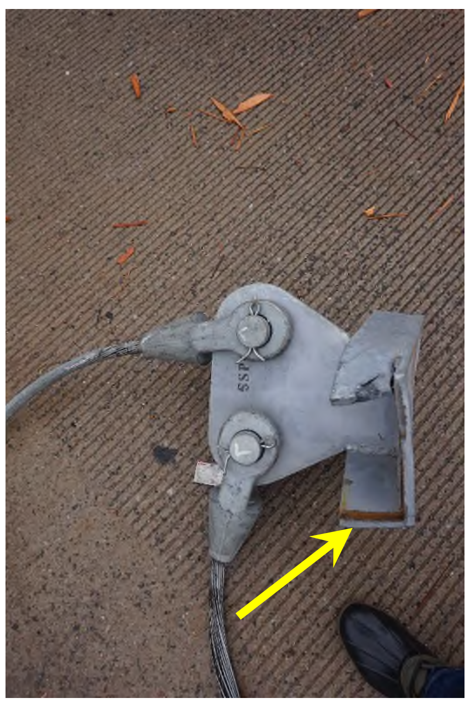
FORCON INTERNATIONAL

Photograph 17) – Another view of the intact cable bracket.

Photograph 18) – Part S15. This would have been connected to the lower part of the arch. It pulled apart from the piece shown in Photo 14.



Photograph 19) – The 7/8" diameter cable that connected to the lower arch with Part S15, was nearly shredded from tensile force. The adjacent cable which was unharmed as it went slack during the failure.



Photograph 20) – Part S15. The piece circled in Photo 14 detached from the end of this Part.

Photograph 21) – Another view of S15. The part in Photo 14 detached from this end.

Photograph 22) – The anchorage of the two cables on the north face.

FORCON INTERNATIONAL

Photograph 23) – Overview of the top node of the arch. The upper and lower arch connected at node 13. The two sets of dual cables connected to the bottom arch and also to the upper arch.

Photograph 24) – The connection of node 4 (or 22 which is identical). From the security videos we see that the arch began to pivot and fall over at this node.

FORCON INTERNATIONAL

Photograph 25) – View of the lower arch.

Photograph 26) – The connection at the apex.

Photograph 27) – The connection at the apex.

Photograph 28) – Part of the connection at the apex.

Photograph 29) – Failure of the kerf plate at a connection.

FORCON INTERNATIONAL

Photograph 30) – Damage to the concrete guardrail system on the Main Avenue bridge.

Photograph 31) – Debris landed below on NC Hwy 127.