

# Investigation of the November 13 and 14, 2014 collapses of two pedestrian bridges under construction at Wake Technical Community College Campus, Raleigh, NC

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U.S. Department of Labor  
Occupational Safety and Health Administration  
Directorate of Construction

April 2015



# Report

## **Investigation of the November 13 and 14, 2014 collapses of two pedestrian bridges under construction at Wake Technical Community College Campus, Raleigh, NC**

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## **Introduction**

On November 13, 2014 at approximately 10:30 a.m., a pedestrian bridge under construction at the Wake Technical Community College at the Northern Wake Campus, Raleigh, North Carolina, suddenly collapsed killing a worker. Four employees were injured. At the time of the collapse, concrete was being poured on the metal deck to provide the walking surface. A few hours later, another similar bridge, also under construction, within a block of the first incident, collapsed at approximately 12:30 a.m. in the middle of the following night. No one was injured in the second incident. No construction activities were going on during the second incident. Both bridges were part of the expansion of the Northern Wake campus which was well underway. consisting of several new buildings, parking garage, etc. The pedestrian bridges consisted of glued laminated timber girders and trusses to blend with the trees and their canopies in environmentally sensitive surroundings.

The North Carolina Department of Labor contacted the Directorate of Construction (DOC) in the OSHA National Office, Washington, DC to provide engineering assistance in determining the cause of the collapse, and to evaluate whether OSHA and industry standards had been violated. A structural engineer from DOC visited the site on December 2, 2014 and February 4, 2015 to examine the failed bridges, obtain documents, interview key personnel, take photographs, and take necessary measurements.

The DOC investigation included review of structural drawings, computations performed by the engineer of record, erection method for the trusses, glued laminated shop drawings, structural steel shop drawings, wire rope shop drawings and performing necessary calculations to determine the cause of the collapse.

## The Project

Wake Community College retained Clark Nexson ,Inc. as Architect; Skanska USA Building, Inc. as construction manager; and Falcon Engineering, Inc. as special inspector for the expansion project. Clark Nixson then retained a team of engineers including Stewart, Inc. as structural engineer of record. Skanska, through a bidding process retained a number of sub-contractors.

## Participants of the project

1. Owner: Wake Technical Community College (A state agency of the State of North Carolina)
2. Architect: Clark Nexson Inc., Raleigh, NC (Clark Nexson)
3. Structural Engineer: Stewart Inc., Raleigh NC (Stewart)
4. Construction Manager: Skanska USA Building Inc., Raleigh, NC (Skanska)
5. Steel and truss erector: Buckner Companies, Graham, NC (Buckner)
6. Steel Fabricator: North State Steel Inc., Greenville, NC (North State)
7. Glued Laminated Timber Fabricator: Structurlam Products LP., Penticton, B.C., Canada (Structurlam)
8. Special Inspector: Falcon Engineering Inc., Raleigh, NC (Falcon)
9. Truss Erection Engineer: Kimley-Horn and Associates Inc., Raleigh, NC (Kimley-Horn)
10. Concrete Contractor: Central Concrete of North Carolina Inc., Raleigh, NC (Central Concrete)
11. Concrete Contractor: J O Concrete Services, Inc., Raleigh, NC (JO Concrete)
12. Concrete Contractor: Lithko Contracting Inc., Raleigh, NC (Lithko)
13. Suspension Cable manufacturer: Pfeifer, Germany

The following is the organization chart for the construction project:

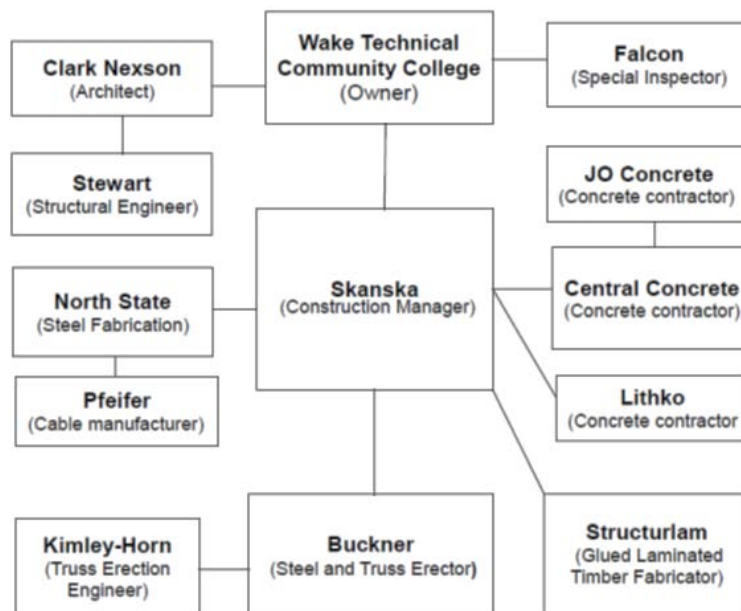


Fig. 1 – Organization chart

## Description of the Project

The Wake Technical Community College embarked to expand its north Raleigh campus by constructing several buildings, adding parking garages and other facilities. This report will concentrate to the two pedestrian bridges that failed. Two pedestrian bridges were envisioned to provide passage from Building F to the parking garage and to the future building L. As the bridges passed through ravines, wetlands and trees, the design of the bridges was based upon minimum interference with natural surroundings and without any environmental degradation. Therefore, four cable suspended trusses were designed to span 140 feet and 130 feet to bridge over the sensitive areas. The bridges were identified as Bridge No. 1 and 2 with identical designs, but with varying spans.

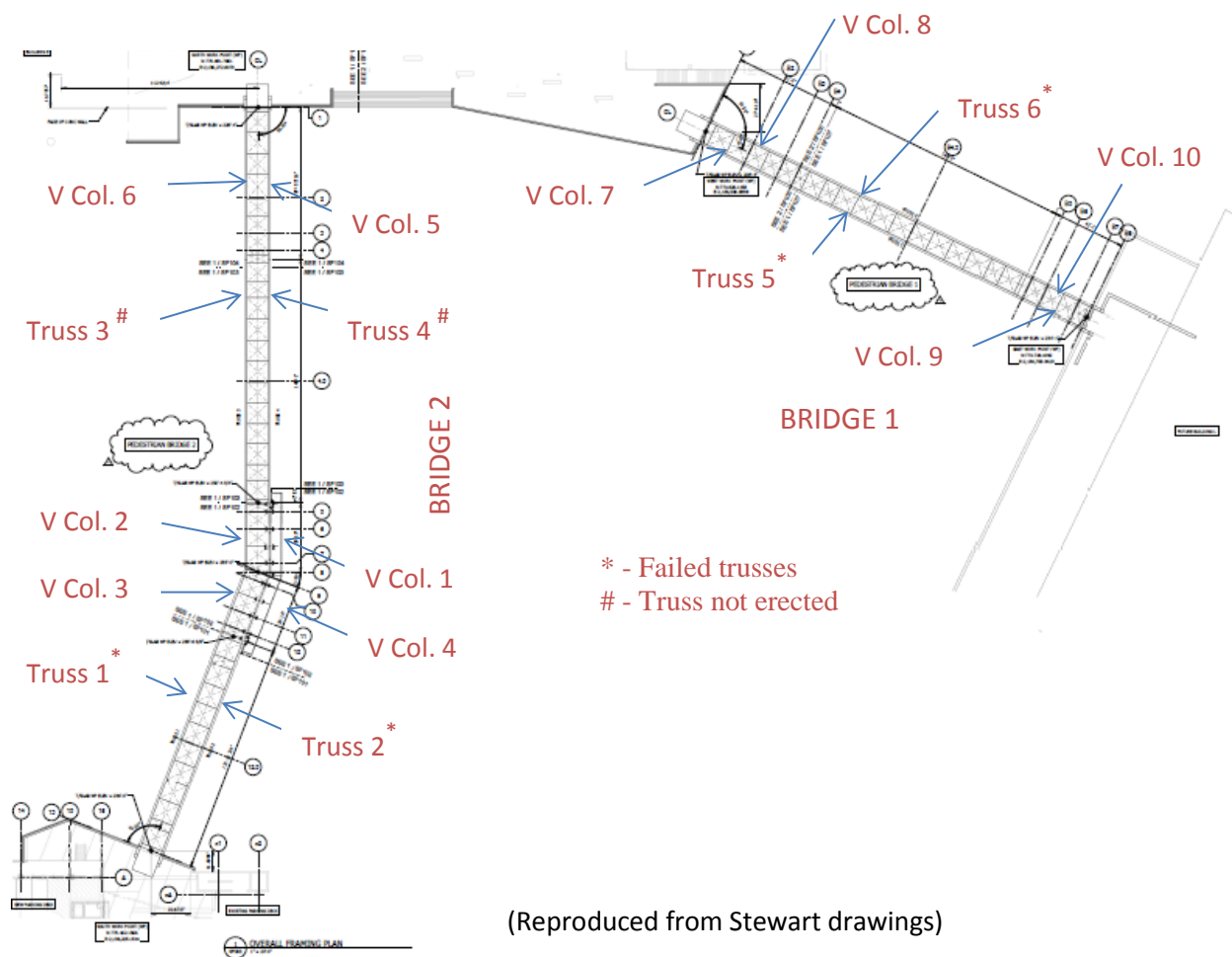
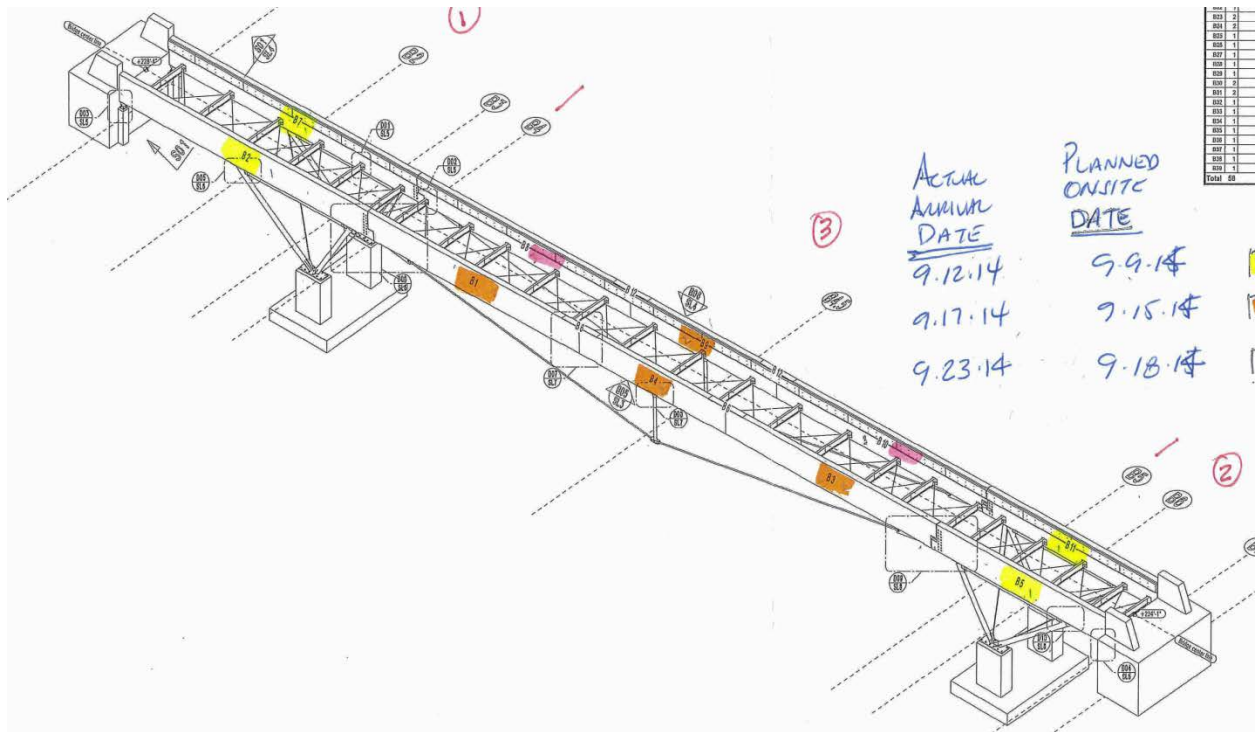


Fig. 2 – Location plan of Bridges 1 and 2



Bridge No. 1 was approximately 245 ft. long with a suspended center span of 140 ft. and with two end supported spans of 62 ft. and 42 ft. The end spans were supported over V-columns. Bridge No. 2 was skewed and had two suspended spans with end and middle spans being supported over V-columns. The total length of the bridge No. 2 was approximately 440 ft. Both bridges were through girder bridges consisting of two 13 1/2" by 60" deep glued laminated (glu-lam) beams with wainscot also of glu-lam. The suspended span had one 6" dia. steel pipe king post at the center supported over wire cables connected at each end of the center span. The clear width of the bridge was 12 ft.



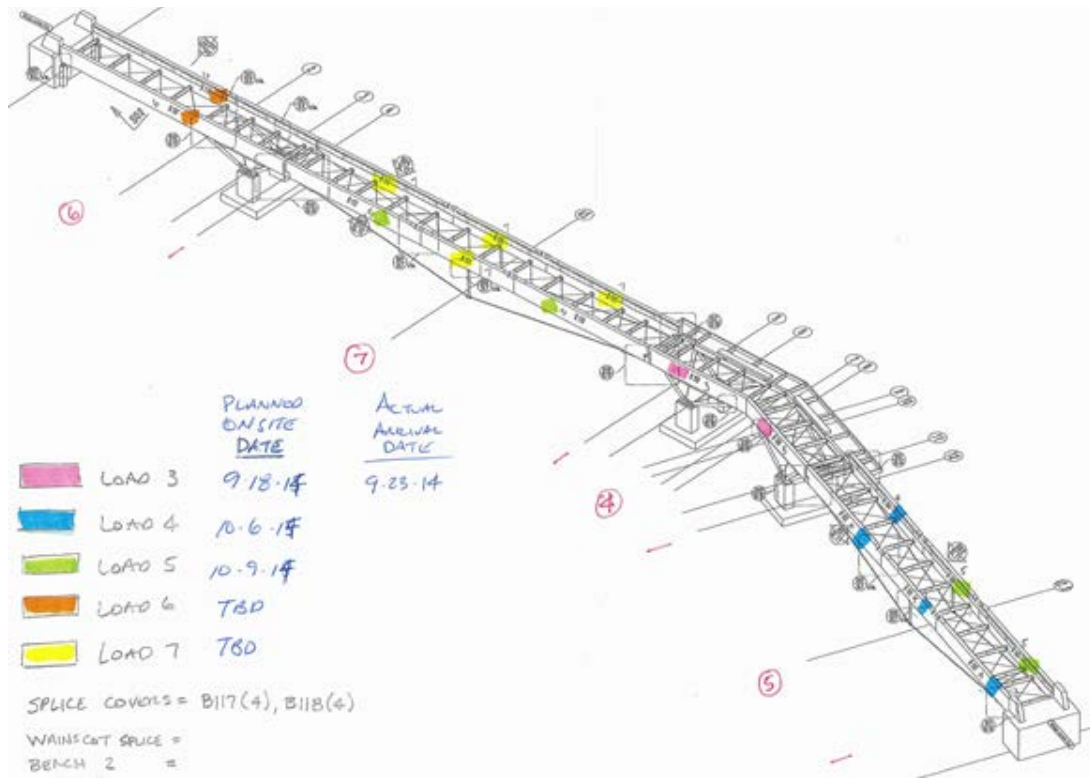


Fig. 4 – An isometric view of pedestrian Bridge No. 2 above

The deck of the bridge consisted of poured in place concrete over metal deck supported over steel beams at 10 ft. or 12 ft. on centers. Below is a typical cross-section of the bridge.

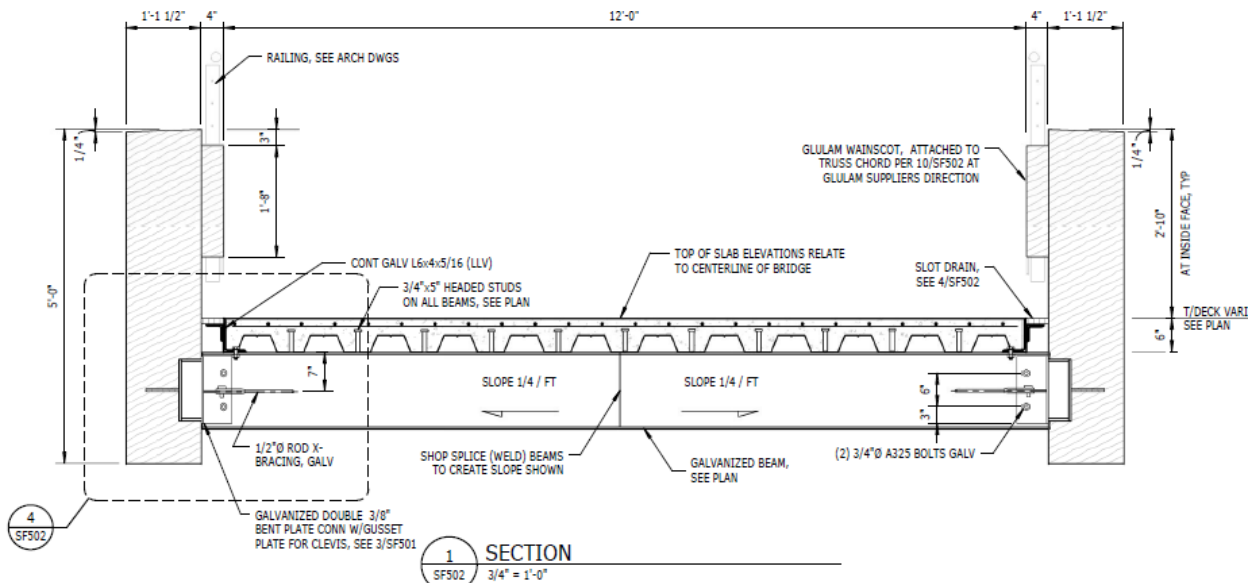
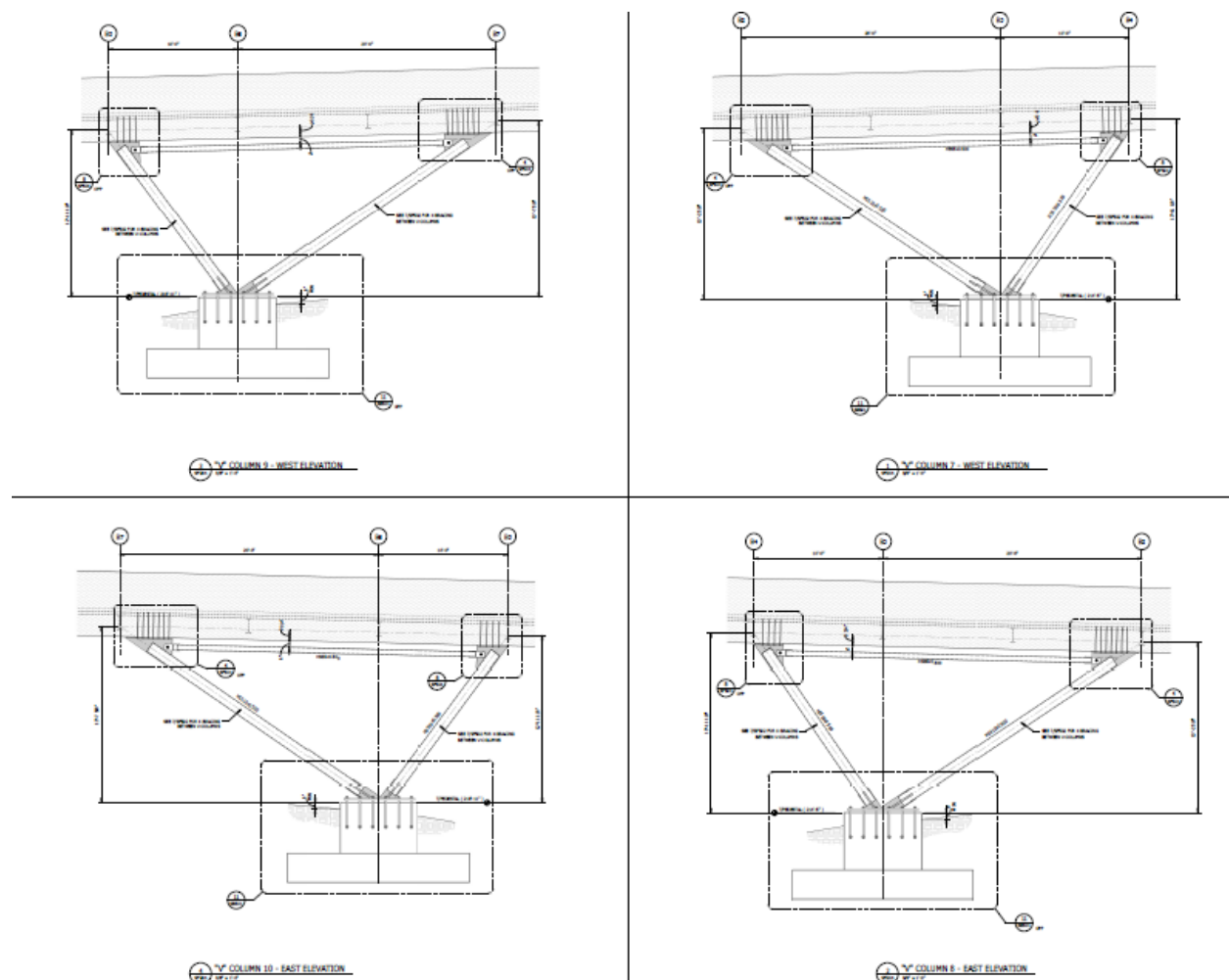


Fig. 5 – Cross section of the bridge

The two end spans of bridge No. 1, and the center and one end span of bridge No. 2 were framed over what was called V-Columns. The V columns were triangular framed structures anchored to the footings with anchor bolts. The two inclined supports generally consisted of 10" round steel pipe; the strut consisted of 5" round steel pipes. The frame was anchored to the glue-lam with screws. Below are the elevations of the four V-columns of the bridge No. 1.



(Reproduced from Stewart drawings)

Fig. 6 – Typical V-Columns for Bridge No. 1

The center long spans of the bridges that collapsed were simple trusses comprising of 13 1/2 x 60" glulam beams as horizontal members, 8" round extra strong steel pipes as king post at the center, and 2 3/16" dia. sloping locked coil steel cables acting as tension members. The cables were eccentrically connected to the glulam beams. The following are the typical elevations of the trusses of the truss 1 and truss 2 for bridge No. 2, and for trusses 5 and 6 for bridge No. 1.

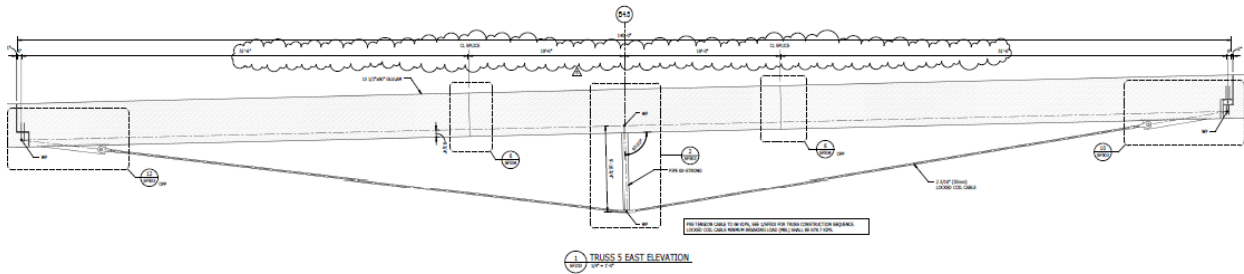


Fig. 7 – Elevation of Truss 5 (Bridge No.1)

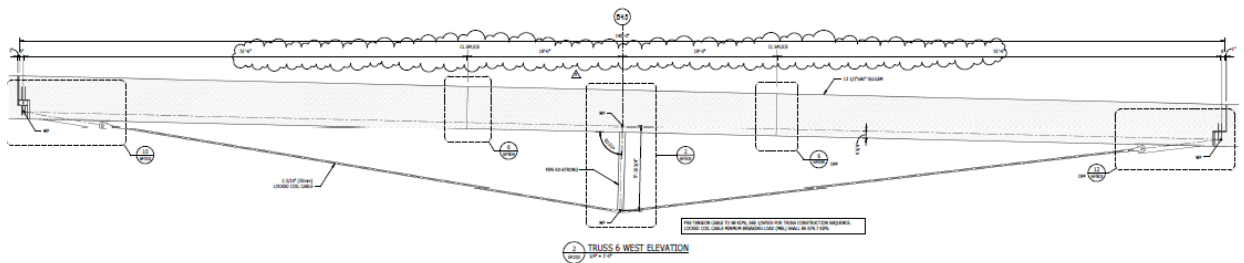


Fig. 8 – Elevation of Truss 6 (Bridge No. 1)

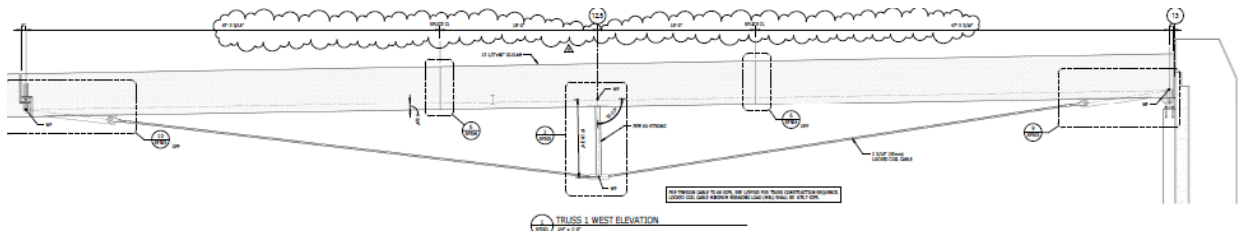


Fig. 9 – Elevation of Truss 1 (Bridge No. 2)

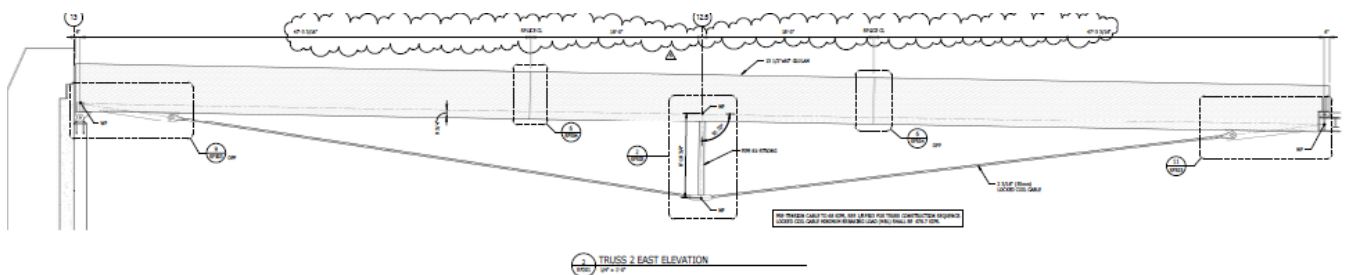


Fig. 10 – Elevation of truss 2 (Bridge No.2)





The center posts were connected and welded to a larger tube attached to the underside of the glulam, see detail below.

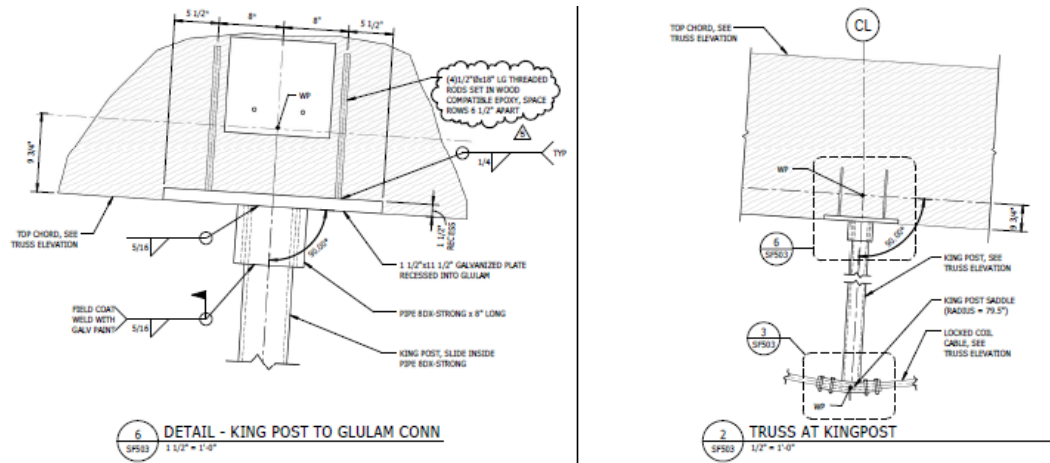


Fig. 14 – King post to glulam connection detail

A saddle was attached to the bottom of the king post to receive the cable as detailed below.

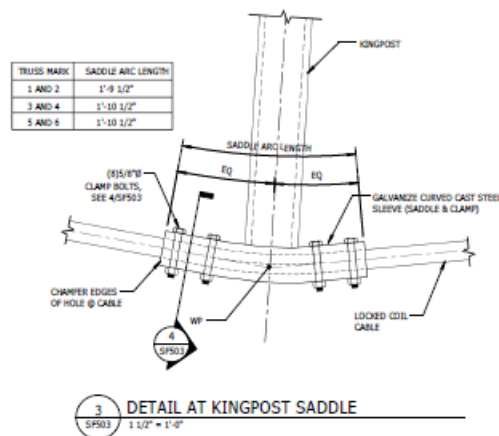


Fig. 15 – Detail at kingpost saddle

The cable at each end was fastened to a plate with a 4" diameter steel pin. The plate was attached to the bottom side of the glulam beam with a series of (50) 1/2" dia. screws, 20" long at 45 degrees. The locked cable was provided with an adjustable forged socket, as shown in the following typical detail.

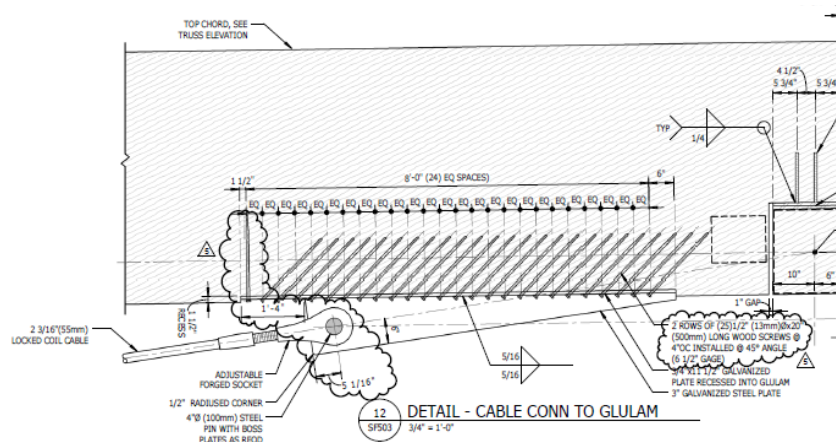


Fig. 16 – Cable connection to glulam at pinned end

The glulam was designed as simple beams pinned at one end, and sliding at the other, and continuous at the center over the king post. The glulam was notched at each end to facilitate connection to the supporting glulam beams over the V columns. The typical pinned connection is reproduced below.

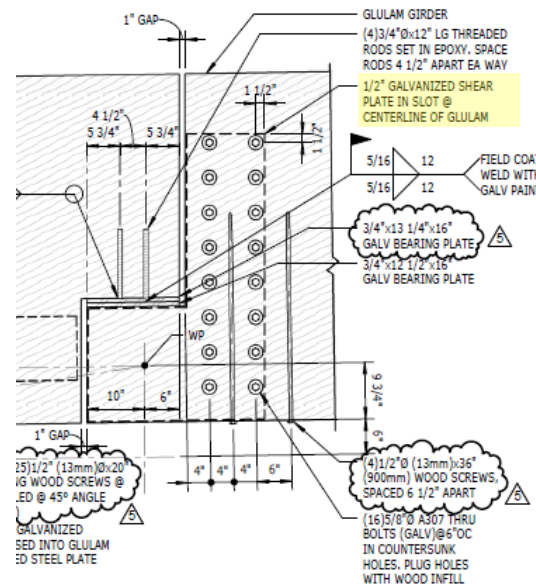


Fig. 17 – Glulam to glulam pinned connection

The sliding joint was provided with a pin which could slide in a slotted hole as shown here:

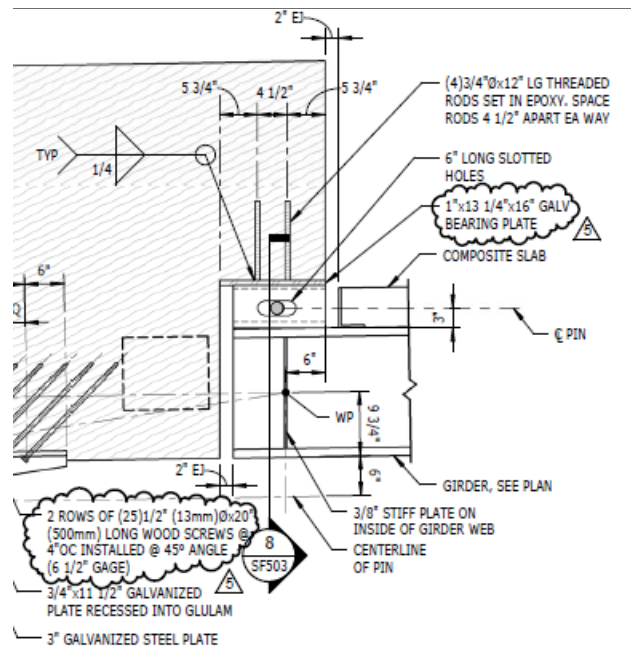
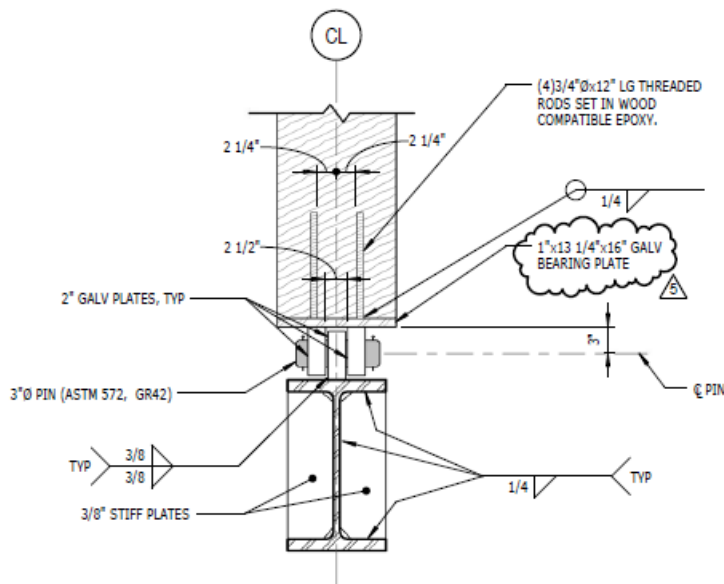


Fig. 18 – Sliding connection of the glulam girders



8 SECTION  
SF503 1" = 1'-0"

Fig. 19 – Section at the sliding connection



## Description of the Incident

### Bridge No. 1:

Buckner erected the V-columns on either side of the central span of bridge No.1 on or about September 8, 2014, and erected the central span with king post and cables on or about October 15, 2014. The steel beams and the metal deck were then erected. Central Concrete placed re-bars over the deck on or about November 10, 2014. Two days later JO Concrete began to pour concrete over the deck in a staggered manner in accord with the structural drawings. The following sketch shows the dates of pouring for the deck up to the day of the incident.

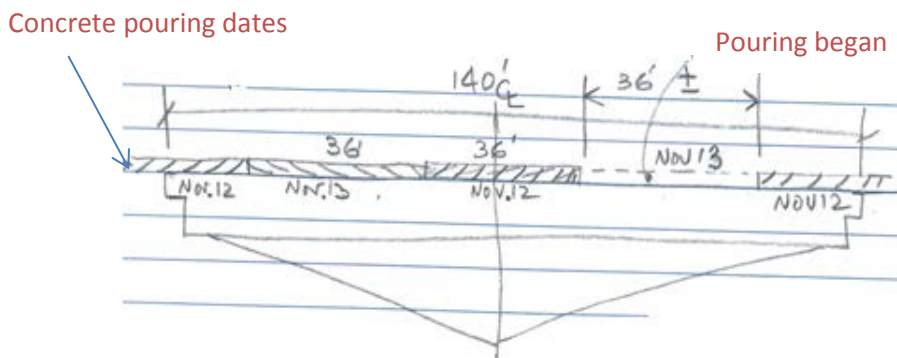


Fig. 20 – Dates of concrete deck pouring for Bridge No. 1

At the time of the incident the segment of the deck near the center of the span was being poured when suddenly the trusses collapsed some 25 feet to the ground with five employees of JO Concrete falling to the ground. One was fatally injured, and four others sustained serious injuries of varying degrees. The glulam girders horizontally split in two sections, as roughly depicted in the sketch below.

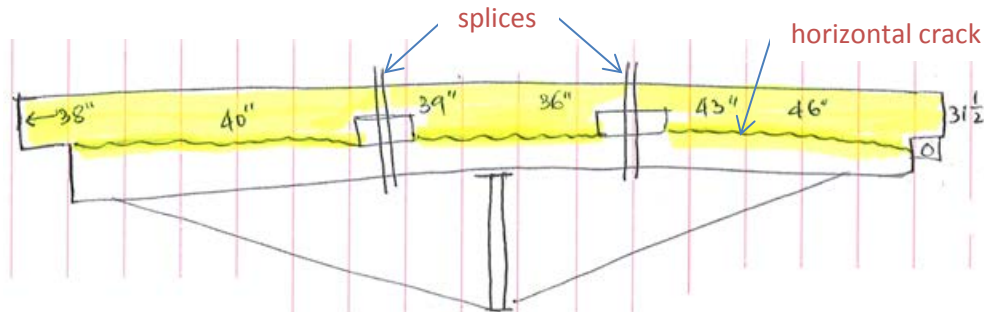


Fig. 21 – Truss TR-5 horizontal crack

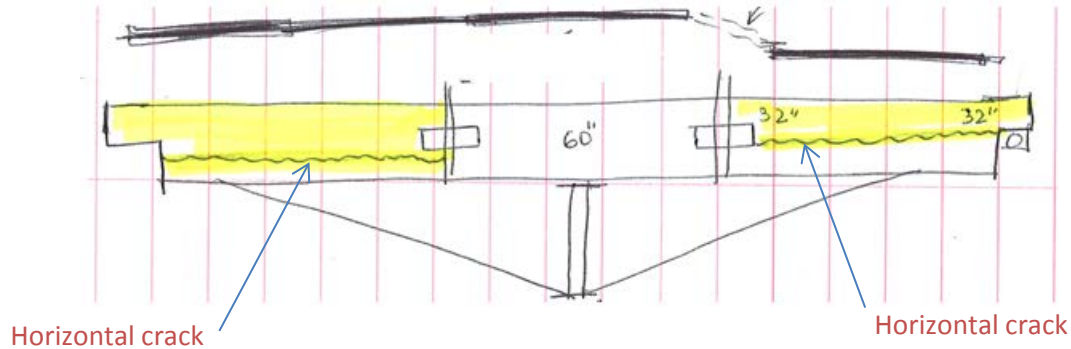


Fig. 22 – Truss TR-6 horizontal crack

The truss TR-5 split horizontally into two sections with a crack emanating from the notched section at the pinned connection towards building F and proceeding towards the two splices near the bottom of the tube inserts, and then ending a few inches below the notch at the sliding joint towards future building L. The truss TR-6 sustained a similar horizontal crack emanating from a few inches from the pinned connection and stopping at the bottom of the first splice near the bottom of the tube insert. The girder remained mostly intact between the two splices. But the horizontal crack again appeared beginning from the second splice ending at the notched section of the sliding joint. In both trusses, the insert tubes at the splice locations were dislodged to varying degrees.

The four threaded rods set in the glulam and welded to the steel embed plates failed. The welds between the threaded rods and the plate generally failed but the bearing plate remained welded to the 1/2" shear plate in the slot of the glulam.

The following photographs show the separation at the notched sections, and the spliced conditions of trusses TR-5 and TR-6.



Fig. 23 – View of the collapsed girders



Fig. 24 – View of the collapsed girders





Fig. 25 – View of the collapsed girders



Fig. 26 – View of the collapsed girders



Fig. 27 – View of the collapsed girders



Fig. 28 – View of the collapsed girders



Fig. 29 – View of the top splice plates



Fig. 30 – View of the top splice plates

### Bridge No. 2:

Bridge No. 2 was longer than Bridge No. 1 and consisted of two suspended spans, approximately 131 ft. and 140 ft., and three V-columns. This bridge was partially completed at the time of the incident but only the 131 ft. suspended span (Truss TR-1 and TR-2) was erected, including the three V-columns. The other suspended span (Truss TR-3 and Truss TR-4) was not yet erected.

TR-1 and TR-2 had a pinned joint at the abutment, and a sliding joint at the V-column girder. The two central V-sections were erected by the end of October 2014. The trusses 1 and 2 were erected in the first week of November 2014, and concrete was poured in segments.

At the time of the incident, truss TR-1 developed a continuous horizontal crack beginning from the notched section at the sliding joint through the splices and ending a few inches below the pin joint at the far side. Truss TR-2 also separated along a horizontal crack emanating from the notch at the sliding joint, but the crack appeared to terminate at the nearest splice. The following sketches indicate the approximate location of horizontal cracks along the longitudinal axes of the trusses.

The top segment of TR-1 above the fracture remained atop the sliding joint while the bottom segment fell on the ground. Similarly, the top portion of the girder at the abutment remained atop the abutment. The top and bottom segments of TR-2 up to the first splice fell to the ground. The remaining girder from the first splice to the pin joint appeared to remain intact with the girder remaining atop the abutment.

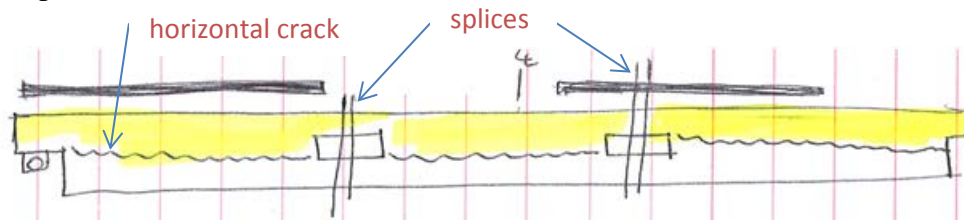


Fig. - 31 Truss TR-1



Fig. 32 - Truss TR-2

Concrete was poured on the deck of the 131 ft. span in a staggered manner as indicated in the following sketch.



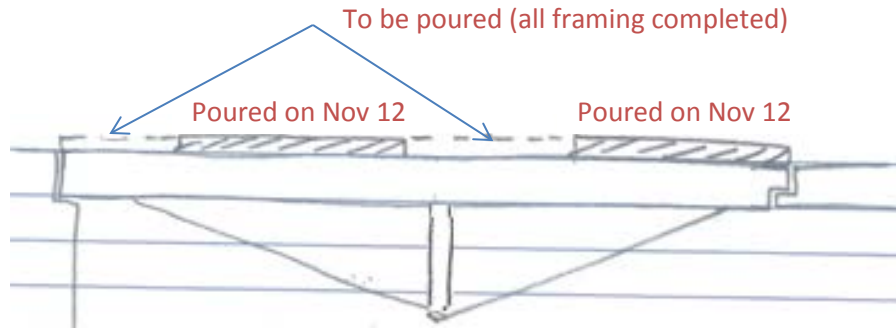


Fig. 33 – Bridge No. 2 Concrete pouring



Fig. 34 – View of the collapsed girders



Fig. 35 – View of the collapsed girders



Fig. 36 – View of the collapsed girders



Fig. 37 – View of the collapsed girders



Fig. 38 – View of the collapsed girders



Fig. 39 – View of the collapsed girders



Fig. 40 – View of the collapsed girders



Fig. 41 – View of the collapsed girders



Fig. 42 – View of the collapsed girders



Fig. 43 – View of the collapsed girders

### Truss Erection:

Skanska retained Buckner to perform the erection of the trusses of the bridges. Buckner contracted with Kimley-Horn to devise a plan to erect glulam girders, kingpost, and the cables to the desired geometry in accord with the design. Kimley held a series of meetings with Skanska, Stewart and Buckner, and corresponded with Pfeifer by email to determine the method of erection to satisfy the intent of the design and the resulting cable configuration. The length and desired pre-tension of the cables, the height of the king post, and the camber of the glulam girders at different stages of construction were among the subjects discussed between the parties.

Finally, it was determined that the glulam girders would be provided with a camber, although the structural drawings did not indicate any camber, to achieve a magnitude of tension in the cable, to facilitate placement of cables in the saddle of the king post, and to ensure that the trusses would remain practically horizontal with the application of the service loads excluding the live load. Kimley conducted independent structural calculations to determine the jacking forces to induce the necessary camber in the trusses while remaining within the permissible flexural and



shear capacities of the girders, in particular at the splice locations. The limits were provided by Stewart. Such calculations and the final method of erection were under constant review by Skanska, Buckner, Stewart and Pfeifer. The following was the agreed upon method and means of erection as reflected in Kimley's engineered erection plan of September 10, 2014.

The girders were placed over the supports at each end and over a temporary shoring tower near the mid-span of the girders. Connections at each end were completed before the crane was released. Steel framing for the bridge deck was completed. The vertical king post was then placed in the sleeve at the underside of the girder. The cable was attached at each end with 4" dia. steel pins and was left hanging loose. A jack at the shoring tower jacked up the girder to a predetermined level. A jacking beam was placed over the cables and a reaction beam was anchored to the footing of the shoring tower with a jack in between them. Adjustments to the cables, by fine tuning the sockets of the wire ropes, were made at each end to get the desired length of the cables, taking into account the fabricated cable length and the cut-off forces. The desired lengths of the cables were pre-determined by Stewart to impart necessary tension in the cables. The cables were jacked down until they were seated in the saddle at the bottom of the king post. Another jack was placed between the bottom of the girder and a jacking collar placed at the bottom of the king post to pull down the king post out of the sleeve to a predetermined height of the king post. The king post was then welded to the sleeve after Stewart approved the final cambered configuration of the bridge. Similar procedures were used for trusses 5 & 6 and 1 & 2 for bridge No. 1 and No. 2, respectively. It must be noted here that the tension in the cable was theoretically computed based on the geometry of the trusses, and were not verified by any actual measurement of the tension. In the case of bridge No. 2, the cable length had to be readjusted after the king post was already welded as instructed by Stewart to fine tune the camber of the trusses.



Fig. 44 – Temporary shoring tower supporting the trusses



Fig. 45 – Temporary shoring tower



Fig. 46 – Temporary shoring tower supporting the trusses



Fig. 47 – King post and the cables





Fig. 48 – Cables being positioned



Fig. 49 – Ready for concrete pour



Fig. 50 – Framing for the deck



Fig. 51 – Sliding joint between glulam girders



Fig. 52 – Screws being driven for splice flange plates



Fig. 53 – Sliding joint plates embedded in glulam girders

## **Structural Analyses**

Due to aesthetic considerations, and as an alternative to using treated wood, Alaskan Yellow Cedar (AYC) was initially selected for the glulam, but this was later changed to Port Orford Cedar (POC). Both of these woods have comparable structural properties.

OSHA reviewed the computations, both computer generated and long-hand performed by Stewart. The bridges were designed to meet the applicable code for wind and seismic loads. A live load of 90 psf was used without any reduction. The girders were first analyzed with the tension cables considered to be concentric with the center line of the girder. They were also analyzed with the actual eccentricity of the cables. The modeling technique to impart the eccentricity to the cable could have been improved as it provided misleading results, although not detrimental to the overall design of the trusses.

Under varying dead loads, the stresses in the glulam girder and the tension in the cable were determined. For trusses 5 and 6, it was determined that the cable should have a tension of approximately 161 kips when subjected to a full unfactored dead load resulting in zero deflection at the king post location. For trusses 1 and 2, the cable would be stressed to approximately 147 kips with zero deflection under a full unfactored dead load. The resulting flexural and shear capacities were checked and found to be within permissible limits. For the application of full unfactored live load, the mid-span deflection was computed to be approximately 4.4 to 4.8 inches in trusses 1 and 2, and 5 and 6, respectively.

### **Notches in glulam beams:**

The main girders of the trusses were designed as a pinned bearing connection at one end and a “sliding” bearing support at the other. The supports at each end for trusses Nos. 5 and 6 were provided by the glulam girders projecting from the V-columns. For trusses Nos. 1 and 2, one end of the trusses had a bearing over the concrete abutment, and the other had a bearing over the glulam girder projecting from the V-Columns. Where the bearings were over the glulam girders, both glulam girders were notched to present the appearance of having one continuous beam. The notched ends of the glulam girders projecting from the V-columns were provided with concealed embedded steel plates, almost for the entire depth, to resist the shear and to alleviate potential

cracking due to notching. There were no cracks observed on those glulam girders projecting from the V-column after the incident. Unfortunately, similar embedded plates were not provided in the glulam girders of the trusses where long horizontal cracks were observed after the incident, and this was a primary cause of the failure.

Notches in wood members have been a perennial concern among structural engineers, and all textbooks and codes recommend against using them unless remedied by providing long lag wood screws crossing the potential line of cracks, or by steel plates bolted to the glulam beams on either side. The well-recognized concern is based upon the reduced section for shear, stress concentration at the notch in re-entrant corners, and the tendency of the notch to induce stress perpendicular to the grain, resulting in tearing of the section where horizontal shear occurs. The potential formation of the cracks is more likely in the tension zone rather than in the compression zone. The end section of the glulam girder underwent a transformation of stresses when the camber initially induced gradually diminished under the application of the dead loads of concrete. In this particular case, the situation was exacerbated by the vertical component of the tension in the cable which could accelerate the tear at the ends. Additionally, the wood near the mid-depth of the section is generally of lower quality laminations, not as high quality as wood near the top and bottom of the glulam contributing to the propagation of the cracks. Both ends were designed as pins with one end having sliding capability. The National Design Specifications (ANSI/AF&PA NDS-2005) permits notches at bearing over a support on the tension side not to exceed lesser of  $1/10$  the depth of the member or 3", and on the compression side not to exceed  $2/5$  of the depth of the member. In this instance, the limits were exceeded.

#### Splices:

As discussed earlier in this report, each glulam truss girder was spliced at two locations symmetrical to the mid-span. Three-quarter inch steel plates were provided at the top and bottom flanges with  $2\frac{1}{2}$ " dia. wood screws at 45 degrees on either side of the splice to resist the flexural moment of the girder. To resist the shear, a cut-out was made inside the glulam to place a 12"-long steel tube, HSS 16x8x3/8. Similar to the predicament created by the notches at the ends of the glulam girders, the cut-outs presented a potential for developing cracks if no remedial measures were taken. One of the measures could have been to extend the screws beyond the line

of the cut-outs to arrest potential cracks. Horizontal cracks were observed extending to the bottom of the cut-outs.

#### Missed opportunities:

The design and structural drawings prepared by Stewart were reviewed and examined by a number of consultants with expertise in structural engineering during the execution of the project, and none raised the issue of notching the glulam girders. There were several missed opportunities when Structurlam Products, Skanska USA Building Inc., Kimley-Horn and Associates Inc., and Pfeifer all examined the structural drawings and its details for their own purposes. The presence of the notches went un-noticed, and none of the consultants raised any concerns with Stewart. While it is recognized that the ultimate responsibility rested with the engineer of record, it is expected that others involved in the project with experience would raise these issues whether they directly impacted them or not.

#### Conclusions

1. The cause of the failures of bridges Nos. 1 and 2 was the structural design flaw in that the glulam girders were severely notched at each end to facilitate end connections. The notches under the application of a full dead load resulted in the formation of horizontal cracks, eventually leading to the catastrophic failures.
2. It is unfortunate that a number of consultants participating in the project who reviewed and commented on the structural design during the course of the project either did not notice the adverse impact of the notches or neglected the opportunity to raise the detrimental characteristics of the notches with the structural engineer.
3. The notches were not in compliance with the applicable ANSI/AF&PA NDS-2005 standard.
4. The consultants/contractors failed in their professional responsibility to share their knowledge and expertise with the structural engineer in regard to the presence of notches. These professionals included Structurlam Products LP, Skanska USA Building Inc., and Kimley-Horn and Associates Inc.
5. The bridges otherwise were adequately designed.