DELAWARE RIVER TURNPIKE BRIDGE FRACTURE REPAIR

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BIOGRAPHY

Richard Schaefer is an associate vice president and chief bridge engineer for the New Jersey office of the HNTB Corporation. Rich has 20 years of experience designing and supervising the construction of conventional and unusual bridges and bridge projects in the northeast region and around the country varying from simple overpass replacement to complex cable supported bridge rehabilitation. Rich uses his experience to assist bridge owners with writing design manuals and is also avidly involved with research and development of innovative concrete for bridge decks, accelerated bridge construction methods, steel fabrication advances, and rapid recovery for bridges. Rich also serves as an adjunct professor at NJIT where he teaches a master's class in bridge design. He is a licensed Professional Engineer in the State of New Jersey. He holds a B.S. in Civil Engineering from the New Jersey Institute of Technology,

Frank A. Corso, Jr. is a Senior Supervising Engineer with the New Jersey Turnpike Authority. He has 30 years of diverse technical and management experience in responsible charge of transportation design and construction projects. In his current position, Mr. Corso leads a team of construction professionals in the management of multiple ongoing capital improvement program projects including new bridge construction, bridge rehabilitation and repairs, roadway reconstruction, major interchange improvements, and security work on the New Jersey Turnpike and Garden State Parkway. His ongoing projects exceed $1 billion in value. Mr. Corso is a Licensed Professional Engineer in the State of New Jersey. He holds a B.S. in Civil Engineering from the University of Delaware, an M.S. in Civil Engineering from the New Jersey Institute of Technology, and an M.B.A. in Finance from Rutgers University.

SUMMARY

The Delaware River Turnpike Bridge carries I-95 and I-276 over the Delaware River between Bristol PA and Burlington NJ. The overall bridge is 6.571 feet long abutment to abutment. The outer approach spans are traditional short steel girder spans leading into continuous three and four span continuous deck truss units. The central river spans unit is a true through truss with a center suspended span that achieves a 682' span between the piers flanking the navigable channel. On January 20, 2017, a full-depth fracture was discovered in the fracture critical designated top chord of the north deck truss. This paper discusses the compressed time frame effort to repair a major deck truss fracture, which has never been attempted.

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Introduction

The Delaware River Turnpike Bridge carries I-95 and I-276 over the Delaware River between Bristol PA and Burlington NJ. The bridge was built in 1954 by a joint effort between the New Jersey Turnpike Authority and the Pennsylvania Turnpike Commission. Both toll road agencies co-own and operate the bridge and connector road it supports.

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On January 20, 2017, a full-depth fracture was discovered in the top chord of the north deck truss between Panel Points U20-U19’, adjacent to Pier 15 on the Pennsylvania side of the bridge. The bridge owners were immediately alerted and the bridge was closed to traffic. Over the course of next 49 days, the cause of the fracture was evaluated, temporary supports were installed, the fractured member was repaired, and the bridge members were restored to their original dead load stresses. This paper discusses the compressed time frame effort to repair a major deck truss fracture, which has never been attempted.

The Fracture

The fracture was noted on the in-service bridge on the afternoon of Friday, January 20th. A primary member in the top chord of the north truss had fractured completely through the cross section of the member leaving an approximate 2” gap between the fracture surfaces. Surprisingly, the bridge showed no change in function and stability. In fact, the surface corrosion on the fracture surface suggested that the fracture had occurred days or weeks prior to being identified. As the member had been recently painted, the difference was readily noticeable. The brittle nature of the fracture with no apparent ductile behavior in the base metal suggested that the fracture was immediate and energetic, a conclusion substantiated by the fact that the recently painted connection adjacent to the fracture cracked and ejected the paint at the connection interface, as seen below in Figure 1.

Figure 1: Fractured Truss Chord

Unlike many built-up riveted member deck trusses in service, this truss was fabricated almost entirely from rolled steel wide flange shapes (W shapes) in two different grades of steel and a wide variety of sizes to economize the weight of the structure. Light weight W shapes rolled from common carbon steel were used in members expected to see minimal stress while super heavy ‘jumbo’ series W shapes rolled from high strength manganese steel were used in areas of high load and stress. Steels which derive higher tensile strength from the alloying of manganese can suffer ductility loss and embrittlement from welding, which is generally prohibited where this type of steel is selected.

The fracture noted above would later be identified as a brittle fracture through the entire section emanating from two mis-drilled rivet holes in the near side flange that had been filled with weld metal. This can be seen in Figure 1 above. thereby creating a stress riser condition in the manganese steel of the member. These welds were traced back to a repair made in the original fabrication of the member in the shop. The forensics of the cause of this failure are discussed by others participating in this session of the Symposium.

validating that it had exceeded its yielding capacity.

How the Bridge Survived the Fracture

After initially surveying the damage, it was noted that failed paint could be traced back to either end of the fractured member at the plate laps. The member adjacent to the fractured member also appeared to be bowed near the point of buckling, and the adjacent floorbeam was visibly deformed. Review of the structure later would reveal that the four span continuous north truss plane had survived the fracture by effectively becoming two, two span continuous truss frames with a plastic hinge over the center bearing at the middle pier (Pier 15). A schematic of this effect is shown below in Figure 4. The adjacent top chord members, which were lightly loaded contraflexure members that were nominally sized W14x87, had now become heavily loaded compression members. The adjacent W14 on the eastern side of the fracture had also absorbed the majority of the energy of the fracture and had visibly buckled along its weak axis. There was concern that this member had survived relying on post yielding strength and that it had little, if any, reserve capacity remaining to support the structure. A photo of this member is shown below with a red arrow pointing to the bow.

Figure 2: Installed Emergency Splice

As can be seen above, the small truss member was under duress and exhibited signs of post-yield behavior. Later work would show that the bow would not elastically relieve from the member,

First Step -Close the Gap

The first step to the repair effort for the bridge was to stabilize the member. At this point, the bridge had successfully survived the fracture and carried live load, possibly for several days to weeks prior to its discovery. However, there was no way to immediately determine the actual demands placed on the still functioning members of the truss, and whether they were near failure. Given the potential for loss of collateral and public safety, combined with a weather forecast of high winds, the decision was made to immediately splice the two fractured ends back together with an ad-hoc bolted connection fabricated from available plates and bolts. Prior to any repair work, coupons were cut from the fractured ends for later testing.

Plates were drilled in the field on the ground and hoisted up. Shimming of the plates with various thickness fills became something of a puzzle to attain adequate fitment. With the space and time available, the decision was made to design the bolted connection using the ultimate capacity of the bolts without the usual factors of safety granted by load and resistance factors, which were omitted.

Limited room for the connection made installing the splice difficult, and the misalignment of the two ends required careful shimming to align the splice plates. Forcing of the two ends back into alignment was not permitted. Work to install the splice progressed on 24 hour shifts through the weekend to beat the forecast high wind event, with the splice work being completed just as the wind speeds began impacting the ratings for the manlifts used to access the splice. A photo of the completed splice is noted below:

Figure 3: Installed Emergency Splice

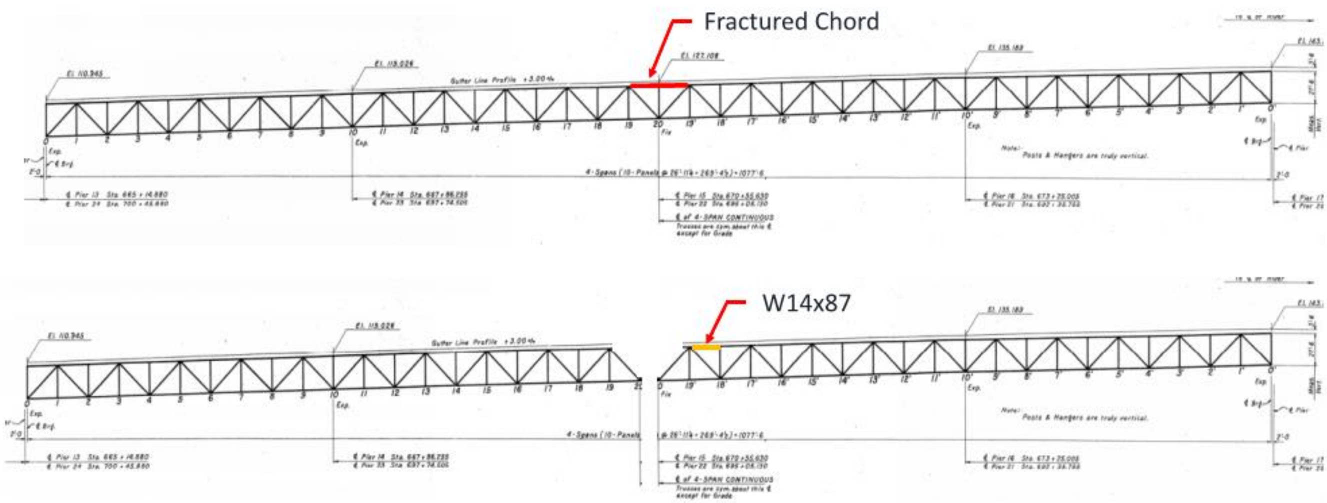

Figure 4: Truss Prior to and Following Fracture.

Second Step -Adding Support

With the emergency splice installed to provide an additional load path should the bowed W14x87 fail, the next step was to design and install temporary support towers to stabilize the structure, and assist in repairs to the truss, which had sagged significantly in its new loading configuration, or to assist in the safe demolition of the truss, should it be found to be unrecoverable.

After constructing a basic model of the truss, it was determined to construct two towers on either side of the fractured truss at gusset plate panel points which represented 40% and 60% of the span away from the fracture at the pier, constituting four towers under the north truss which had fractured and sagged, and four more towers under the south truss, which had not fractured, but had sagged under the additional load shed from the north truss.

This created a total of eight shoring towers that had to be erected in short order. The Contractor, Cornell and Company which had been retained to perform the steel work for this project, had ample supply of tower crane sections, which they commonly used in vertical construction, and were capable of supporting more than 1,000 kips, each.

The foundations for the towers were evaluated as either drilled micropile foundations or cast in place concrete spread footings. While the cost and construction duration of the two options was roughly equivalent, it was ultimately decided to advance the micropile option as it allowed for a much smaller footprint in the confined work area than the 20’x20’ spread footings, eliminated what would have been large scale earth moving to dig down to solid base material, and eliminated the longer cure time for concrete. The micropile foundation also provided a more vertically rigid foundation that would allow for tighter control over future vertical jacking work.

After notice to proceed was given, a specialty contractor, Moretrench, was able to provide three micropile installation rigs which worked around the clock to install the micropiles. The effort to concurrently drill in such tight confines created large-scale water runoff and muddy conditions that had to be mitigated with free draining gravel fill. The drilling operation can be seen below in Figure 5.

Figure 5: Micropile Installation

An unanticipated benefit of the micropile foundation selected was that it readily allowed for moving of any one of the six piles supporting each tower to accommodate utilities found in the roadway under the bridge at tower locations.

Design responsibilities for the towers were divided to speed the work. The design engineer (HNTB) designed the micropile foundations and worked directly with Moretrench to expedite installation and testing. Cornell and Company designed the support grillage across the tops of the piles using steel shapes from their available stocks with HNTB reviewing the designs and shop drawings. A photo of the erected towers can be seen in the below Figure 6.

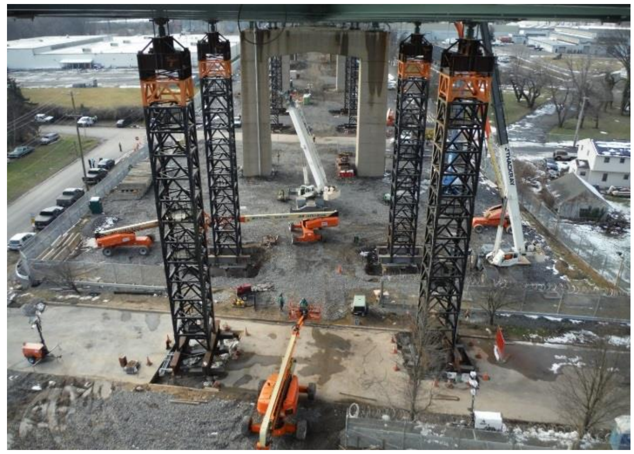

Figure 6: Support Tower Installation

Third Step -Can We Fix It?

While the foundation and tower installation advanced, a parallel task was undertaken to further develop a 3D model in CSiBridge to analyze the deck truss superstructure unit. Given the limited time frame to repair the structure, rather than traditional checking, parallel independent models were prepared by other members of the project. Near the end of the project, four independently constructed models of the bridge were used to compare results for various staged repairs to the superstructure. Results within 5% of parallel model results were considered acceptable.

Various schemes were investigated using different sequenced lifting arrangements with jacks placed atop of the eight temporary towers. A parallel option included using a compact post tensioning scheme to pull the fractured chord back together.

The final selected scheme included elements of both the vertical jacking from the towers, and the longitudinal post-tensioning of the fractured chord. The benefits of this arrangement became apparent as the design progressed.

The vertical jacking work done at the temporary towers allowed the bridge to be reset to its original geometry, eliminating the sag from the bridge and moving the fractured chord closer to its original position. While repairing the bridge in a single operation would have been preferable, to restore loading in the fractured member, the truss would have required much greater forces. In effect, the truss would need to be over-jacked beyond its as-built profile to camber the truss up, then splice the fractured member, then release the jacks and hope that the final stresses in the repaired truss were approximately matching to the as-built condition.

The over-jacking force would have been significant enough that either major strengthening modifications to the truss and/or removal of the deck slab would have been required. This option was therefore discounted.

Conversely, simply post-tensioning the fractured truss member back together had no guarantees that the bridge would return to its original geometry. Simply forcing it back together may have incurred more damage to already deformed members.

The final decision was made to use the jacks to vertically force the truss back to original geometry and hold it there while a subsequent longitudinal post-tensioning arrangement would restore load to the fractured member.

Minimal strengthening of the truss at the jack points was required to resist the force from the jacks, and a temporary cross brace was installed at the jack points as can be seen below in Figure 7. In addition to the cross braces, the gusset plates at the jacking locations were heavily reinforced with stacked plate steel saddles to uniformly distribute the jacking force across the full width of the gusset plate without damaging it.

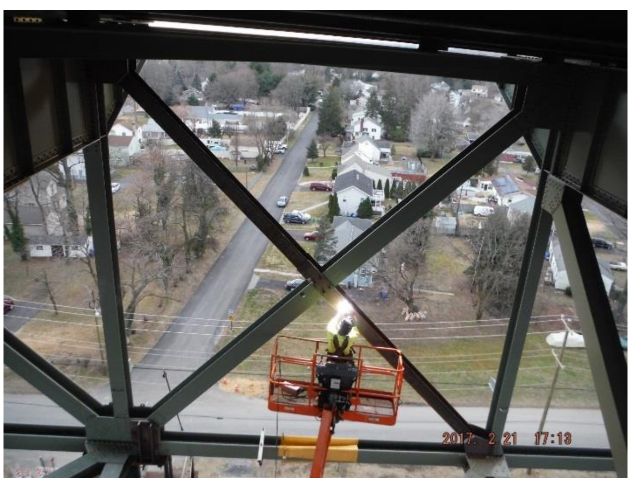

Figure 7: Cross Brace Installation

It is at this time worth noting that the strengthening work was being completed, the jacking towers were still in the final stages of completion and the 600 Ton jacks were being installed atop them. Construction forces had to balance 24 hour work shifts, multiple trades, access, heavy equipment movement, constant materials deliveries, and no less than 13 high reach manlifts in an area no larger than two football fields. This is in addition to the over 100 workers, engineers, inspectors, local residents and media interests on, about, or in regular viewing of the site.

The Dry Run

Prior to the vertical jacking of the truss, a detailed procedure for the work, informally referred to as the 'Playbook' was created to document each step of the process for the jacking and prepare contingency plans should work not go as expected. The Playbook was then subjected to a 'dry run', which amounted to a dress rehearsal to validate the planned procedures prior to actually energizing the jacks. All parties were on-hand to either participate or view the dry run, including the bridge owners and representatives of FHWA. The dry run was found to be an essential step in the process: encountered communications and equipment issues were readily identified and resolved.

The Fourth Step – Jacking Day

Vertical jacking of the truss back to its original profile was completed on February 24th. The combined approach with the horizontal post tensioning meant that only a fraction of the vertical jack capacity was required and the procedure was distilled into four runs of eight separate incremental jacking pressure increases of 100 psi increments until the target pressures were reached. Throughout the entire jacking procedure, the behavior of the truss was continuously monitored using a suite of strain gauges linked to a central reporting facility in an ad-hoc 'command center' located adjacent to the bridge. The sensor suite was installed and maintained by WSP, Inc. In the end, the behavior of bridge showed an elastic response throughout the operation. The vertical jacking was completed in a single day.

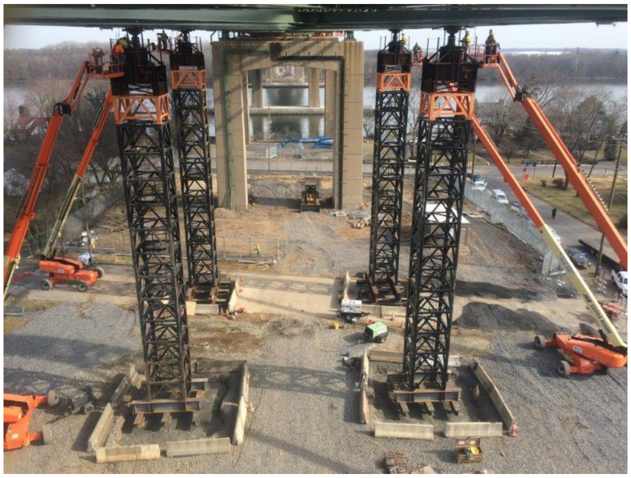

Figure 8: Vertical Jacking Day

Lessons learned from the jacking operation were used to inform our understanding of the bridge and prepare for the next step of the process. Most notably, it was found that the truss responded elastically, but was actually much stiffer than expected. Trusses are traditionally designed with the assumption that the connections between members at gusset plates act at pinned and are free to rotate. In truth, gusset plate connections are incredibly stiff and offer significant restraint against rotation. Therefore, we prepared two models of the structure with both pinned and fixed end conditions considered. After jacking, these models were recalibrated to correspond with the actual encountered stiffness of the truss, which was approximately 30% stiffer than the pinned end condition assumption. This additional stiffness is likely due to the contribution of the deck and secondary members offering unanticipated load paths.

A second lesson learned from the vertical jacking was that the sophisticated measuring systems employed for use, which ranged from precision survey, sensor suites, even laser measurement, all had their own peculiarities, difficulties, and processing time delays in use during construction. At the end, the final call was made that the vertical jacking goal was achieved using a simple hand measurement at each jack location.

The Fifth Step – Post Tensioning

While the bridge was being prepared for vertical jacking and preparations made ready for 'game day', a parallel path was progressing behind the scenes. The remaining portions of the fractured truss chord were less than required to install a full capacity permanent splice. The decision was made to replace the chord member on the east side of the fracture with a new member fabricated from modern HPS bridge steel plate. After the vertical jacking was complete, the contractor installed the new member and cut-back the existing member so that a new permanent splice could be installed. Provisions were made for the longitudinal post-tensioning frames to be installed on both the new and existing ends of the member following its design.

The limited room available for the post tensioning work meant that traditional jacking equipment would not fit. Instead, compact bolt tensioning hollow jacks sourced from the wind tower industry were used. These jacks are intended to pretension tower foundation bolts and have extremely short stroke capacity. However, their power and compact footprint made them ideal for this application. Custom tension bars were cut and threaded to work with the jacks. The west end of the jacking arrangement can be seen below in Figure 9.

Figure 9: Post Tensioning Installation

Post tensioning work commenced on March 3rd after completion of the new member installation, the setting of the jacking arrangement, and recalibration of the models to account for the revised stiffness estimates of the truss.

The post tensioning work proceeded according to plan and the post tensioning force was introduced in small increments so that the sensor suite monitoring could process and verify that the structure was reacting elastically. Geometric measurements of the distance between adjacent panel points on the truss indicated that 1.8” of post tensioning would be required to restore the truss to ‘as-built’ condition. however, our revised models suggested that only 1.3” of post-tensioning would be achievable at the target dead load force of 1500 kips. The decision was made to restore the bridge to correct loading rather than risk overstressing other components by forcing the truss back to ‘as-built’ geometry. At the end of the post-tensioning work, the final goal of 1500 kips was achieved at a distance of 1.375”, closely in-line with the expectations.

After completion of the post tension work, the new field splice between the new and the old member sections was field drilled and fully tightened. The vertical jacks were retracted and the truss was permitted to stand on its own for the first time since January. A photo of the repaired member is shown below in Figure 10.

Figure 10: Repaired Chord

Lessons learned from the post tensioning work again validated that despite the sophisticated measurement tools available, the final call on whether the work was successful was made using simple hand measurements and the pressure gauges on the jacks, which were used as ‘true’ indicators of actual load in the member. Many secondary members restored to their original geometry, or close to it, but the truss itself exhibited some residual sag after the vertical jacks were retracted. Also of note, the strain gauge readings from both jacking and post-tensioning operations indicated very little stress relief on the undamaged south truss, suggesting that while it also deflected down after the fracture, it did not accept much additional load from the fractured north truss.

The bowed member adjacent to the fractured member also did not return to normal geometry and retained its bow, albeit somewhat lessened. This validated the belief that the member had in fact experienced post-yield force effect and almost buckled. The decision was made to fully reinforce this member with bolted cover plates to double its original capacity and effectively neglect the contribution of the yielded base metal.

The Last Step – Why not 8 Trucks?

After work was completed, the final step was to prove the truss capable of carrying live load and behaving elastically in response. Various loading configurations were considered following load testing guidelines outlined in the AASHTO Manual for Bridge Evaluation. However, at the end, and given the severity of the damage to the bridge, the decision was made to load the bridge with eight fully loaded (approximately 90 kips each) triple axle dump trucks explicitly placed in gangs of four to effect maximum negative moment flexure in the repaired beam. The belief was that while this loading configuration is highly extreme and incredibly unlikely in the normal service of the bridge, it offered a high level of confidence in the event of success. A photo of the trucks in the loaded position is shown below in Figure 11.

Figure 11: Load Test Trucks in Position

The load testing confirmed that the bridge behaved elastically and predictably. For surety sake, a parallel load test was performed on the nigh-identical four span unit truss mirrored on the east side of the main span. The compared results between the two spans indicated that the damaged and restored four span unit behaved similarly to the undamaged sister four span unit, this validating the results. The bridge was then reopened to traffic on March 9th.

Final Thoughts – Lessons Learned

After the completion of the repair work and the load testing of the spans, the following can be treated as lessons learned from this project:

The fracture was precipitated in part by an original fabrication error that was repaired in the shop using methods that would not be permitted today. Faults such as these in bridge metal can be difficult to locate and are not uncommon in older bridges. Quality control practices from earlier eras of bridge construction were not executed to the same stringent levels that are required in contemporary construction. This should be considered carefully when examining an older complex structure.

The energy released in the fracture was later back-calculated to be approximately 2.0 times the dead load force in the member prior to fracture. This level of energy is in-line with what would be expected of a perfectly brittle break. Where considering potential fracture of members in a structure, the response of this structure would suggest that a fracture 'loading' of double the dead load force in the member should be considered.

Simpler solutions are almost always better. Our team had nigh-unlimited resources in restoring the bridge, but success of the repair was ultimately determined with simple hand measurements.

The design effort was considerable. The varied team involved consultants and owners and included almost 200 people working in orchestrated parallel path tasks to get the work done on time.

Critical path items were made parallel path items wherever possible. It was understood from the beginning that the bridge would not be opened until it was inspected for other flaws and damage to the satisfaction of the engineers and the owners. To that end, design of repairs to the truss were already well underway. At the same time, multiple inspection teams actively scoured the bridge for other instances of similar defects using visual and Ultrasonic Testing. The result was that potential critical path items were completed and removed from the critical path in time to reopen the bridge without any additional delay.

The construction effort was almost herculean in terms of coordinating the varied parallel trades, construction activities, materials procurement, testing, contract management, limited physical working space and site security. Organizing and controlling this effort was a 24/7 task for the owners and the resident engineer.

While the project was successful and on-time, this is largely attributable to extensive behind-the-scenes work to put multiple contingency plans in place so that obstacles could be overcome immediately, extra machinery and equipment was on hand for when, not if, failures occurred, and informing and ensuring that the owners of the bridge were flexible in accommodating changes as they occurred.

Most importantly, the value of communication cannot be understated. This project was made possible by the insistence of having frequent and regular conference meetings to discuss progress and critical path items. All parties were involved in these meetings and were treated as equal partners, including the contractors. In this open forum, the entire group worked through issues in real-time rather than in traditional series discussion.