HEAVY MOVABLE STRUCTURES, INC.

NINTH BIENNIAL SYMPOSIUM
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DESIGN AND CONSTRUCTION OF THE TOMLINSON VERTICAL LIFT BRIDGE

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OWNERSHIP/PUBLIC USE and MANAGEMENT

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ABSTRACT: The Tomlinson Bridge paper will discuss, in detail, this project site and the design that was a direct response to it, including the unusual design and construction aspects of the temporary bridge erected to facilitate traffic movement during construction of the permanent bridge.

SITE HISTORY

The Tomlinson Bridge project was one that presented challenges of many different kinds for both the owner and their consultant. It is a success in that the complexity of the project is now apparent to both the casual observer as well as those actively involved in the project.

The Tomlinson Bridge is located in New Haven, Connecticut and crosses the Quinnipiac River. The Quinnipiac is the city's main river and forms New Haven Harbor at the confluence of the Mill River. The bridge is located on the site of the original settlers' first structural crossing of the river, first noted almost 250 years ago. It crosses what is one of the widest points of the river. The original crossing was one-half mile wide. As long ago as the early 1800's, a movable swing span was constructed at this site and the harbor mouth was decreased to afford more protection from weather. The current structure width of approximately 500 feet has been the typical crossing width for some time.

New Haven, home of Yale University, needed significant infrastructure improvement as the resurgent economy stressed the decayed and antiquated road network of the city. The Tomlinson Bridge carries US Route 1, a primary local north-south arterial along the East Coast of the United States. In this old urban setting, the Tomlinson Bridge underwent many different faces. Various small swing bridges were constructed on top of one another throughout the 1800's up to 1929. At that time the State of Connecticut decided to put up a larger bridge which could accommodate the needs of the growing community. They selected a popular patented bridge design by Joseph B. Strauss, the noted movable bridge designer. The Strauss patent provided a self contained bascule bridge with superstructure, mechanical and electrical design all included. The selected bridge was a four-leaf bascule span. That is, a pair of parallel double leaf bascule spans, structurally connected at the center of the inboard bascule girders. A local firm was hired to supplement the design by providing the approach span and bascule substructure plans. The bridge was designed to carry four lanes of vehicular traffic or one electrical railroad track for trolley cars. The design was unusual in that it essentially carried a train across a double leaf bascule span. Like most of the northeastern United States, the second half of the last century brought rough economic times to New Haven and the bridge fell into a state of disrepair.

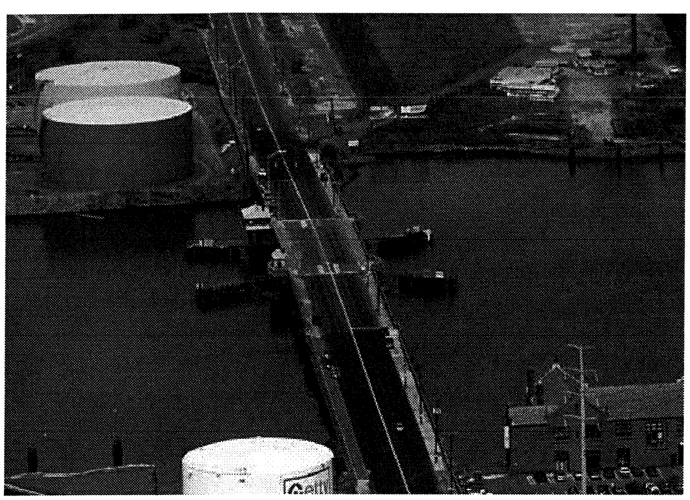

PHOTO: Aerial of Prior Bridge

TYPE SELECTION

When Hardesty & Hanover was initially selected for this project in the late 1980's, it seemed like there would be some type of rehabilitation or maybe even a span replacement utilizing the existing bridge substructure. As time went on, the City, the Coast Guard, the local rail carrier and local groups, made more and more requests of the Department. Even internal DOT bureaus, reflecting on problems with other bridges, required significant inspection and maintenance aspects for the project. The bridge scope quickly shifted from rehabilitation to replacement.

However, the size of the replacement was not fully identified at the outset. The original bridge, as noted above, carried either four lanes of traffic or a trolley. However, the electric trolley cars had long since been replaced with freight. These train loads on the double leaf bascule were damaging the lock machinery, girders and floorsystem, since the bridge was never intended to carry such heavy loads. Inspections performed when the trains were on the span actually revealed the bridge to lift up and impact the main trunnions and rear anchorages as the load ran out onto the cantilevered leaf. Obviously a double leaf bascule was not the answer. A single leaf bascule, swing and lift bridges were now the options available to the state.

The roadway requirements for Route 1 in New Haven had to accommodate the vehicular and rail traffic without causing the delays and safety problems of the existing crossing. To do this, a separate dedicated rail path had to be established. This was done by developing a new rail alignment throughout this portion of downtown New Haven. Over 3000 feet of new on-grade track structure was developed to the north of the existing roadway, separating the traffic as much as possible and providing for standard signalized grade crossings.

Lastly, the Coast Guard saw a history of problems with the bridge that included numerous severe vessel collisions with the bridge as well as many close calls. The bridge is just south of the fixed high level interstate structure, which provided a 240-foot horizontal channel clearance and a 62-foot vertical clearance. The channel parameters for the interstate bridge were utilized for the new Tomlinson Bridge. These parameters limited the options for the replacement structure even further. A swing bridge would be too wide and too long for any conventional design and costs would be enormous. A single leaf bascule bridge and lift bridge were then evaluated. It was decided that the bridge channel was very long and a single leaf bascule bridge would have more long-term maintenance concerns than any lift bridge. The required vertical clearance was low, the span very large and the rail loads were high (Cooper E60). A lift bridge appeared the perfect fit for the site replacing the existing double leaf bascule bridge.

A lift bridge recommendation made by Hardesty & Hanover was accepted by the state of Connecticut. After about 18 months of study, a new lift bridge would replace the old bascule span. The project would also address the permit, rail, and local urban streetscape problems identified during the study. The project would deliver a new lift bridge of enormous proportions, over 90-feet wide by 270-feet long, over 6.4 million pounds. The proposed bridge was to provide a 240-foot clear horizontal channel with 13-foot vertical clearance with the span closed and 62-foot clearance with the span opened. The new bridge increased the vertical clearance in the closed position from 10-feet to 13-feet by increasing the approach grades while still maintaining all rail geometry requirements.

In addition, the project would provide an aesthetically pleasing fishing pier, enlarge the channel, provide new approach roadways with a modified intersection to the east and an improved intersection to the west, and improved highway geometrics through the half mile length of the project. Driveways were improved for many local businesses, mainline access was enhanced and lighting and other utilities were upgraded including local drainage, which is critical in these low-lying waterfront areas. Also, the rail would be separated and signalized in connection with the bridge operation.

THE TEMPORARY BRIDGE

The city required that traffic be maintained in both directions throughout construction. In the past, the bridge was rehabilitated using one-directional traffic flows in the principle direction of traffic during the morning and afternoon rush hours. Traffic was restricted completely during the late morning-early afternoon period. This traffic scheme was no longer considered to be acceptable. Further compounding the traffic maintenance issues was the fact that the upper New Haven Harbor is home to significant oyster seedbeds, and the Mill River is the location for the oyster boat docks. The continuation of this operation, heavily seasonal, but year round, was critical. After review, we selected a temporary movable bridge as the best option for maintaining traffic.

The design would utilize standard prefabricated steel bridge panel-type structures, modified in accordance with the supplier's design preferences. The design would be performance based, using standard panel details and meeting requirements for movable bridge operational features. AASHTO Standard Specifications for Movable Bridges was considered the primary design code and the design allowed a 25% increase in all allowables for the temporary bridge. Seismic considerations were waived. The temporary bridge was required to provide a 110-foot horizontal channel with 13-foot vertical clearance in the closed position and 62-foot vertical clearance with the span open. The bridge was to be operated 24 hours a day, year round and maintained by the Contractor. In addition to the movable span, about 600 feet of approach span structure was also required. The approaches were aligned so that they were off line with the permanent construction to provide as much unrestricted access to the new bridge and grade approaches as possible (See Figure 1).

About two months prior to the final plan completion, the Department received a temporary bridge structure from another project. This temporary bridge was made up of precast prestressed slab beams. Hardesty & Hanover was asked to see if the inventory of beams could be used to substitute for the prefabricated panel approaches. The beams were investigated and spans selected that could be incorporated into the plans with the fewest geometry changes on the temporary alignment. This was done and the plans were changed using the reused slab beams.

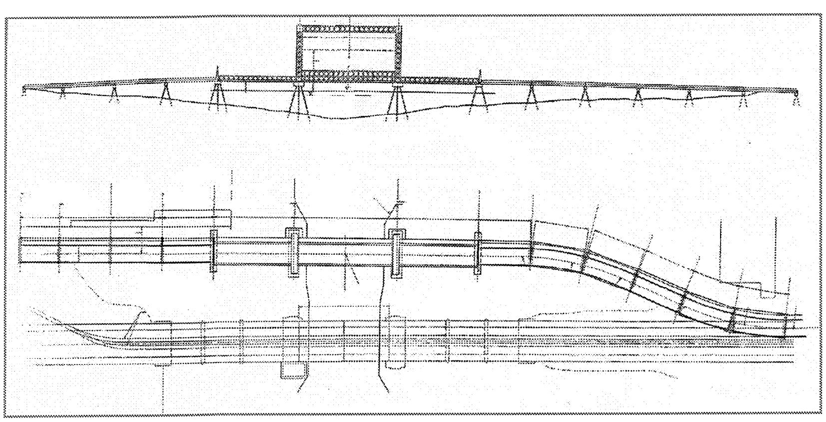

FIGURE 1: Temporary Bridge

For the movable span, two different suppliers competed for the project, bid as part of the project as a whole. Both companies were involved in the design phase as well, supplying information on the incorporation of proprietary details into a generic specification. Both firms used slightly different design philosophies but both were very similar in appearance.

The construction of the temporary bridge followed the Contractor's proposal closely. The lift towers were constructed in place. The longitudinal gantry was assembled on site, picked as a unit and installed on the transverse cross frames. These upper components supported the machinery, which was a rope driven winch system that could raise the span fully open in about 60 seconds. The span was lowered, essentially, by gravity. Brakes were available to slow the span, but the bridge was not powered down, as a typical lift bridge would be. The electrical system for the bridge was originally a two motor drive system which had a small low speed starting motor, which would be powered off, and a larger horsepower main motor would drive the span the majority of the distance. This system was replaced early on by a small variable frequency electrical motor control. The bridge traffic control and span operation were fully interlocked. That is, all signals, gates, and barriers as well as the span operation were interlocked for proper sequencing of operation to occur. For maximum safety, systems could not be casually overridden if a malfunction occurred.

The lift span was also assembled on site and picked in place. Here, however, the end bays were not constructed, allowing the picked assembly to fit between the tower legs. The end bays were installed with the center 75% blocked and supported. The roadway for the temporary bridge provided one lane of traffic in each direction. There was a full sidewalk for pedestrians outboard of the lift truss and separated from traffic by a barrier and fence.

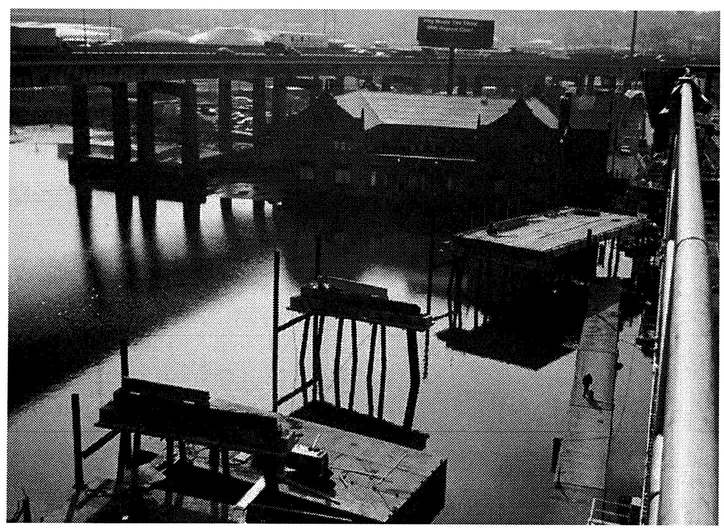

PHOTO: Temporary Bridge Removal

The temporary bridge performed satisfactorily. The bridge opened about 3000 times annually and functioned for 7 years with only minor problems. The motors have required several replacements and all the ropes and some sheaves required replacement in 1999. Under traffic the bridge performed well and while there was wear and some local damage, this bridge provided dependable service right up until the new bridge was in place and the Temporary Bridge removal began on March 4, 2002.

THE PERMANENT BRIDGE

The Tomlinson Bridge is one of the largest movable bridges in the United States. While not exceptionally long at 270-feet between bearings, the bridge is wide at 90-feet between trusses and is very heavy. The lift span weighs over 6.4 million pounds and the load to move at approximately 13 million pounds is exceptionally large. There are very few movable spans in this weight class and only several movable bridges, which have span weights near or at the 6 million pound category. Since most heavy spans are swing bridges, the span weight and load to move are the same. In this case, it is necessary to power twice the span weight.

The width of the bridge and the dual rail/highway loadings presented challenging design and detailing features. The bridge was designed to meet the then current seismic 1992 AASHTO design criteria. The bridge was also designed to accommodate significant vessel impact. While full AASHTO Guide Design loadings were not considered appropriate for this site, over 50% of the full vessel impact was used which resulted in increased fender capacity and incorporated filled cellular cofferdams in front of each lift pier. The bridge was designed structurally to meet both AASHTO and AREMA design criteria. AREMA design controlled loadings for the most part and most details and design requirements defaulted to this code throughout the project.

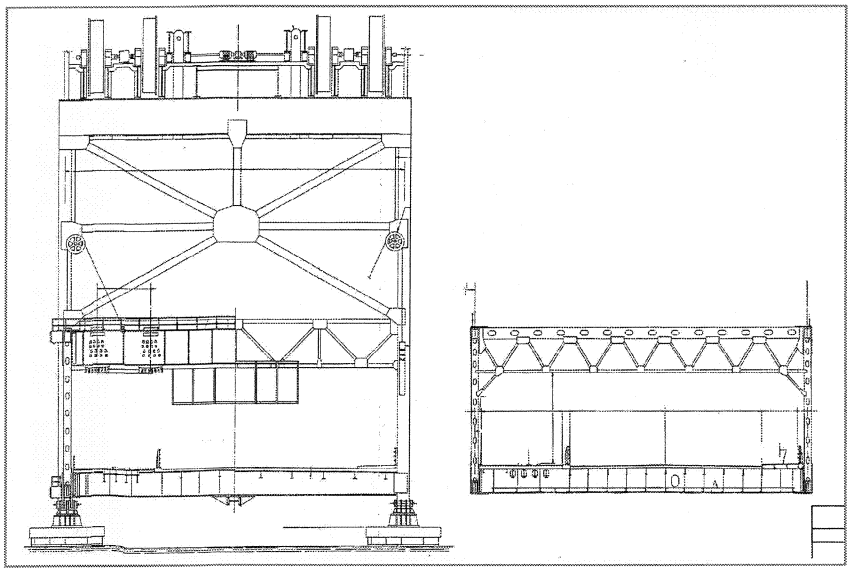

FIGURE 2: Lift Span – Typical Sections

The lift span type selected was a tower drive (See Figure 2). All the machinery is located on the towers and the span is driven through the main counterweight rope sheaves by ring gears mounted to the sheaves and driven through conventional gearing to a prime mover. In this case a 100 horsepower motor is required to drive the span in each tower. Each tower has 4 sheaves. Each sheave is 21-feet in diameter and each sheave supports sixteen 2-1/4-inch diameter wire ropes for a total of 128 ropes. The weight of the ropes themselves is counterbalanced by an auxiliary counterweight system to offset the weight of the ropes when they fall primarily on one side of the sheave. The sheave ring gears are driven, as noted, by conventional gearing.

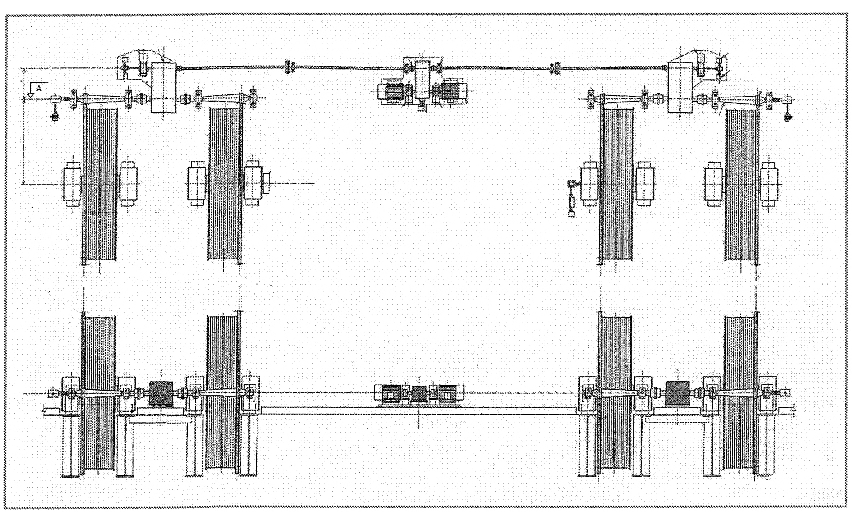

FIGURE 3: Machinery General Arrangement Plan

With the exception of the ring gear and main pinion, all other gearing is located in enclosed gear boxes. From the low speed end, two sheaves are located at each corner of the span and a main reducer is located between each pair of sheaves. The main reducer drives the main pinions. The main reducers/main pinion also have all skew control instrumentation mounted to them. The main reducer is driven by long shafting to the primary differential reducer, which equalizes torque to both ends of the span. There are two motors on each differential reducer (See Figure 3).

Only one motor is required to operate the span at any given time. The energized motor drives the other motor. The two motors are part of independent control systems, which are to provide operational redundancy for the span. Motor brakes are located on each side of the primary reducer and a machinery brake is on the back end of the main reducer input shaft. Electrically the main motor control is an AC SCR drive supplying preset ramped raising and seating with full regenerative motor capability. The control system itself is a hard-wired relay logic system. The primary electrical design consideration of the tower drive lift span is skew control. The selected method here was a synchro-tie transmitter. This system checks shaft rotations from each tower and corrects skewing by either requiring one side to speed up or slow down so that both ends of the span travel within an acceptable limited envelope, which defines level.

The approach spans for the permanent bridge are parallel welded plate girders. The highway portion of the bridge has a composite concrete deck and the rail portion of the span has an open timber tie track structure. The original concept called for a bridge to match the existing shoreline but after review it was felt that the channel should be opened up more. It was decided to increase the opening to almost 1000 feet - far shorter than the historic half mile noted in the 1700's but much better than the 500 feet of the existing 1929 bridge. The bridge is symmetrical with three approach spans on either side of the lift towers.

As we noted earlier, we increased the height of the bridge in the closed position to permit more ships to use the channel without requiring a bridge opening. The two grade approaches required different designs. On the west side there was room to use sloped embankments while on the east side we had to use long retaining walls. Over 500 feet of retaining walls were constructed on the east side of the bridge. The east side walls were constructed around a variety of existing structures and several were built around historically significant retaining walls for the Historic Yale Boathouse. Other walls were built around oil tank retainer dike walls. Also, the cofferdams constructed to build the retaining walls struck portions of the very old structures which previous plans stated were already removed. These obstructions consisted of timber mats, large stone riprap and other miscellaneous debris complicating construction.

For the approaches, the substructure foundations were two distinct types. The east side was completely founded directly on rock, which was located from 15 to 40 feet below the mudline of the channel bottom. Here 30-inch diameter rock-socketed caissons were utilized. On the west side, the ancient river bottom carved a very deep path in the local rock formation and subsequently filled in. The bedrock to the west was much deeper and at times steeply sloped. For the approach spans we used steel-H friction piles driven approximately 100 feet below the mudline. The piers were all standard solid concrete walls on concrete footings placed in the dry on a tremie pour. Granite stone facing was placed around all piers in the waterline.

For the lift piers, only rock-supported foundations were considered acceptable. The same 30-inch diameter caisson was to be used for the lift piers. Exceptional effort was spent locating the rock profiles of the lift piers, especially the west side, which was identified early as having a deep and sloping rock profile. The Contractor developed a combined pilot and drilled bit for seating the caissons. This device, which the Contractor called an inverted 'wedding cake', successfully seated the caissons so that drilling of the rock sockets could proceed. Each caisson had a 250-ton capacity in compression and there were 36 caissons per lift pier, one pier under each of the four lift towers. The lift piers themselves were constructed similar to the approach piers in that they included large solid concrete stems on concrete footings placed in the dry by use of a tremie pour. Granite stone facing was also used at these piers.

The lift towers for the Tomlinson Bridge are huge steel frames, which support the entire weight of the lift span in its normal condition. The towers are 90-feet 6-inches between tower columns in the transverse direction and 30-feet between columns in the longitudinal direction. The front and back leg of each tower is supported on the same pier. There is no transverse substructure element between the tower legs. The centerline of the sheave is almost 132 feet above the water line and, considering the architectural machinery room enclosure; the top of the tower is almost 160 feet above the waterline. The tower legs of a lift bridge are designed to carry the entire dead load as a freestanding structural element. There is no dead load in any of the tower laterals. These members are designed to take only lateral force, wind, seismic and some impact loads from motion.

The primary reason for this design concept was to insure that the fabrication of the tower leg provided a straight member that will be used for the lift span guides to travel against. Developing a truss member that met the straightness required for proper span tracking proved too difficult and was not recommended. Instead, the member was cambered only for dead load and nothing else. For the Tomlinson Bridge several special design features were incorporated. Due to the width of the tower, slightly over 90-feet, it was decided that the front transverse sheave girder would be designed as a king post truss. A king post truss is an assembly that acts both in bending and as the compression chord of a truss element, resulting in additional rigidity. Structural connections to tower members would also be easier to make by virtue of the reduced end rotations and accompanying ease for fairing bolt lines on the massive connections.

Some fabrication problems had to be overcome. The first fabricated tower base assembly had a twist in it, which affected connections over the entire length of the tower column. This was troubling since the significance of the twist was not discovered for some time. There were also problems with several connections of members where access to bolt lines was not sufficient. These issues were all addressed forthrightly and the intent at all times was to solve problems and not create new ones in the process. The structure is a very complex one combining the need to carry very large structural loads which move, as well as the mechanical components, which require more precision in erection and alignment than any typical structural member. Also, the towers must house all the electrical components, their conduit and wiring.

Making the Tomlinson Bridge even more complex was the fact that the architectural requirements were very challenging. The bridge towers were originally to be open steel towers but discussions with local groups resulted in the towers clad with a stainless steel panel type enclosure. The towers are intended to look as grand arches welcoming travelers as they cross over the Quinnipiac River. The panels were specially patterned matte-finished to reduce glare and the front portions were colored to reduce the effect of grease splatter from the adjacent counterweight ropes. The need to wrap the panels around existing members to form the arch proved difficult but ultimately successful. The inclusion of cladding required that all vertical utilities be placed behind the panels.

A painted roof blended into and over the panels, taking its basic shape from the sheaves underneath. These machinery rooms had to blend in with the panel aesthetics as well as provide room for sheaves, overhead cranes, and other equipment. Another problem addressed was that the coefficient of expansion for the carbon steel structure and the stainless steel enclosures was sufficiently different to require transverse expansion of the enclosure relative to the frame. Support details had to account for this movement which, while not critical to the performance of the tower, could have considerable effect on the appearance of the enclosure panels.

The east tower contained the primary electrical room. The control room hung below it and was just above the traffic clearance envelope. This control room location provided the best sightlines possible for a cladded structure as wide as this one. The use of CCTV was required to supplement the operator's vision.

The west tower contained an equipment room. The two rooms were constructed within the forward mid-level truss structures, which were the portal frames of the tower. Again, the architecture, conceptually developed by the City of New Haven, was a challenge for the houses, as they required many curved and blended shapes from stainless clad panels.

The lift span for the Tomlinson Bridge was also a challenge for the Hardesty & Hanover design staff. The structure was very wide and the loadings from the traffic side to the rail side were different. The design had to incorporate this difference so that simultaneous traffic could cross the bridge. Just as difficult were the detailing considerations of the structure so that fabrication would be optimized and erection simplified. To do this, an attempt was made to keep the dead loads from side to side as equal as possible so transverse dead load forces and span balance would be optimized. It was possible to keep the two sides in fairly close balance so that the rail side dead load and the span side dead loads required no special detailing. For considerations of the live load, the behavior of the spans under combined and separated loadings was evaluated. Once it was understood how the span was going to behave under live load, it was decided that both trusses would be designed the same. This resulted in simplified camber and fabrication requirements. There was some minor material excess but the material costs were low compared with the labor cost of fabricating and erecting the truss.

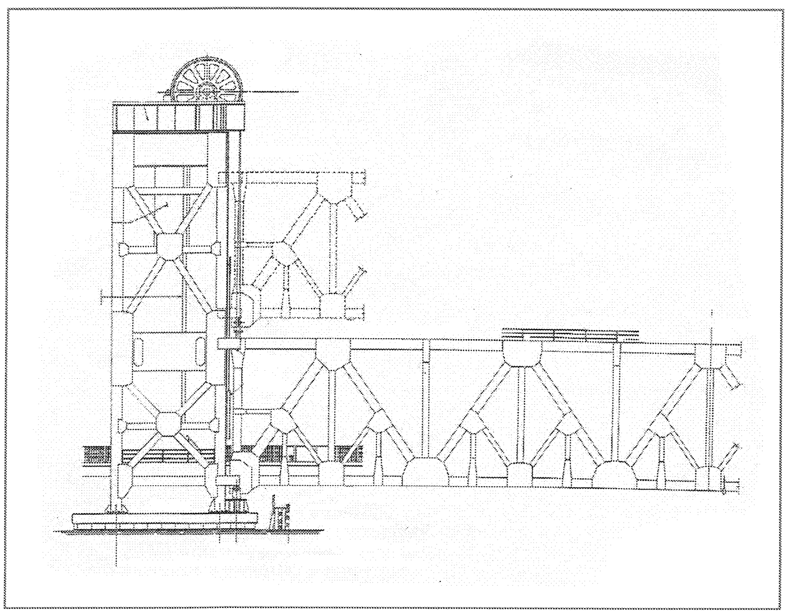

FIGURE 4: Tower and Lift Span Elevation

The lift span is a subdivided Warren through truss span (See Figure 4). The selection of a Warren truss was made because it is the most efficient fixed truss type. The subdivided panels are a direct result of minimizing the deck and floorsystem depth so that more under clearance for marine traffic could be realized. The chords and web members are all welded boxes with regularly spaced hand holes for access to diaphragms and connections. The truss is 40-feet deep between chords with a typical floorbeam spacing of 13-feet 6-inches.

To accommodate the combined highway and rail traffic, two separate deck systems were designed for the span. The highway deck is a 5-inch deep steel grid with a monolithically placed microsilica deck filler and one and a half inch overfill supported on steel stringers spaced at 5-feet 3-inches. The original deck fill material selected was latex modified concrete, which was later changed to a microsilica concrete. The deck has expansion relief joints at the third points to accommodate the span deflections under live load.

Adjacent to the southern side of the roadway is a 10-foot wide sidewalk made of a 2-inch thick concrete filled steel grid. For the rail side an open timber tie track structure was used with an orthotropic steel plate walkway. Below the floorsystem there is a series of access walkways, both transverse and longitudinal, so that access could be made to almost 100% of all steel members. The roadway side has a 4-rail aluminum railing with aluminum ballastrades to give a picket fence type appearance.

For the design of the span we looked carefully into the AREMA live load camber requirements, which call for full dead load camber plus 3000 pounds per linear foot live load camber. For this bridge we decided that we needed to only develop half the live load camber for the rail since the frequency of the rail loadings would be very low and the effect would be less than a structure carrying rail load only.

The lift span was designed with a heavy portal system to accommodate the large width and heavy loads. This portal frame system, combined with a heavy upper and lower lateral system, was essential to keep the span square and true under live load. The span was detailed with the rail stringers milled to the floorbeam webs and the floorbeams milled to the truss connections as per AREMA criteria. The roadway stringers were conventionally detailed so that the members could be swung into position with the floorbeams erected. The Contractor was initially concerned that the stringers would be difficult to install due to the uncambered stringer length and the shortened bottom chord but this proved not to be of any concern as the floorbeams were easily manipulated to permit connections to line up. Once full dead load was on the span, the actual plan length, geometric shapes and distances would relieve any erection stresses as a result of floorsystem members being forced into position.

Likewise, it was necessary to get all parties to agree that the same concept of forcing cambered members into their no-load position is correct for the truss. Some thought that truss members should be fabricated to the span profile camber geometry, but the effect of that would be to introduce secondary bending stresses into these members, which would reduce live load capacity and place unaccounted for bending stresses in members. Instead we required the truss members to be forced into position while unloaded. We were confident that when the full dead load of the truss was in place, the truss geometry would match the plan geometry, and the dead load secondary stresses and live load affects would be eliminated. The effort to achieve this was less than anticipated for the members throughout the bridge and has been achieved historically since the initial pinned trusses were replaced by riveted truss connections as far back as the late 1800's.

CONSTRUCTION

The construction of the Tomlinson Bridge took over eight years. The original Contractor could not complete the project and a significant delay resulted. Cianbro completed the actual erection of the bridge superstructure and all mechanical and electrical work. The construction and fabrication of any large steel structure is an impressive feat. This was no exception. The steel fabricator, National Eastern Corporation, had not done any welded box truss bridges before and their initial efforts were spent developing box element fabrication jigs and other fabricated devises. The towers and truss members were fully laid down in the full chord assembly method. One chord of the truss at a time is laid down with all web members aligned and all bolted connections reamed to full size using final geometric angles. After the one chord is finished, the second chord is laid down with its web members aligned and then reamed. This method results in temporary erection stresses to align members prior to the full dead load being realized on the span. However, it eliminates secondary stresses resulting in the fixed truss connections in the final loaded position.

The lift span was erected on a "flexi-float" barge system, which is a system of prefabricated barge segments jointed together. On top of the barges a 10-foot deep erection truss was erected which was used to obtain more height above water as well as stiffen the "flexi-float" system. The erection barges were not as adjustable in height above the water as what would be ideal, but they provided a stable enough platform to erect the permanent trusses in an economical way. The float-in would have to use extreme high tides to clear the piers and lift span base weldments but this was well within normal tolerances for a float-in of this type. The Contractor utilized a Manitowoc 4100 Ringer Crane positioned on land as well as another Manitowoc 4100 Ringer Crane positioned on a barge for lift span erection.

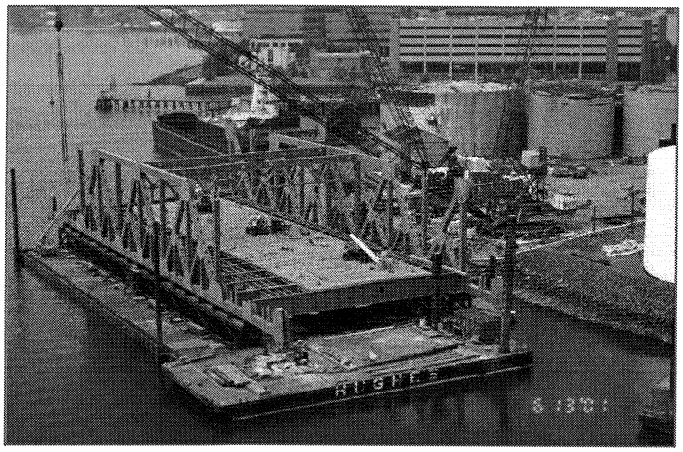

PHOTO: Lift Span Erection

The towers were also shop aligned and reamed in the full chord method so that all geometric angles from the plans were maintained. As with the lift span, the object was to eliminate secondary stresses in the final loaded condition. The issues for the towers were the erection of the transverse sheave girders and king post truss. These transverse members were large, and since they were designed with camber as well, they needed some significant force to be applied so that all connection holes would fair in the no load erection condition.

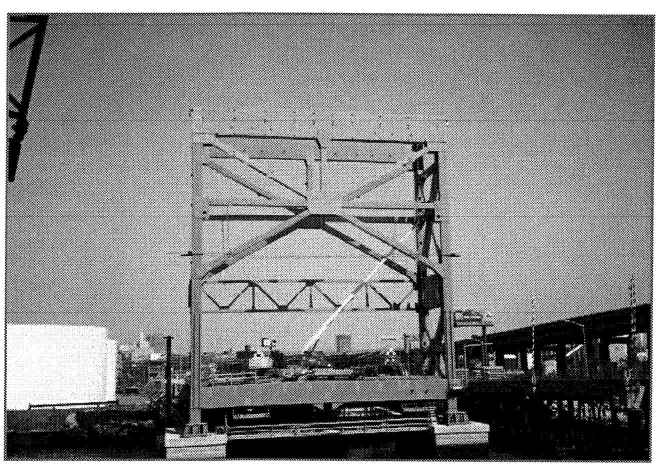

PHOTO: Transverse Sheave Girder Erection

The erection of the transverse sheave girder required the Contractor to install a temporary jacking strut at the lower portion of the column, to ensure that the columns rotated outward at the top and that the end rotation camber of the transverse sheave girder matched the shape of the deflected column. This maneuver was extremely successful and the Contractor installed all the elements with relative ease given their size and height above the ground.

After the girder was installed, the next step was developing the proper loadings in the king post truss elements. To make up the connections, it was necessary to use the weight of the filled counterweight to force the members to fair bolt holes. This procedure was done using calculated dead loads at the tower as the dead load of the counterweight, on it's temporary support hangers, brought members into alignment. The construction required significant thought and the Contractor and Hardesty & Hanover formed a close working relationship throughout the project in order to move the construction along to meet very tight schedules.

It was interesting how close the calculated stresses or deflections came to the actual loads measured throughout the erection of both tower and lift span. While not exact, the order of magnitude for such aspects as pulling members into alignment or jacking members to make up connections was quite close.

CONSTRUCTION – FLOAT-IN

On Monday, October 15, 2001, at about 2:30 PM the outgoing tide dropped the construction barge clear of the Tomlinson Lift span. The heaviest movable span designed by Hardesty & Hanover (nearly 6.5 million pounds) was 'swung' (support conditions changed from temporary erection to permanent condition) without a sound as the span sat comfortably on the tower bearings while the construction barge was pulled from underneath.

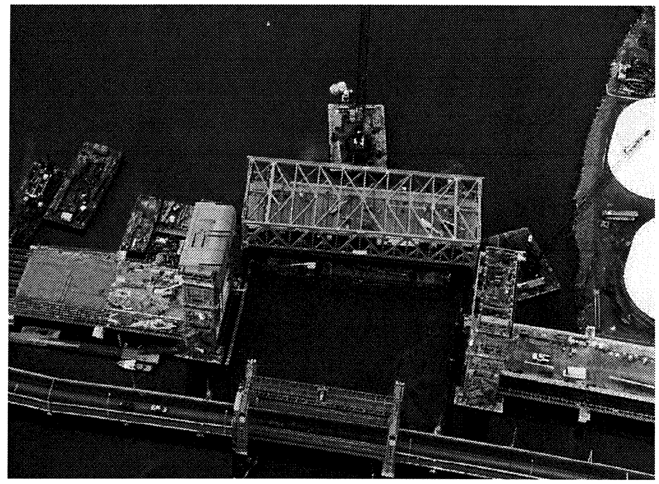

PHOTO: Span Float-In

By the end of the five day navigational closure, the span was attached to the 128 - 2-1/4-inch diameter steel counterweight ropes, the deck concrete was placed, and the span was raised to the open position under back-up power.

CONSTRUCTION – OPEN TO TRAFFIC

On Monday, February 25, 2002, the Tomlinson Bridge was opened to vehicular traffic. At this time, the structure had two lanes of vehicular traffic running in the east-west direction. Bridge operation was under full electrical power with the control system construction now complete.

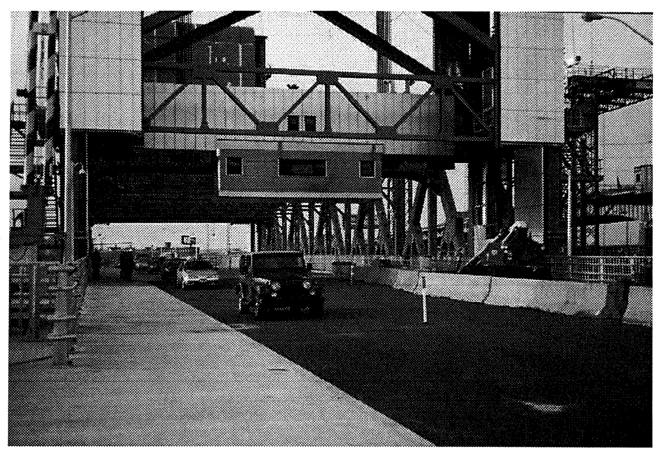

PHOTO: Bridge Opened to Traffic

The movable span of the temporary bridge was floated out overnight on Monday March 4/5, 2002 and, after 7 years of use, the remainder of the "temporary bridge" was removed by the end of May 2002.

On August 12, 2002, the bridge was opened to four lanes of vehicular traffic, with pedestrian access at the 10-foot wide sidewalk and the trackwork completed for access by rail.

During Stage 4 Construction, in addition to the construction of the northeast approach tie in, the major construction items to be completed include the fishing pier, fender system, filled cellular cofferdams for pier protection, railroad work, and cladding installation at the north and south faces of the tower structures. This work should be completed by the end of this calendar year.

In conclusion, the design and erection of the Tomlinson Bridge was both a challenging and rewarding project for all involved. Classic theories were re-affirmed, new methodologies were used for design, fabrication and erection, and the bridge operated flawlessly during its initial trials. These accomplishments certainly make this project a successful one by anyone's definition.

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Contractor at Completion:	Cianbro Corporation
Key Subcontractors:	National Eastern Corporation
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Construction Inspection:	Parsons Brinckerhoff Construction Services
Designer:	Hardesty & Hanover, LLP