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Modjeski and Masters
"Replacing Swing with Vertical Lift to Modify RR Bridge"

REPLACEMENT OF SWING SPAN WITH VERTICAL LIFT SPAN
TO MODIFY RAILROAD BRIDGE OVER THE RED RIVER WATERWAY

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ABSTRACT

As part of the U.S. Army Corps of Engineers' project to improve navigation on the Red River Waterway, the Union Pacific Railroad's existing bridge at Alexandria, Louisiana, including a swing span, required modification. A vertical lift bridge was designed to replace the swing span with minimal interruption of rail and marine traffic service during construction. Construction began in November 1990 and the bridge modifications are expected to be complete in November 1992.

This paper addresses key structural aspects of the design and construction. The new piers for the lift span and towers encase the existing rest piers of the swing span, and are founded on rock near the riverbed surface. The rock prevented pile driving, so special techniques were needed to seal the cofferdam sheet piles to the riverbed. A sequence of excavation and concrete placement was used to construct the new pier footings in segments without jeopardizing the stability of the existing rest pier footings carrying rail traffic. The swing span was temporarily modified including removal of its end panels so that it could remain in service in its shortened configuration while the slender lift bridge towers and counterweights were constructed atop the new piers. The existing approach spans to the movable span were unaltered by the project. The swing span was replaced within a rail traffic closure period of 12 hours, as required. The contractor's successful span change-out method included simultaneous float-out of the swing span and float-in of the lift span by supporting both spans on the same sets of falsework and barges. The steel, through-truss, lift span is 306 feet long.

GENERAL PROJECT INFORMATION

Many past projects have included the replacement of movable bridge spans, for various reasons, while maintaining traffic during construction. However, this type of project requires specialized techniques, and each project has its own unique set of characteristics and circumstances. That specialization along with the wide variety of engineering, construction, and other disciplines involved, and the relatively short time available to coordinate and perform the work, is what makes these projects so interesting. The intent of the following presentation is to highlight some of the key structural aspects of the recent modification of the Union Pacific Railroad's bridge over the Red River at Alexandria, Louisiana. The U.S. Army Corps of Engineers administered the design and construction contracts. Johnson Brothers Corporation was the General Contractor for construction. Union Pacific Railroad forces also performed portions of the work. The author was the on-site design firm representative during construction.

The existing, single track, bridge structure before modification included an open deck, steel, through-truss, swing span about 358 feet in length. The open deck portion of the west approach consists of two steel, through-truss, simple spans, each about 129 feet long, plus a 75 foot long, steel, deck girder span. The open deck portion of the east approach consists of three steel, deck girder spans, each about 60 feet long. The open deck portion of each approach is joined to a series of precast concrete, ballasted deck spans over land. The total length of the crossing between abutments is about 1067 feet. The existing bridge's history includes many past rehabilitations and partial replacements, so its components are of various ages. The oldest parts are some of the original concrete piers which date back to the late 1800s. The swing span, before its replacement, provided a channel for navigation on each side of its center pivot pier and fender. Each channel had a horizontal clearance of about 158 feet.

To improve navigation, the existing swing span and its pivot pier and fender were recently replaced by a new vertical lift span. The geometry existing at the site and required for navigation allowed for the vertical lift span with its towers to be designed to fit entirely between the existing rest piers of the swing span. The existing approach spans were not modified. The steel, through-truss, lift span is 306 feet long (center to center of bearings), 50 feet deep at mid-span (center to center of chords), and provides a navigation channel with 286 feet of horizontal clearance between the pier protection cells. The bridge provides about 65 feet of vertical clearance for marine traffic with the span in the normal full open position and the river stage at the present normal pool elevation. The normal lift of the span is about 38 feet. With the span seated, the low steel elevation of the lift span is about the same as it was for the swing span. Rail elevations across the bridge also remained generally unchanged by the project except for a slight increase due to changing from 119# to 133# rail on the open deck spans. Two new deck girder spans, 22.5 feet long from center to center of bearings, provide the transitions from the ends of the lift span, through the towers, to the approach spans. The tower girder spans bear on the existing swing span rest piers toward the approaches and on the new tower piers toward the lift span, and are not connected to the tower structures. Both towers are about 104 feet high from pier top to centerline of the counterweight sheave bearings on the tower top. The tower width, parallel to the track, is 15 feet between centers of the front and rear tower columns. The dimensions of the counterweight boxes, which travel vertically within the tower structures, are 9 feet by 21 feet by 30 feet deep.

The design and construction specifications for the project generally followed the American Railway Engineering Association Manual, including fracture critical member and Charpy V-notch impact test requirements for structural steel. The structural steel is mostly ASTM A588 Grade 50 weather-resistant steel, without paint. Miscellaneous metalwork including walkways is painted ASTM A36 steel. Trusses and tower columns are mostly welded box members with bolted end connections. Welded plate girders were fabricated for floorbeams, lifting girders, and tower top girders. Stringers, girder spans, and bracing members were fabricated from rolled shapes. Bolted joints are mostly friction connections using ASTM A325 high strength bolts. Cast-in-place concrete for the piers, counterweights, and pier protection cells was required to have 3000 psi minimum 28 day compressive strength.

The project required both rail and marine traffic to be maintained during construction, except for occasional, scheduled interruptions of short duration when necessary. A maximum rail closure period of 12 hours was permitted to remove the swing span and install the lift span. Also, a maximum marine closure period of 7 days was permitted for the span change-out operation during which time the new lift bridge was to made operational.

PIER CONSTRUCTION

Construction started in November 1990. The first major work on the site was construction of the cofferdams and two new piers. Due to the proximity of the towers to the existing swing span rest piers, one large pier was designed to encase the existing rest pier and support the tower and lift span at each end of the new lift bridge. Plan dimensions of the octagonal pier shafts are 36 feet by 39 feet. Cofferdam and pier construction was mostly performed during the period from February 1991 to March 1992, with the construction sequence of the second pier lagging the first by about two months. Progress of the substructure work was hampered by a flood in the Spring of 1991 and a marine vessel collision with the second pier's cofferdam in the Fall of 1991.

The new pier foundations were designed as spread footings, similar to the existing pier foundations, only much larger. Although the dimensions vary between the two piers, the footings are approximately 50 feet square by 10 feet deep. Founding elevations of the footings vary from 26 to 34 feet below the river's present normal pool elevation. The footings bear directly on a claystone/siltstone material near the riverbed surface. This rock material prevented pile driving. Construction of the new footings, which encapsulate the existing rest pier footings, required special techniques. Excavation of soil and rock to the required founding elevations, slightly lower than the existing footing elevations, and placement of the footing concrete, were required to be done within dewatered cofferdams. Since the sheet piles for the cofferdams could not be driven into the rock, the contractor used an auger to drill overlapping 2 foot diameter holes, a minimum of 3 feet deep, around the perimeter of each cofferdam frame to form a trench. The sheet piles were then set and their toes sealed and anchored to the river bottom by placing tremie concrete in the trench. Also, to maintain stability of the existing pier foundations which continued to carry rail traffic during pier construction, a sequence of excavation and concrete placement was used to build the lower lifts of the new footings in segments. Rock was excavated in only one or two small segments at a time before those portions of footing concrete were placed. A shelf, varying from 2.5 to 5 feet wide, of undisturbed rock was left around the perimeter of each existing footing. The lower footing lifts were placed in 8 segments each, with no two adjacent segments having been excavated before concrete placement. Threaded couplers were used to splice reinforcing steel across the vertical construction joints in the footing lower lifts. The upper lift of each footing was placed in one continuous pour. A problem encountered during the footing construction was control of water weeping upward through natural seams in the founding material, and in a couple cases through earlier bore holes, into the excavated footing segments. At the first pier, the solution consisted of encasing a pipe vertically through a footing segment as a relief passage for the water during concrete placement. The pipe was later capped, and remained within the footing. At the second pier, the weeping problem was worse, and seemed to increase as more concrete segments were placed. A drain system and impermeable sheeting were installed on the rock surface to relieve the water prior to placing concrete in the last two segments.

The pier shafts were constructed atop the footings in 7 lifts each. The lifts varied in depth from 4 to 8 feet except that the cap pours were about 1 foot deep. Repetitive work performed to prepare for placement of each lift included scaling existing pier shaft surfaces to sound concrete, drilling and grouting reinforcing dowels into the existing shafts, raising formwork, installing reinforcing steel, pressure washing construction joint surfaces, and applying epoxy bonding agent to the existing shaft surfaces. Before the shaft construction, to assure that the existing shafts would be satisfactorily joined to the new shafts, tension tests were performed in the field on sample dowel bars grouted into the existing shafts. Those tests indicated that the design dowel sizes and spacing, the existing concrete strength, and the grout anchorage were adequate. The existing shafts were also reinforced by installing prestressing strands to tighten existing steel plate bands around the shafts. The plate bands had been installed in the past due to vertical cracks in the shafts. Bearing grillages and anchor bolts for the tower and lift span bearings were embedded in the upper lifts of the shafts. The caps of the existing rest piers remained exposed above the new pier caps, leaving the existing approach span bearings undisturbed. The temporary cofferdams were removed after the piers were built.

SWING SPAN MODIFICATIONS

Structural steel for the bridge superstructure was shop fabricated mostly between April 1991 and February 1992, while the piers were being built. Just before the piers were completed, retrofit work was performed on the ends of the existing swing span. That work was necessary to shorten the swing span so that it would continue to carry rail traffic and have the clearance to open for marine traffic during erection of the towers on the piers and until the lift span was installed. A full truss panel needed to be removed from each end of the swing span, and new temporary truss bearings needed to be established at Panel Points L1 of the swing span. The design drawings offered details and a construction procedure for the temporary swing span modifications which were basically adopted by the contractor. The swing span modifications required the conversion of the original U1-L1 truss hip hangers to span end compression posts. That was accomplished mostly by adding a channel section to each flange of the original U1-L1 truss members. To equalize stresses in the new and old components of the modified members, jacks were used to apply a portion of the panel dead load under the P.P. L1 floorbeams while tightening the bolts connecting the retrofit channels to the U1-L1 members. Retrofit portal bracing, stringer diaphragms, and span locks were also installed at Panel Points 1. Since the locations of the final lift span bearings were close to the swing span Panel Points L1, the lift span bearing grillages were designed to accommodate the temporary swing span bearings needed at P.P. L1. Some of the same anchor bolts were used for both the temporary truss bearing bolsters and the final lift span bearings. In April 1992, when the swing span retrofit work and the new pier work was ready, both end panels of the swing span were cut off, at the same time to maintain span balance, within about a 6 hour rail closure period. Calibrated jacks were used for this operation at Panel Points L0 and L1 to relieve the dead loads in the bottom chords and end posts in the end panels before they were cut and removed. The jacks were also used after removal of the end panels to verify that the dead load reactions of the temporary bearings established at Panel Points L1 were satisfactory. The stringers and truss laterals were also cut in the end panels near Panel Points L1, and the shortened swing span was then operable for either rail or marine traffic. The end panel floor system units remained in place on the piers to carry rail traffic until the next day. Temporary supports had been installed under the cut stringers, making temporary use of some anchor bolts that would later be used for the span locks of the lift span. On the next day, within another 6 hour rail traffic closure period, the end panel floor system units were removed and replaced by the new tower girder spans. The girder spans were erected on a barge before the rail closure period with most of the trackwork complete and spliced with temporary girder extensions to make the transitions from the permanent tower girder spans to the swing span.

TOWER AND COUNTERWEIGHT CONSTRUCTION

Tower erection started immediately after the tower girder spans were in place on the piers, with erection of the second tower lagging the first by about one month. Many of the tower members were assembled on barges and then erected as larger units with the cranes. Similarly, the steel plate counterweight boxes with their bracing and lifting girders were erected on barges and then set into position within the partially erected tower structures. The tower tops were designed with jacking beams and eyebar hangers capable of supporting the final weights of the counterweights, estimated to be about 475 tons each including structural steel, concrete, ballast rails, wire ropes and balance blocks. When tower erection was sufficiently complete, the counterweight boxes were suspended from the eyebars and filled with concrete. The design required a minimum concrete density of 144 pcf. Months before placing the counterweight concrete, test blocks were made to estimate the wet density and dry density of the concrete mix. A variable quantity of ballast rail sections, depending upon the concrete density, were also required to be placed on shelves in the boxes and embedded in the concrete for additional weight. During the actual pours of the counterweight concrete, the wet unit weight of the mix was monitored by taking samples. The average wet density as determined by the samples was about 146 pcf. For final weight adjustments to balance the span with the counterweights, pockets were provided in the tops of the counterweight boxes to contain cast iron balance blocks, weighing about 98 pounds each, ideally to account for about 4.5% of the weight of each counterweight. Before the span change-out operation, the weight of each constructed counterweight was also measured with two 500 ton calibrated jacks using the eyebar/jacking beam assembly designed into each tower top.

The sequence of building the piers, modifying the swing span, and erecting the towers and counterweights was critical to prepare for the span change-out operation. Counterweight sheaves and wire ropes, and electrical power and control circuitry, also needed to be installed on the towers before the new lift span could be installed. The other generally parallel work path to prepare for the span change-out consisted of fabricating and erecting the lift span, railroad trackwork, operator and machinery houses, drive machinery, and the electrical control system. Since this vertical lift bridge is a "span drive" system, as opposed to a "tower drive" system, most of the operating machinery and electrical controls, and their houses, are centrally located at the top of the lift span. Operating ropes running along the top chords link the drive machinery to the towers. Preparation work on the lift span and the towers finished up at about the same time for the span change-out operation.

LIFT SPAN ERECTION AND THE SPAN CHANGE-OUT OPERATION

An integral part of the method of erecting the lift span was the contractor's method of replacing the swing span with the lift span. The plan was developed early in the project, and it did raise concerns with parties involved in the contract. The basic concerns were for possible increased risks due to the plan to float-in the new span from the upstream side of the existing bridge, and to have both the new lift span and the old swing span supported on the same falsework/barge assembly at the same time. However, the change-out method did later prove to be engineered and executed well. The lift span was erected atop the same falsework/barge assembly that would be used later to change-out the spans. Steel erection started in January 1992 when the barge shipment of fabricated lift span steel arrived at the site. Use of an old river bendway separate from the main river channel, just upstream of the existing bridge, as the site to erect the lift span provided protection during erection against river currents and traffic. The falsework/barge assembly included four barges arranged in two pairs. The span was mostly erected as two separate halves, with one half on each barge pair, and then the halves were joined at mid-span. The low steel of the erected span on the falsework towers was about 27 feet above water level, depending on the ballast level in the barge compartments. That height would put the lift span close to its final installed elevation on the piers when the river stage was close to the present normal pool elevation. Each barge supported three steel falsework towers. The arrangement provided for each lift span truss joint at Panel Points L2, L4, L4' and L2' of the 12 panel span to be supported on a falsework tower. Two of the three falsework towers on each barge were used for those 8 lift span support points. The third tower on each barge was for support of one side of the swing span during the span change-out operation. The other side of the swing span shared the center falsework tower on each barge with the lift span, leaving about 5.5 feet clear between the adjacent spans when both were floating on the barges. Since the panel points of the swing span did not align with those of the lift span, additional diagonal truss members were added to one side of the swing span before the change-out operation to create bearing points for the span on the shared center falsework towers.

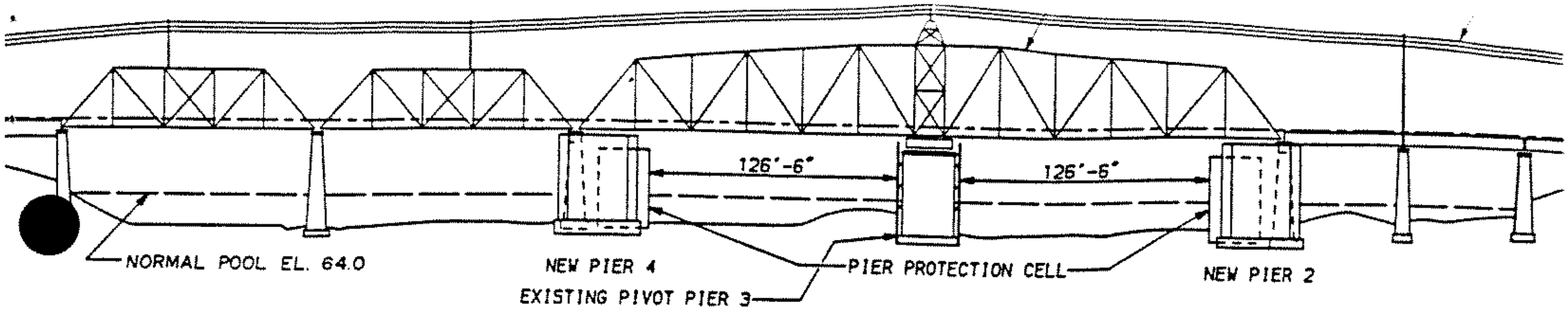
The 7 day marine traffic closure period for the span change-out operation started on August 2, 1992. On that first day, the lift span on the barges was moved downstream with tug boats to a position alongside the swing span. During that journey, cables from an upstream hoist were attached to the falsework/barge assembly after it moved into the main river channel in case additional control would be needed. Also, as the assembly approached the upstream nose of the pivot pier fender, bracing members and cables between the two barge pairs were removed to allow one barge pair to move into each of the swing span's navigation channels while the lift span straddled the fender walls. On the second day, the swing span was removed and the lift span was installed on the piers within the 12 hour rail traffic closure period. The spans were interchanged through a precise sequence of adding and removing ballast water from the various barge compartments. The ballasting and deballasting procedures raised and lowered the barge assembly as needed to move under and pick up the swing span, and then set down and move out from under the lift span. Level barges needed to be maintained throughout the operation as the barge loading cases changed. The weight of the lift span was estimated at about 950 tons, the weight of the swing span at about 550 tons, and each barge with its falsework at about 300 tons. Each barge pair was equipped with a system of submersible pumps powered by diesel generators and complete with interconnected pipelines and valves to each barge compartment. Each barge compartment also had a graduated rod for monitoring the ballast level. The crew was coordinated well for the ballasting operation and the system performed efficiently. Due to the limited amount of elevation change that the ballasting operation could provide, the river stage had to be within about a two foot range to be able to obtain the maximum and minimum elevations needed to replace the spans. Therefore, the river stage, which varied during the project within a range of 17 feet including a flood period, had to be close to the present normal pool elevation for the change-out operation. Replacing the spans at higher river stages would have been undesirable anyway due to effects of increased river current velocities. The river almost caused a problem when the stage rose to the maximum level during the 12 hour rail closure period, making removal of the falsework/barge assembly from under the lift span more difficult than expected. Other major work items performed to change the spans and restore rail traffic during the 12 hour period included replacing the swing span bearings with the lift span bearings on the piers, replacing the temporary tower girder span extensions with shorter permanent girder extensions, attaching counterweight ropes to the lifting girders of the lift span, and installing the rail joints at the ends of the lift span.

After rail traffic was restored, other work was required to make the lift span operational during the remainder of the 7 day marine closure period. That work included installing span locks, guide rollers, and centering devices. Connections and adjustments of operating ropes and the electrical system were made. The counterweight loads were transferred to the ropes using the jacking systems in the tower tops. The weights of the counterweights were also adjusted to balance the span with the counterweights and obtain the design pier reactions as determined by calibrated jacks at the lift span bearings. The lift bridge was operated for the first time on the sixth day for test runs. The bridge was put into service after the seventh day, although additional system adjustments continued.

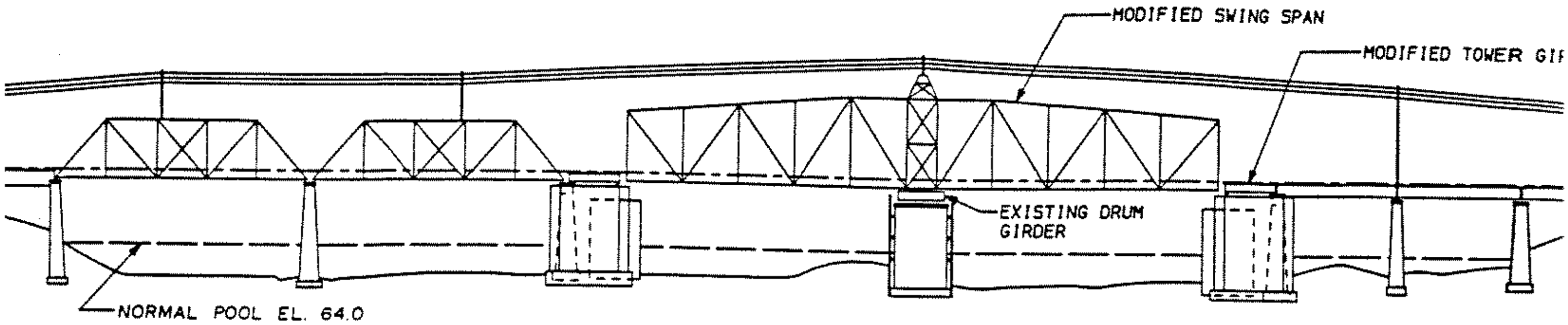
PROJECT COMPLETION

During the last several months of the project, work continued to finalize the structure and system operation. Other major work that followed the span change-out operation included demolition of the existing pivot pier, fender, and swing span. Three of the four pier protection cells were also built after the change-out. The 25 foot diameter cells consist of sheet piles filled solid with concrete. Riprap was also required to be placed around the new piers. The project is expected to be completed in November 1992, about 7 months ahead of schedule. The project was a good example of many disciplines successfully working together.

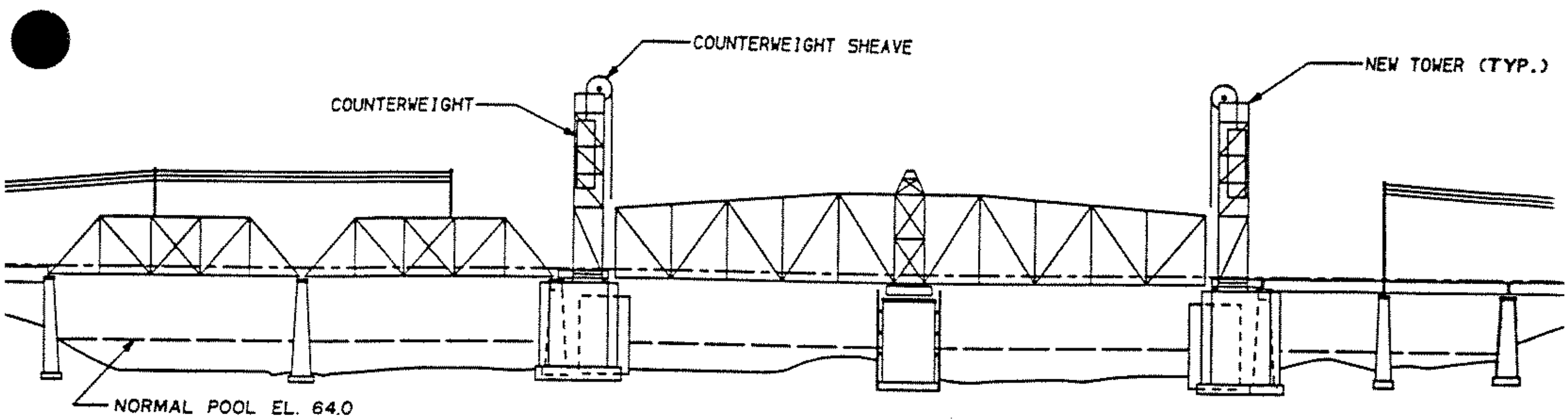
APPENDIX



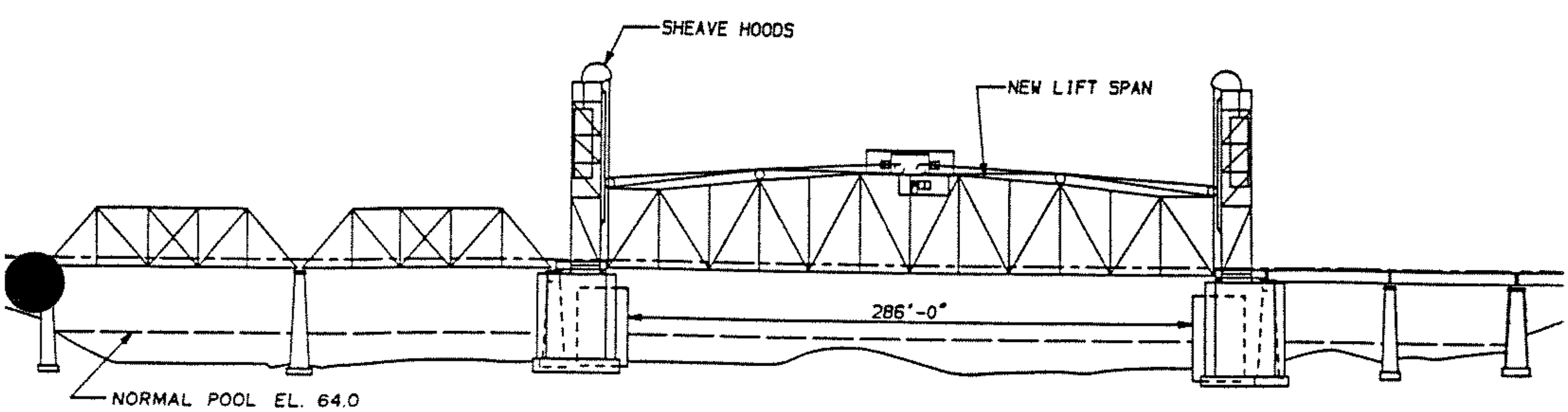
STAGE I
(LOOKING NORTH)



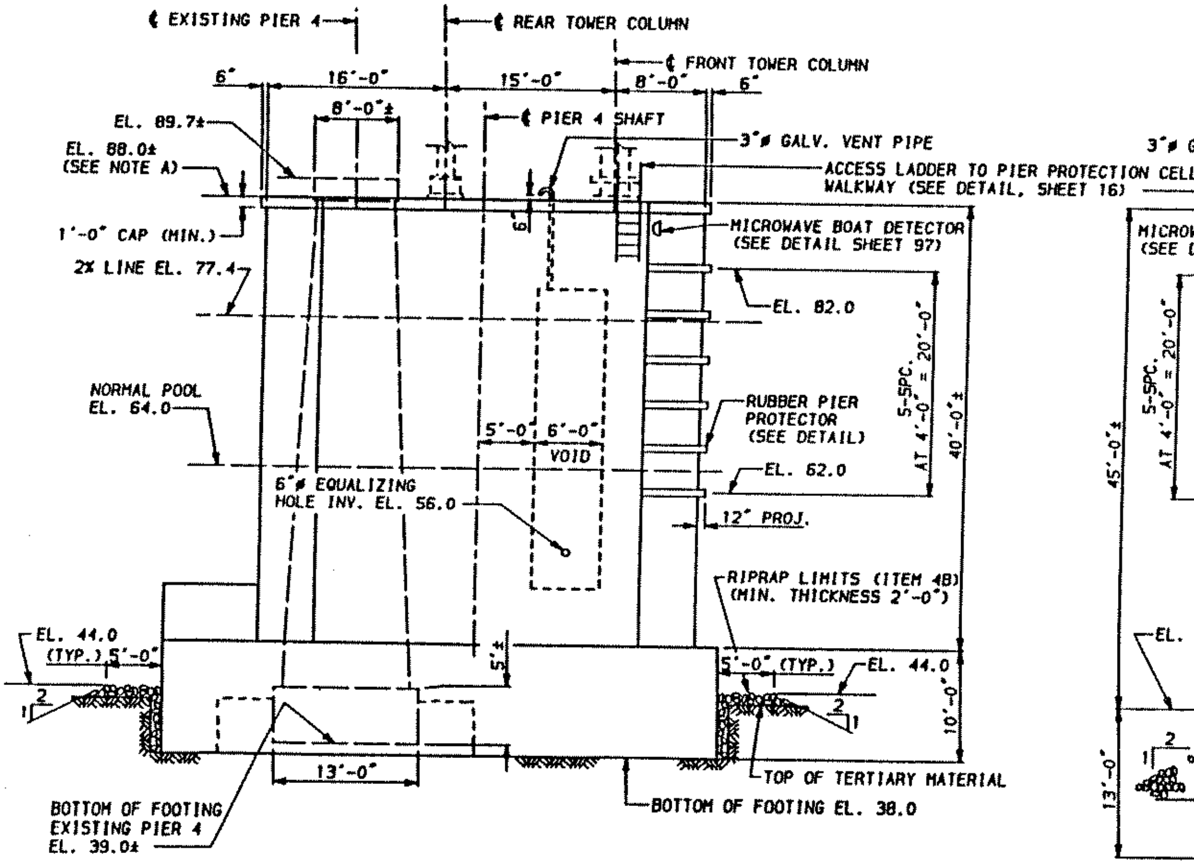
STAGE II



STAGE III



STAGE IV

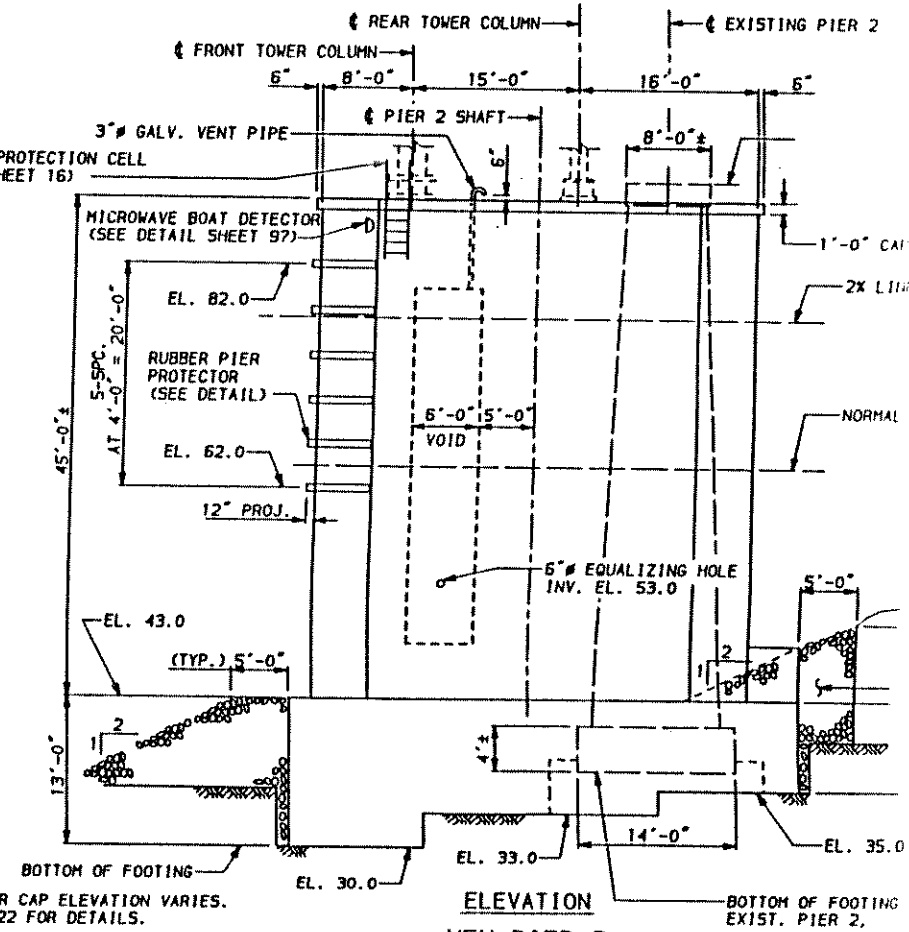


ELEVATION

NEW PIER 4

SCALE: 1/8"=1'-0"

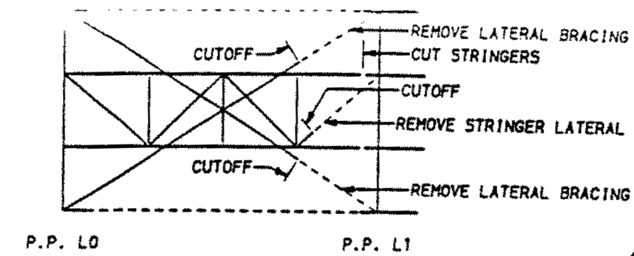
NOTE A: TOP OF PIER CAP ELEVATION VARIES.
SEE SHEET 22 FOR DETAILS.



ELEVATION

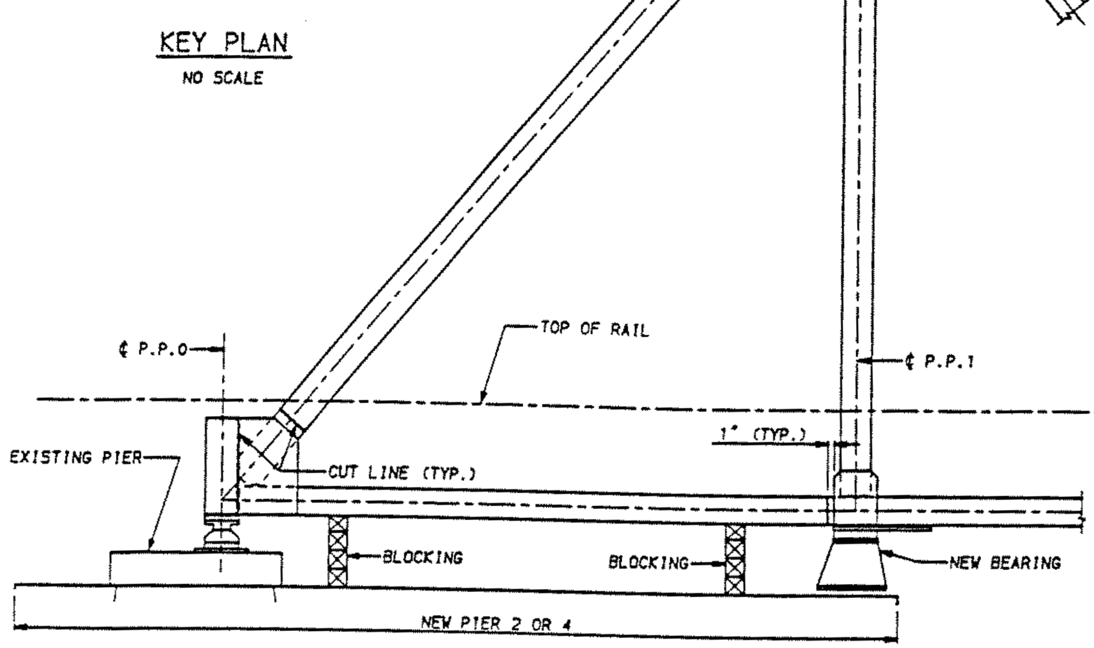
NEW PIER 2

SCALE: 1/8"=1'-0"



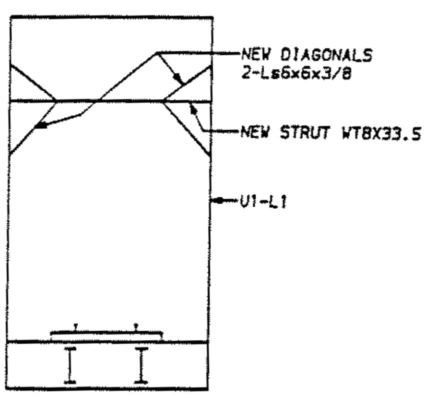
KEY PLAN

NO SCALE



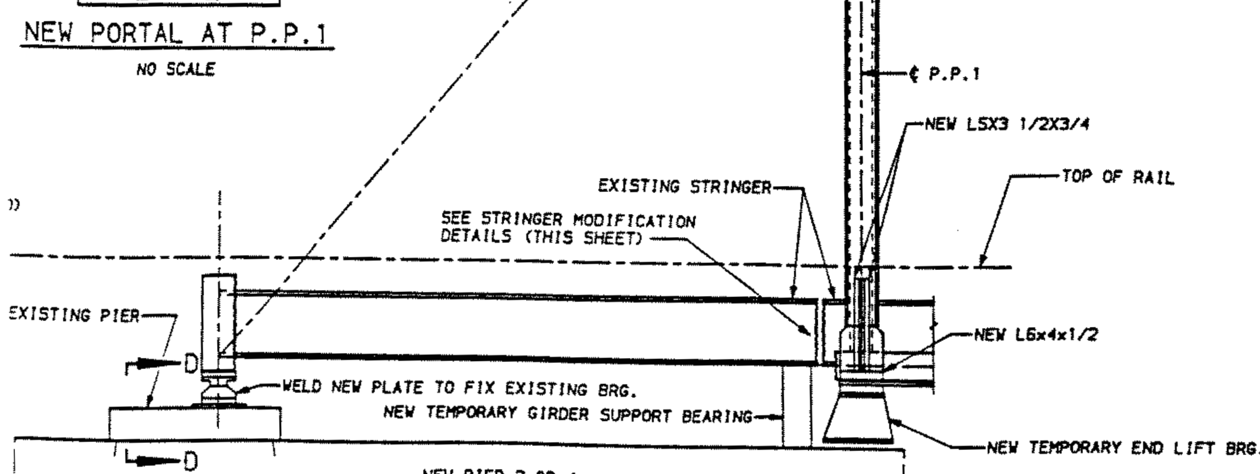
TRUSS PANEL 0-1, EXISTING

SCALE: 1/4"=1'-0"



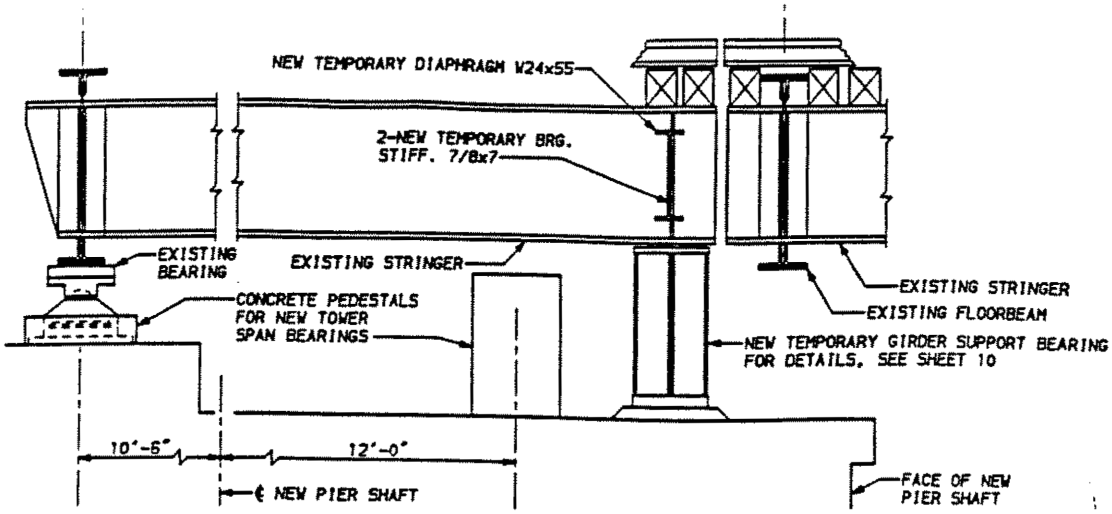
NEW PORTAL AT P.P.1

NO SCALE



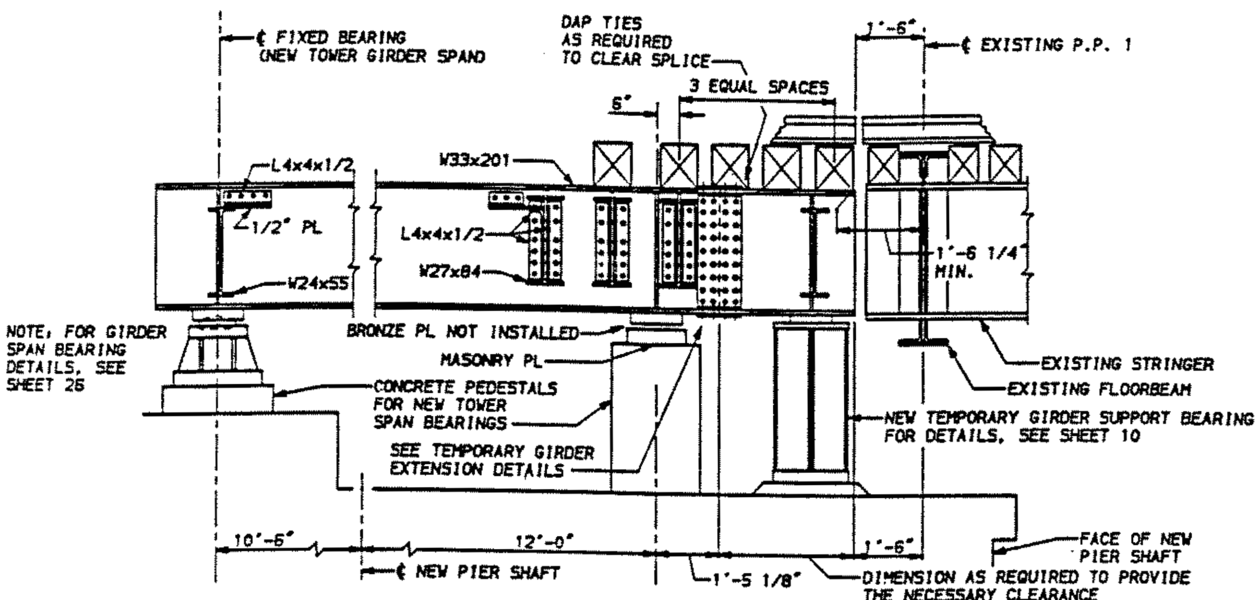
TRUSS PANEL 0-1, MODIFIED

SCALE: 1/4"=1'-0"



CONSTRUCT CONCRETE PEDESTALS FOR NEW TOWER SPAN

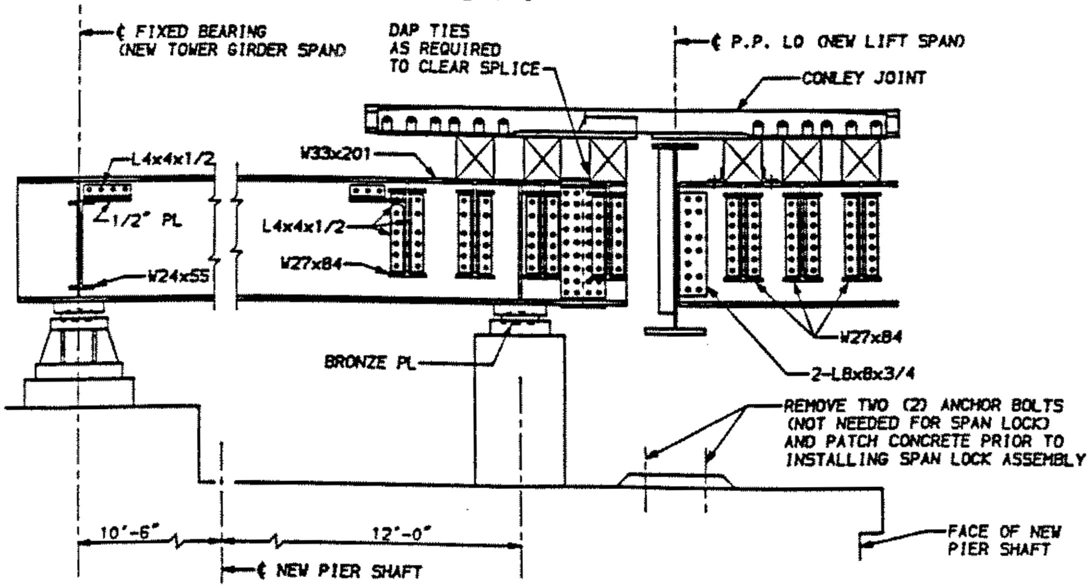
SCALE: 1/2"=1'-0"



NEW TOWER GIRDER SPAN WITH TEMPORARY EXTENSION

NOTE: FOR ADDITIONAL GIRDER DETAILS, SEE TOWER GIRDER SPAN DETAILS SHEET 26.

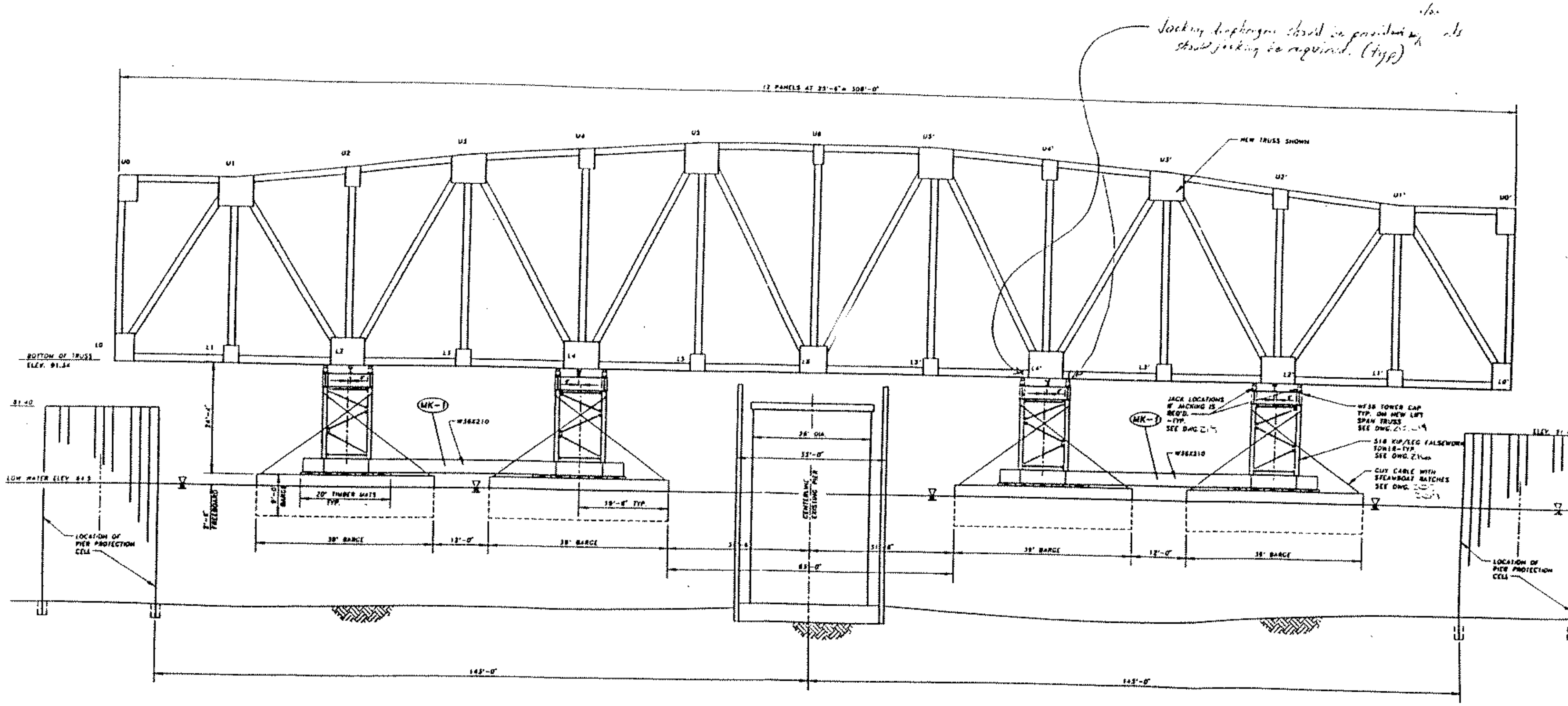
SCALE: 1/2"=1'-0"



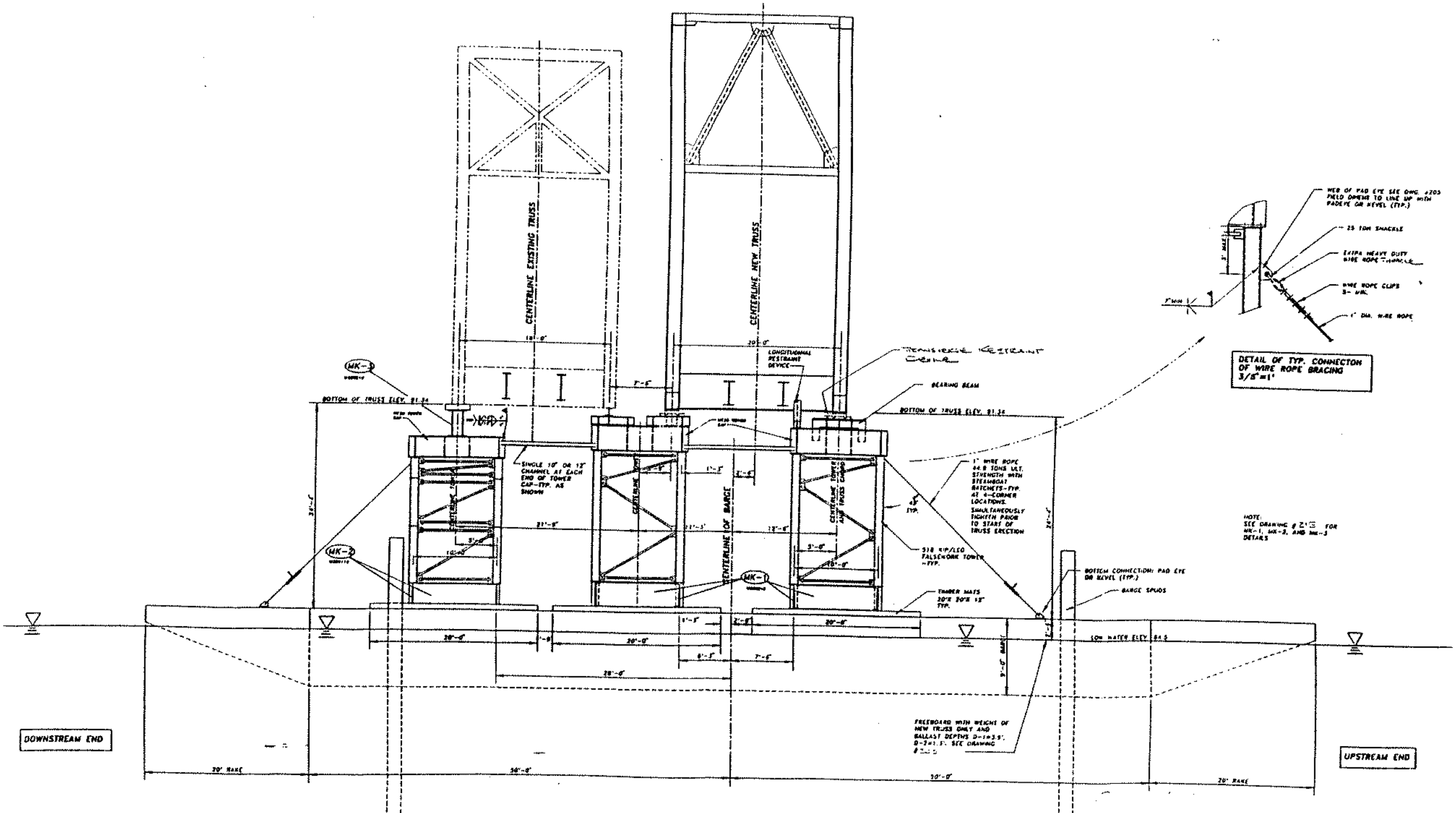
CONSTRUCTION SEQUENCE STAGE IV

NOTE: FOR ADDITIONAL GIRDER DETAILS, SEE TOWER GIRDER SPAN DETAILS SHEET 26.

SCALE: 1/2"=1'-0"



ELEVATION VIEW-LOOKING NORTH
NEW LIFT SPAN ERECTION
DWG. TYP. FOR SOUTH AND NORTH TRUSS



GENERAL SECTION VIEW LOOKING WEST