Tab E.Current Description and Conditions Assessment

From south to north, Memorial Bridge consists of the Portsmouth concrete approach structure, the main tower, lift and tower spans and the Maine beam and girder approach structure. On the main portion of the bridge, the flanking or tower trusses are variable depth Warren trusses of ten panels. The lift span is a parallel chord, constant depth, Warren Truss of ten panels. The towers supporting the lifting sheaves are in turn supported by the two central piers with truss work connected to the top chords of the flanking or tower spans. All of the machinery needed to lift the span is located at the middle of the lift span. The counterweights balance the dead weight of the lift span and up-haul and down-haul cables, wound on drums that are powered by electric motors, provide the small force required to lift or lower the lift span. Within this Tab, the three major components of Memorial Bridge (the Portsmouth Approach, the three central spans, and the Kittery Approach) are discussed in separate sections.

1. Memorial Bridge

a. Description

<u>Superstructure</u>

FLANKING OR TOWER SPANS

The bridge's flanking/tower spans are identical, 297'–1" long (from bearing to bearing) steel spans.

The flanking spans are variable depth Warren trusses with ten panels lengths of 29'–8½". The depths of the trusses vary from 35' (L1-U1), 44' (L3-U3) and 47' (L5-U5) as labeled below.

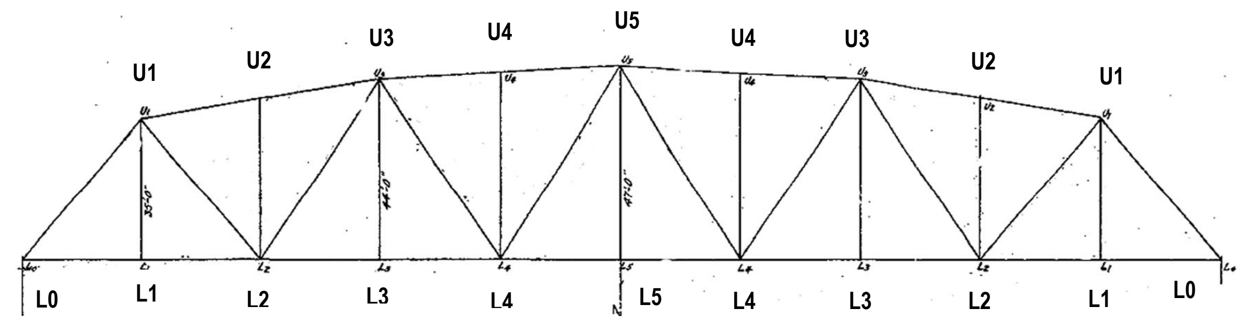

Figure E-1: Tower span with panel point designations

The top chords and inclined end posts are built up riveted box sections consisting of twin web plates connected to top and bottom cover plates with four angles. The size of angles, plates and web members vary across the length of the span as follows:

Member	Web plates	Cover Plates/lattice*	Angles	Side Plates
L0-U1	2 –26" x 11/16"	1 – 28" x 9/16" (top) 5" x 9/16" (bot.)	2 – 4" x 4" x 5/8" 2 – 6" x 6" x 5/8"	
U1 – U3	2 –26" x 9/16"	1 – 26" x 9/16" (top) 5" x 9/16" (bot.)	2 – 4" x 4" x 3/8" 2 – 6" x 6" x 5/8"	
U3 – U5	2 –26" x 9/16"	1 – 28" x 9/16"(top) 5" x 9/16" (bot.)	2 – 4" x 4" x 5/8" 2 – 6" x 6" x 5/8"	2 – 15 1/2" x 9/16"

*Lattice bars are in diagonal ("x") pattern

The lower chord consists of two or four web plates, four angles and lattice members of variable dimensions across the length of the span as follows:

Member ID.	Web plates	Lattice *	Angles
L0-L2	2 –28" x 9/16"	2 1/2" x 3/8"	4 – 4" x 4" x 1/2"
L2 – L4	2 –28" x 5/8" 2 – 28" x 9/16"	2 1/2" x 3/8"	4 – 4" x 4" x 1/2"
L4 – L5	2 –28" x 5/8" 2 –28" x 11/16"	2 1/2" x 3/8"	4 – 4" x 4" x 1/3"

*lattice bars are single (zigzag) pattern and are replaced with plates at all panel points.

The verticals at panel points L1- L5 are built up with four angles and lattice members of variable dimensions across the length of the span as follows:

Member ID.	Angle irons	Lattice *
L1- U1	4 – 6" x 4" x 3/8"	2 1/2" x 3/8"
L2 – U2	4 – 7" x 3 1/2" x 7/16"	2 1/2" x 3/8"
L3 – U3	4 – 6" x 4" x 3/8"	2 1/2" x 3/8"
L4 – U4	4 – 7" x 3 1/2" x 7/16"	2 1/2" x 3/8"
L5 – U5	4 – 6" x 4" x 3/8"	2 1/2" x 3/8"

* lattice bars are single (zigzag) pattern

The diagonals are built up with twin web plates, four angles and lattice members of variable dimensions across the length of the span as follows:

Member ID.	Web plates	Lattice *	Angles
U1-L2	2 –24" x 9/16"	2 1/2" x 7/16"	4 – 4" x 4" x 1/2"
L2 – U3	2 – 20" x 9/16"	2 1/2" x 7/16"	4 – 3 1/2" x 3 1/2" x 3/8"
U3 – L4	2 – 16" x 3/8"	2 1/2" x 7/16"	4 – 3/1/2" x 3 1/2" x 3/8"
L4 – U5	2 – 16" x 3/8"	2 1/2" x 7/16"	4 – 3/1/2" x 3 1/2" x 3/8"

* lattice bars are single (zigzag) pattern

All of the main truss members are connected to one another through pairs of gusset plates at each upper and lower chord panel point. The gusset plates vary in lateral dimension as necessary to provide the required number of rivets. The thickness of the gusset plates is a consistent 7⁄16".

The portal framing at each end of the trusses is built up of four horizontal angle 3½" x 3½" x 3⁄8" with double lattice work between the top panel points U2 and the same size members between the end diagonals at the lower level. “X” bracing at 45 degree angles consisting of two $3\frac{1}{2}$" x $3\frac{1}{2}$" x $\frac{3}{8}$" angles with double latticing is placed between the top and bottom horizontal members.

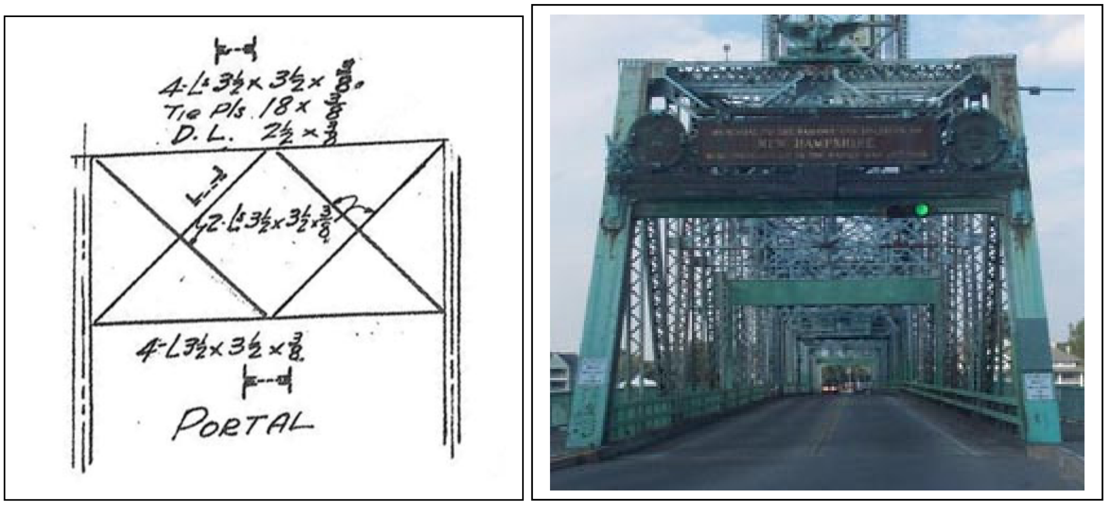

Figure E-2: Portal Bracing Plan and Photograph

Lateral bracing in the plane of the top chords between the trusses at all intermediate panel points consist of twin steel angles perpendicular to the planes of the trusses and built up members of four steel angles and lattice members as “X” bracing. The members perpendicular to the chords at the panel points consist of twin 6" x $3\frac{1}{2}$" x $\frac{5}{16}$" angles and the diagonal braces of four 4" x 3" x $\frac{5}{16}$" angles connected with double latticing of $2\frac{1}{2}$" x $\frac{3}{8}$" bars with $\frac{3}{8}$" plates placed at the ends and middle of the span.

Sway bracing in the vertical plane at U2 to U5 consists of single angle members varying in size from $3\frac{1}{2}$" x $3\frac{1}{2}$" x $\frac{5}{16}$" to 6" x $3\frac{1}{2}$" x $\frac{5}{16}$" as shown in the plan below:

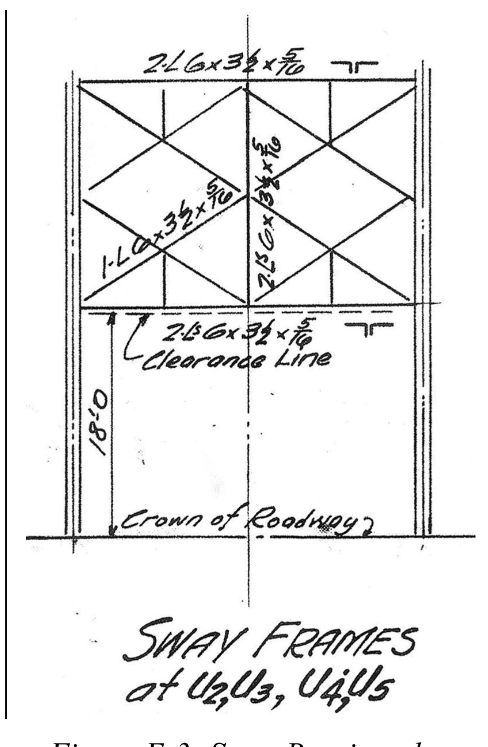

Figure E-3: Sway Bracing plan

Principal members are 6" x $3\frac{1}{2}$" x $\frac{5}{16}$". All the rest are $3\frac{1}{2}$" x $3\frac{1}{2}$" x $\frac{5}{16}$". The sway frames vary in height.

The floor beams are riveted to the vertical and bottom chord members at each lower chord panel point. The beams are built up of two 6" x 6" x $\frac{3}{4}$" angle top and bottom riveted to a 48" x $\frac{3}{8}$" web plate with $3\frac{1}{2}$" x $2\frac{1}{2}$" x $\frac{3}{8}$" web stiffeners 6' on centers. The sidewalk outriggers are also riveted to the vertical and bottom chord members at each lower chord panel point. The cantilevered brackets are of variable depth, with twin 3" x 3" x $\frac{3}{8}$" angles top and bottom, bracketing a $\frac{3}{8}$" plate with a pair of $3\frac{1}{2}$" x $3\frac{1}{2}$" x $\frac{3}{8}$" web stiffeners.

At each beam location an additional diaphragm is placed between the web plates of the lower chord to provide additional stiffness to the cantilever bracket. The diaphragm consists of four 6" x 4" x $\frac{3}{8}$" angles and a 19½" x $\frac{3}{8}$" x 1' – 4" plate.

The deck structure consists of 24 "I" stringers varying in size from 79.9 #/foot to 105 #/foot. The second stringer from the westerly side of the bridge is heavier at 105# with the remaining stringers being the 79.9# beams. (To accommodate the anticipated a trolley line on the bridge.) The stringers are riveted to the webs of the crossbeams and set on a seat built of a vertical angle and a horizontal seat angle. Under deck cross bracing consists of twin 4 x 3 x $\frac{3}{8}$ angles riveted together with one set with upstanding legs and the crossing member with downstanding legs.

The beams are spaced 6'–0" on center with the outer beams 2'–10" from the center lines of the trusses. Crossing the stringers are I - 7 x 17.5 beams that are crowned and at a spacing of 2'–5". The deck consisted of 5" reinforced concrete slab with $\frac{3}{8}$" deformed reinforcing bars top and bottom and a 2" thick bitulithic covering.

The sidewalk is elevated off the cantilever brackets by a 15" I 42.9# beam and a 15" channel at 33.9# per foot. Three-inch thick planks, smooth one side, are spiked to 4" x 6" timber bolted to the beam and channel. To add stiffness to the sidewalk diagonal bracing consisting of 3" x $2\frac{1}{2}$" x $\frac{3}{8}$" angles was placed in a zigzag pattern along the length of the fixed spans.

LIFT SPAN

The lift span is a parallel chord Warren Truss with ten panel lengths of 29'–$8\frac{1}{2}$". The depth of the trusses is 35'. The total length of the truss from bearing to bearing is 297'–1".

The top chords and inclined end posts are built up riveted box sections consisting of twin web plates connected to top and bottom cover plates with four steel angles. The size of angles, plates and web members vary across the length of the span as follows:

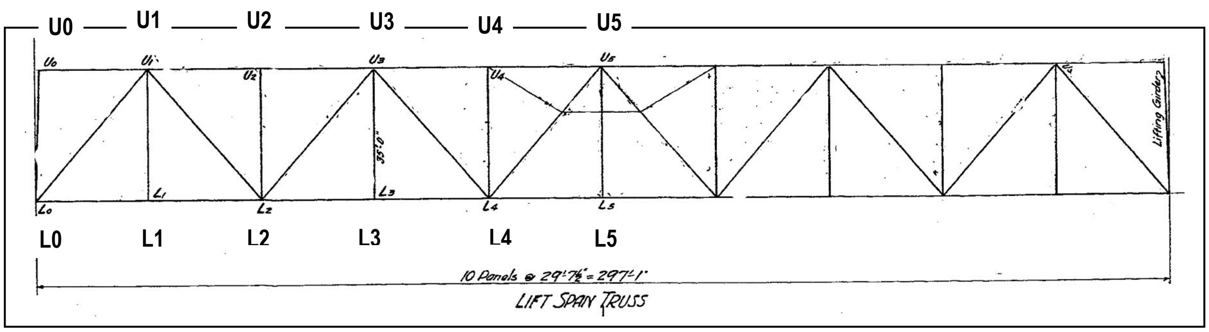

Figure E-4: Lift span with panel point designations (NHDOT)

Member	Web plates	Cover Plates/lattice*	Angles
L0-U0	16" x 5/8"		4 – 8" x 6" x 5/8"
U0-U1	2 – 26" x 3/8"		4 – 6" x 4" x 3/4"
L0-U1	2 – 26" x 5/8"	1 – 28" x 9/16" (top) 5" x 9/16" (bot.)	2 – 4" x 4" x 1/2" 2 – 6" x 6" x 1/2"
U1 – U3	2 –26" x 5/8"	1 – 28" x 9/16" (top) 5" x 9/16" (bot.)	2 – 4" x 4" x 3/8" 2 – 6" x 6" x 5/8"
U3 – U5	2 – 26" x 9/16" 2 – 26" x 5/8"	1 – 28" x 9/16"(top) 5" x 9/16" (bot.)	2 – 4" x 4" x 5/8" 2 – 6" x 6" x 5/8"

*Lattice bars are in diagonal ("x") pattern

The lower chord consists of two or four web plates, four angles and lattice members, top and bottom, of variable dimensions across the length of the span as follows:

Member ID.	Web plates	Lattice *	Angles
L0-L2	2 –28" x 1/2"	2 1/2" x 3/8"	4 – 4" x 4" x 1/2"
L2 – L4	4 –28" x 1/2"	2 1/2" x 3/8"	4 – 4" x 4" x 1/2"
L4 – L5	4 –28" x 1/2" 2 –20" x 1/2"	2 1/2" x 3/8"	4 – 4" x 4" x 1/2"

*lattice bars are single (zigzag) pattern and are replaced with 15" x $^3\!/_8$" plates at all panel points.

The verticals at panel points L1-L5 are built up with four angles and lattices members of variable dimensions across the length of the span as follows:

Member ID.	Angles	Lattice *
L1- U1	4 – 6" x 3 1/2" x 3/8"	2 1/2" x 3/8"
L2 – U2	4 – 6" x 3 1/2" x 7/16"	2 1/2" x 3/8"
L3 – U3	4 – 6" x 3 1/2" x 3/8"	2 1/2" x 3/8"
L4 – U4	4 – 6" x 3 1/2" x 3/8"	2 1/2" x 3/8"
L5 – U5	4 – 6" x 3 1/2" x 1/2"	2 1/2" x 3/8"

* lattice bars are single (zig-zag) pattern

The diagonals are built up with twin web plates, four angles and lattice members of variable dimensions across the length of the span as follows:

Member ID.	Web plates	Lattice *	Angles
U1-L2	2 –24" x 11/16"	2 1/2" x 3/8"	4 – 4" x 4" x 9/16"
L2 – U3	2 – 20" x 5/8"	2 1/2" x 3/8"	4 – 4" x 4" x 5/8"
U3 – L4	2 – 16" x 1/2"	2 1/2" x 3/8"	4 – 3 1/2" x 3 1/2" x 7/16"
L4 – U5	2 – 16" x 3/8"	2 1/2" x 3/8"	4 – 3 1/2" x 3 1/2" x 3/8""

* lattice bars are single (zigzag) pattern

All of the main truss members are connected to one another through pairs of gusset plates at each upper and lower chord panel point. The gusset plates vary in lateral dimension as necessary to provide the required number of rivets. The thickness of the gusset plates is a consistent $^7\!/_{16}$".

Lateral bracing in the plane of the top chords between the trusses at all intermediate panel points consist of built up beams with four 6" x $3^1\!/_2$" x $^5\!/_{16}$" angles, two on top and two below, with double latticing $2\frac{1}{2}$" x $\frac{3}{8}$" throughout. 18' x 2'–$2\frac{1}{2}$" x $\frac{3}{8}$" plates are used at the ends with a 24" x 2'–$2\frac{1}{2}$" x $\frac{3}{8}$" plate at mid span.

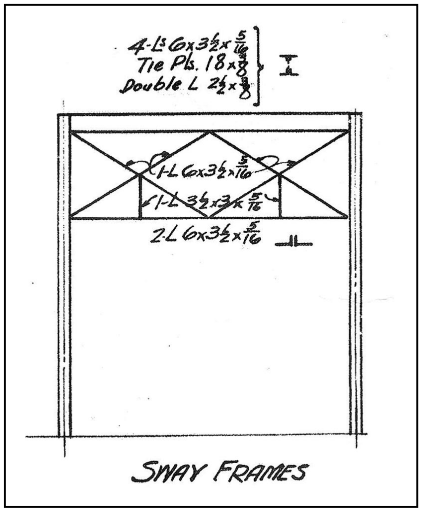

Figure E-5: Sway Bracing Plan (NHDOT)

Sway bracing in the vertical plane at U2 to U5 consists of single and double steel angles varying in size from $3\frac{1}{2}$" x $3\frac{1}{2}$" x $\frac{5}{16}$" to 6" x $3\frac{1}{2}$" x $\frac{5}{16}$" as shown in the plan above:

The floor system of the lift span is identical to that of the two fixed spans. (See above page E-4)

TOWERS

The 180' high towers are framed off of the two fixed spans connecting at panel points L0, U1 and U2. Their main function is to support the sheaves that are mounted at the top of the towers. The towers are designed to support the entire load of the counterweights as well as the lift span and its equipment. Since the sheaves are mounted directly above panel point L0 the main vertical loading is directly down the vertical members. All other tower framing is to keep this member in its vertical position under wind loadings. The twin vertical load carrying members are braced in a transverse and longitudinal direction by truss work.

The tower framing in the longitudinal direction for the top three panels consists of four angles separated by lattice work with the size angle 4" x 3" x $\frac{5}{16}$" to 5" x 3" x $\frac{5}{16}$" to 6" x 3" x $\frac{5}{16}$". The next lower panel consists of the four angles with lattice work using 6" x $3\frac{1}{2}$" x $\frac{5}{16}$". The lower two panels are built up with twin 12" x 20.5" channels with lattice work. The curved back legs of the towers are built up with twin 12" x 20.5" channel latticed together. The face of the tower facing the lift span, shown below, is braced with sections of four angles with lattice bracing with the size of the angles for the cross bracing and horizontal members increasing in size moving from top to bottom of the tower. Special framing for the sheave supports is shown at the top of the tower.

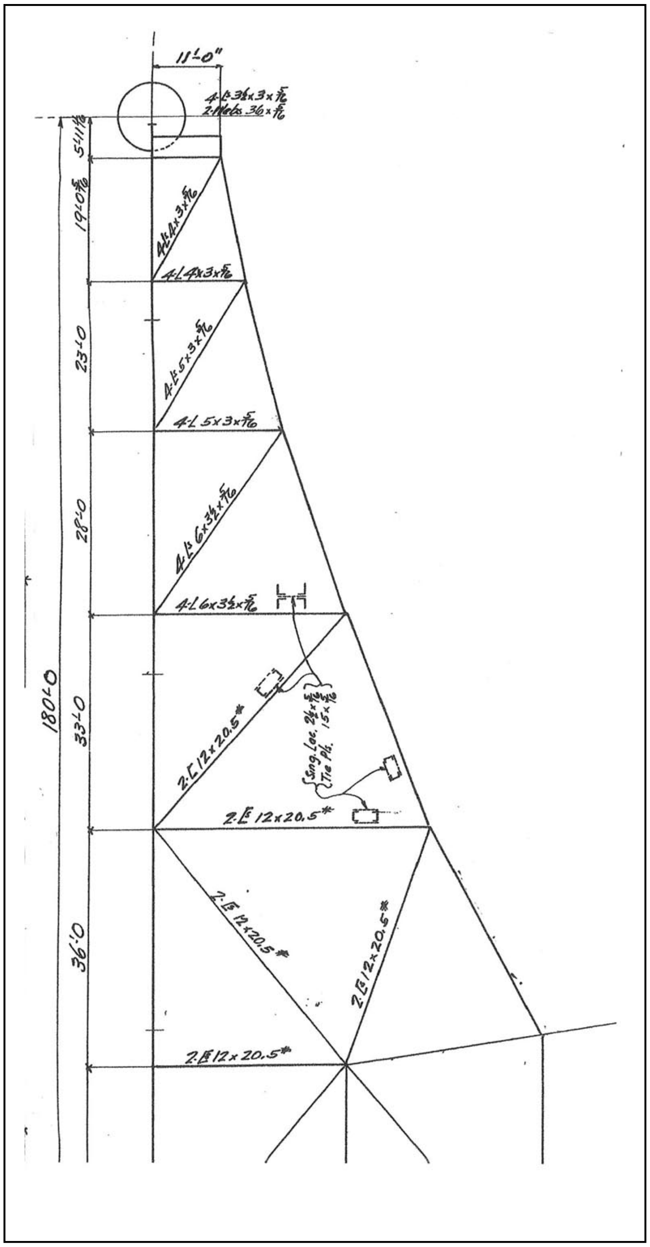

Figure E-6: Tower longitudinal bracing

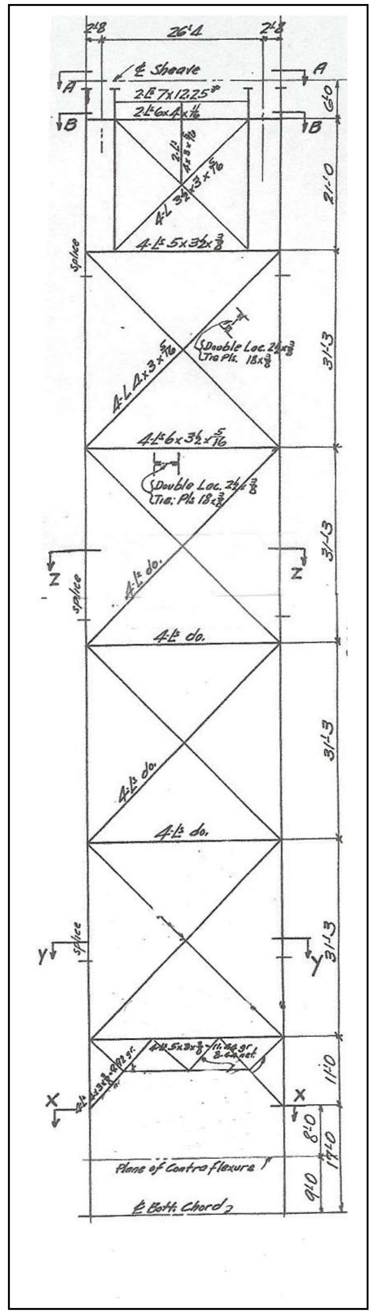

Figure E-7: Tower Front Bracing

The main tower leg is built up with eight 4" x 3⁄4" angles, one plate 18" x 3⁄4" and four plates 26" x 9⁄16" as shown in the sketch below. The angle on the lower left is to guide the lift span as it moves along the tower and the angle on the upper right is the guide for the counterweights. The overall dimensions of the built up tower leg is 2'–2½" by 1'–8½".

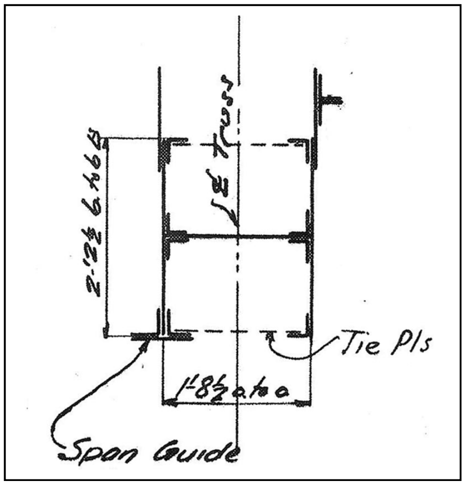

Figure E-8: Main Post

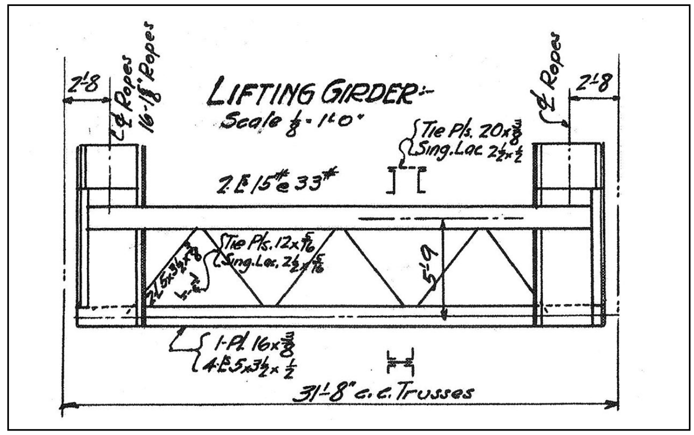

Figure E-9: Lifting Girder

The last main structural element is the lifting girder. This is the structural element, which ties the counter weight cables to the lift span and is located above panel point UO. As such it is a key link in the connections between the sixteen cables at each end of the lift span at its upper end and the lift truss at panel point U0 on its lower end.

COUNTERWEIGHTS AND MAIN CABLES

A vertical lift bridge operates by having counter weights closely balancing the dead weight of the lift span. The lifting equipment therefore needs only to overcome friction and some unknown variable loads such as snow, dirt, etc. The design of the bridge required that, at each end of each lifting girder, a load of 523,000# be transferred. They are braced steel boxes 24' long, 7'–6" wide and 28'–6" deep filled with concrete. The counterweights are connected to the sixteen 1⅝" diameter cables through an "equalizer" connector shown below:

Plate E-1: Sheaves and lifting cables – sixteen per end of truss

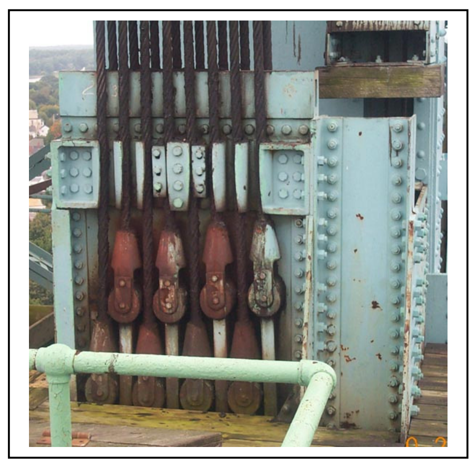

Plate E-2: Lifting Cables and Girder connection

The multiple pins ensured that each cable is subjected to the same amount of tension. The equalizer is connected to the counterweights with two 6" diameter pins.

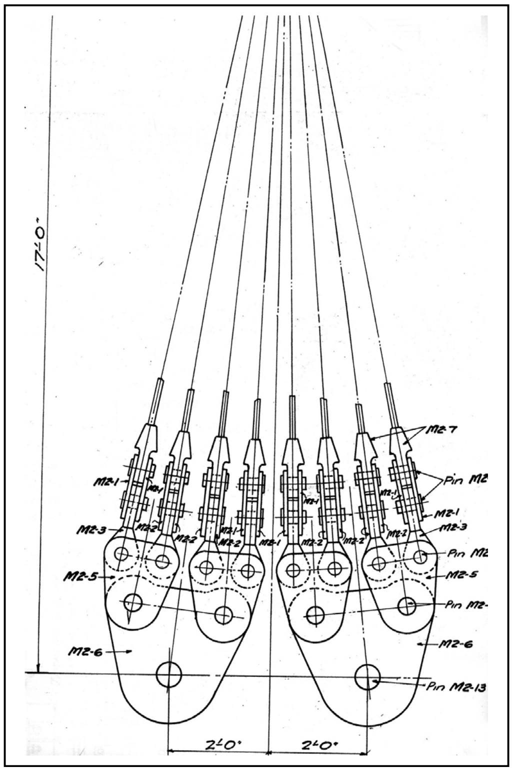

Figure E-10: Equalizers

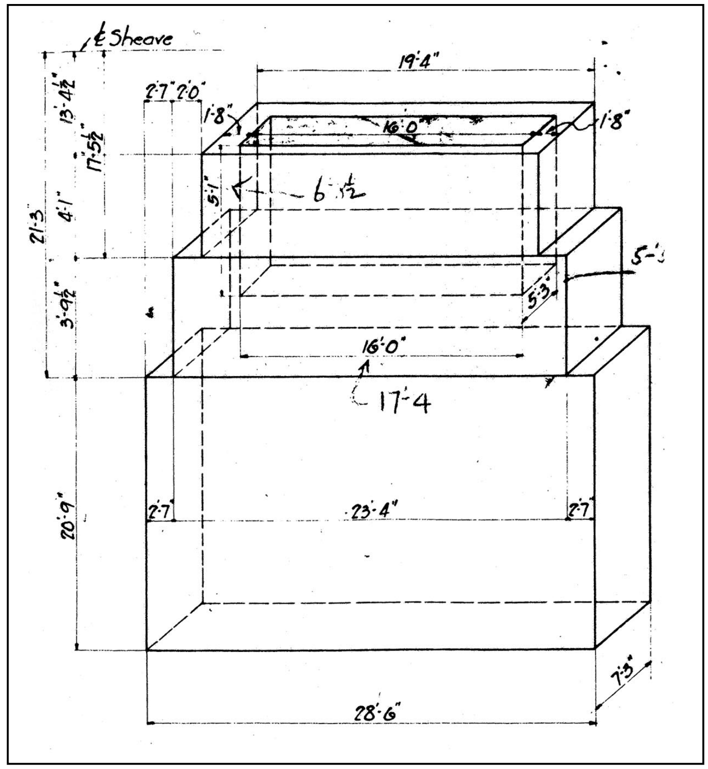

Figure E-11: Counterweight Steel Box and dimensions

The counterweight box was fabricated from steel plates and channel sections. The total weight of the box and 155 #/cubic foot concrete was determined to be approximately 880,000# with provision for adding as necessary 400 concrete blocks weighing 150# each into a pocket formed into the counterweight. The counterweight cables looped over a pair of 10'–6" diameter steel sheaves running on a 17" shaft that sits in 15½" diameter roller bearings. The outer surface of the sheaves is grooved to receive the sixteen cables. The cables then drop down to the lifting girder and its connections with the lift span.

OPERATING EQUIPMENT

The operating equipment is housed in a 25'–8½" x 29' control house, between the inside face of walls, set near the top of the lift span. Its roof is set just below the bottom of the top chord of the lift span truss and the main control room centered on the centerline of the span transversely and longitudinally. The house is set between the trusses and set on the framing shown of the lift span between panel points L4 – U4 and L5 – U 5.

The movement of the lift span either in an upwards (opening) direction or downward (closing) direction is initiated by the action of two 100 Horsepower electric motors. At the direction of the operator, these motors, which are tied into a gear system shown below, transmit power to the winding drums.

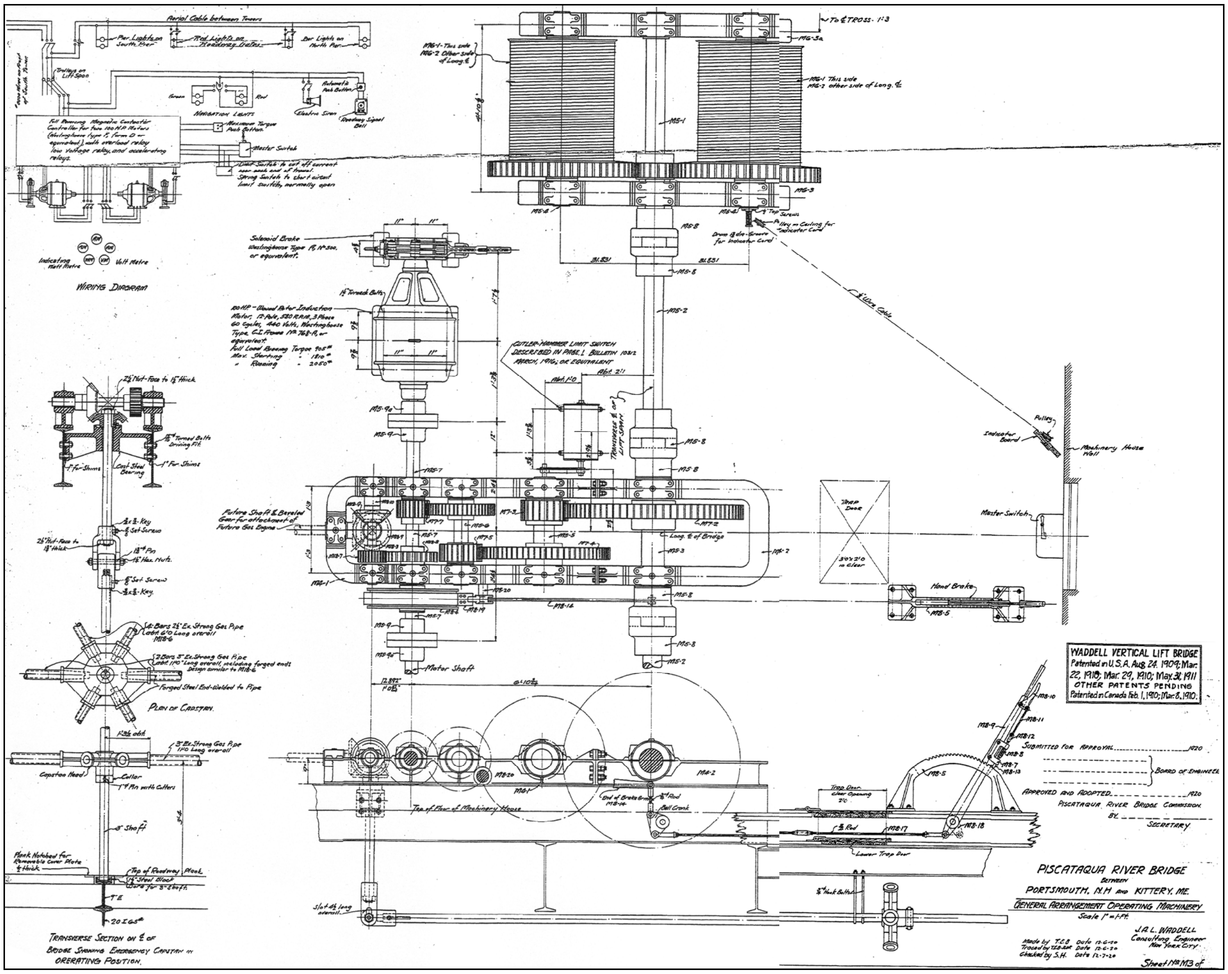

Figure E-12: Hoisting or lowering equipment (See full drawing at O-15)

The main shaft from the gear train is at the center of the lift span with the winding drums an equal distance from the shaft. Twin 100 HP electric motors connect to a common shaft M5-7. From this shaft moving from left to right, the torque generated by the motors is transmitted through a gear train and three shafts M5-6, M5-5 and M5-3; the latter is the shaft that transmits torque to the winding drums. As shown below (Figure E-13), the diameters of the toothed gears get larger as the torque is transferred through the system. This system results in the winding drum shaft moving at much lower rotational speed than the shaft connected to the electric motors that rotate at 580 revolutions per minute.

At the left end of the gear train, the original design made provision to raise the span by manual means. In the event of a power outage, a shaft (see the lowest horizontal member in Figure E-12) dropped down to the deck below and engaged an insert in the deck. Wooden poles could be set into a capstan (see lower left portion of Figure E-12) and the shaft attached to the capstan rotated, engaging a gear, (M18-9 attached to a shaft M18-3), which in turn rotated a gear M18-7, which in turn rotated a larger diameter gear (M18-8) which rotated the same shaft (M5-7) that the electric motors was attached to (see Figure E-13).

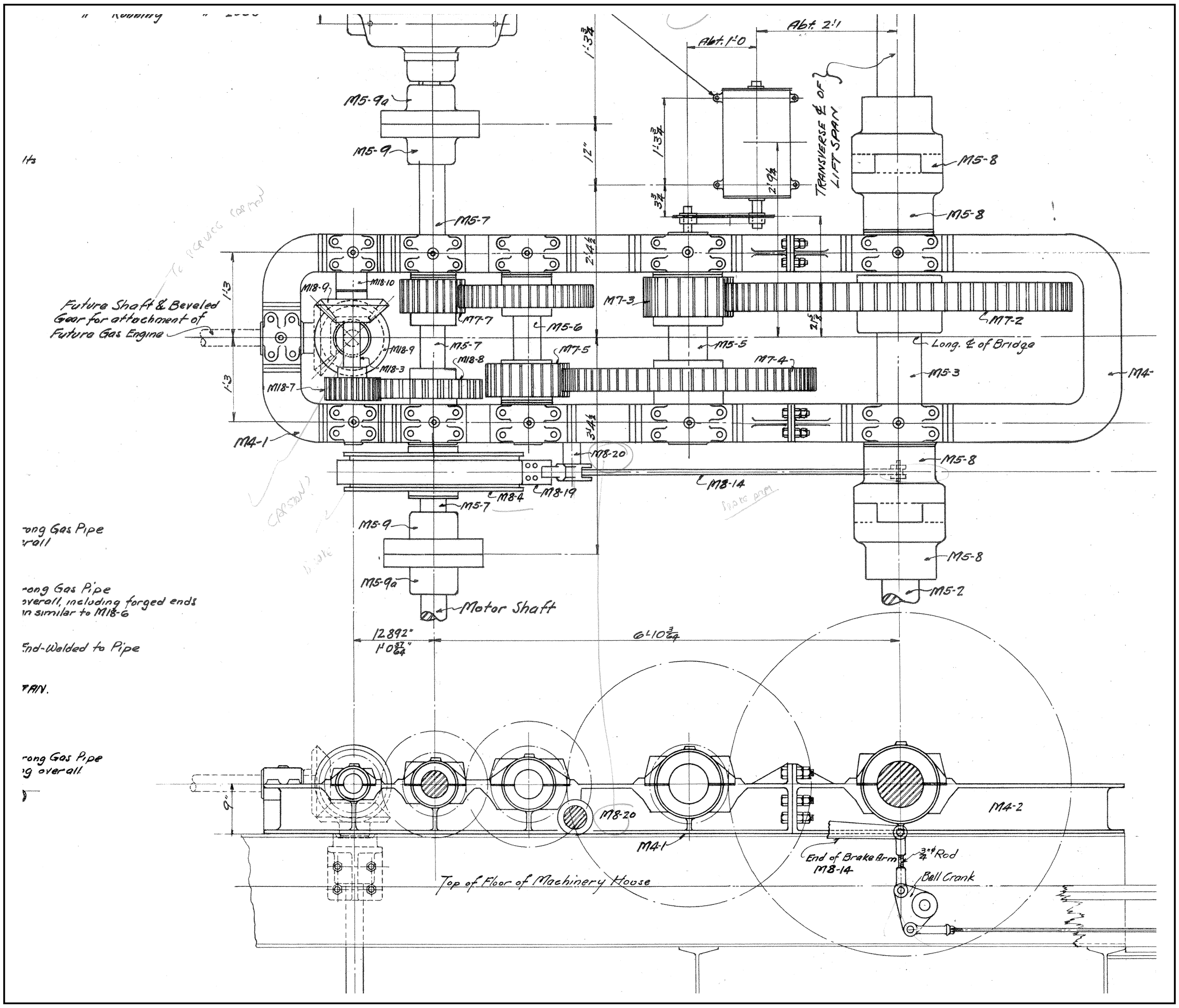

Figure E-13: Gear Train Detail (See full drawing at O-15)

The original design also made provision for a gasoline motor to be utilized in the event of a power outage. It is connected at the far left end of the power train and shows a beveled gear, similar to the manually operated gear discussed above. This beveled gear turns the same shaft M18-3 which through gears M18-7 and the larger M18-8 rotates the same shaft, M5-7 that the electric motors are attached to.

Torque is transmitted through the gear train to the winding drums through twin shafts M5-2, on through couplings M5-8, to twin shafts M5-1 and to a gear attached to them. As the main shafts turn, the small gears engage the large main toothed gears on the winding drums. Since both winding drums are turned by the same toothed gear, the drums will turn in the same directions. If the shaft is turning in a clockwise direction both drums will turn in a counterclockwise direction and vice versa when the shaft is turning in counterclockwise direction.

If the motors are operated so that the drums turn in a clockwise direction, they cause the downhaul cables (two 1" diameter ropes) to tighten and drop, with the assistance of the span weight, the bridge at all four corners equally. If the drums are turned in a counterclockwise direction the uphaul cables (also two 1" diameter ropes) tighten causing the bridge to rise with the assistance of the counterweights to its open position. The paired uphaul and downhaul ropes rest in grooves fabricated in the winding drums. On the left drums, looking westerly, the uphaul ropes are wrapped so that they leave the bottom of the drums and the downhaul ropes leave the top of the drum. On the right drums the uphaul ropes are wrapped so that they leave the top of the drums and the downhaul ropes leave the bottom of the drums. As one pair of ropes is tightened, the other is loosened and the tightened pair of ropes wrap around the winding drum. In other words when the span is in its open position, most of the downhaul cables are spread out along the span and down to the base of the tower and the uphaul is wound on the drums. As the downhaul ropes are tightened they wind onto the drums at the same time the uphaul cables are winding off the drums. The pitch diameter of the drums is 3' with 18½ or 19½ turns of spiral grooves for each pair of the uphaul and downhaul cables to handle the length of cable to go from the drum out along the lift span on rollers and then around a sheave and thence up to the top of the towers for the uphaul cables and from the drum out along the lift span or rollers to a sheave and thence down to the base of the tower for the downhaul cables. Each of the uphaul and downhaul cables would have to be approximately 325' long. That is, long enough in the case of the downhaul cables to go from the base of the tower up to the top of the lift span when in an open position and thence back to the winding drum or in a closed position from the winding drum along the span and up to the top of the tower.

The operator has several indicators telling him when the bridge is high enough or when it is approaching a closed position. To slow the rise or fall of the lift span, the operator can also use a manual friction brake (shown in the lower right hand corner of Figure E-12). This braking action was applied to a large brake drum attached to shaft M5-7 In addition, friction brakes were also attached directly to the motor shafts.

FOUNDATIONS

The depth of the Piscataqua River varies across the width of the river with a maximum depth of approximately 80'. The river bottom at the two main, or tower piers, is rock with a thin layer of soil at the northerly and southerly piers. The two piers are similar in size. The two abutments are much smaller in depth even though similar in cross section.

More specifically, the north pier is 91.11' deep, going from an elevation of 25' (12.5' above the original river bottom) at its base to elevation 116.11' at its top. The pier is of concrete from its base to elevation 90.32'. From this elevation up to elevation 102.33', the pier is faced stone to prevent erosion between the high and low tide area. A copingstone is located at the top elevation of 116.11'. The pier tapers up to elevation 80.5' and then has vertical sides from there up to 90.3'. From that elevation to the top, the pier is also tapered. The dimensions of the pier at its top are 12' x 58'–3".

The south pier is 98.91' deep, reaching from an elevation of 17.2' (12.5' above the original river bottom) at its base to elevation 116.11' at its top. Like the north pier, it is of concrete construction up to elevation 90.32', then stone to elevation 102.33'. A copingstone is located at the top elevation of 116.11'. The pier tapers up to elevation 80.5' and then has vertical sides from there up to 90.3'. From that elevation to the top, the pier is also tapered. The dimensions of the pier at its top are 12' x 58'–3".

In cross section, the piers have a pointed shape on both upstream and downstream sides to act as icebreakers.

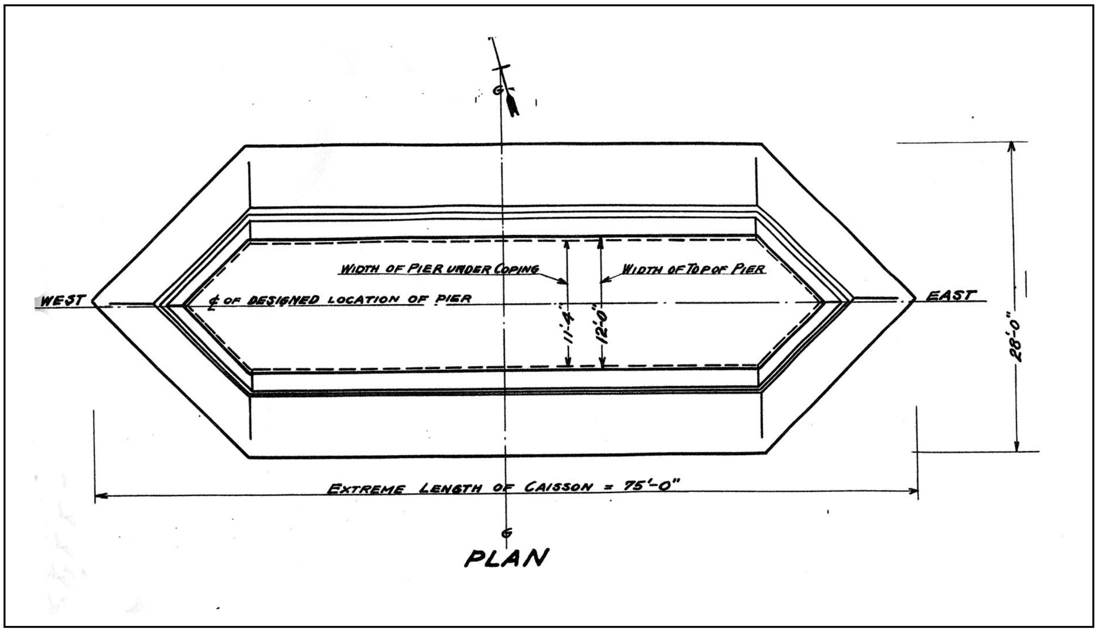

Figure E-14: Caisson and pier plan

DECORATIVE/MEMORIAL ELEMENTS

Embellishing the south portal of the bridge are bronze decorative elements added to the bridge in 1924 (see MB 11 page L-15). These decorations included a large rectangular plaque centered over the entrance to the bridge ("Memorial to the Sailors and Soldiers of New Hampshire who participated in the World War 1917-1919"). The United States seal (to the left) and the seal of the State of New Hampshire (to the right) flank the plaque. Foliage surrounds the lower portion of each seal. An anchor is located to the left side of the U.S. seal and a cannon is located to the right of the New Hampshire seal. Above the plaque is an eagle with outstretched wings. Its talons hold an olive branch and a bundle of arrows. The work was done by the Gorham Manufacturing Company of Providence, Rhode Island. Bronze plaques are also located on the end posts on the Maine and New Hampshire side of the bridge. These tablets were made by William H. Highton and Son of Nashua. The inscription gives the date of the bridge and lists members of the Building Commission, the Board of Engineers and members of the Preliminary Committee. For more information about the decorative plaques see Tab L

b. Alterations

Memorial Bridge strongly retains all elements of its integrity. The bridge's major features, foundations, nearly all of its structural steel in the trusses and towers, and its counterweights, are original to the bridge. Most changes to the bridge have related to the replacement of mechanical parts or other types of elements that by their nature frequently wear out and typically need to be replaced after years of heavy use.

Alterations to the bridge have included: replacement of the sheaves (1933 and 1940-41); replacement of the wood planking on the lift span (with steel decking) (1947); replacement of some of the original pipe hand railings (1947); replacement of the decks on the flanking spans and Kittery Approach (1960); replacement of the counterweight cables (1962); expansion of the machinery house and major overhaul of operating and mechanical systems (1977); replacement of some of the concrete bent columns supporting the Kittery Approach with sonotube-formed columns (1984); and construction of new barrier gates (1999). The significant repairs to the bridge will be treated chronologically in greater detail below.

1924

In 1924, bronze sculpture, decorative elements and plaques were added to the bridge. These included a large rectangular plaque and various decorative elements centered over the New Hampshire entrance to the bridge. At around the same time bronze plaques (one on the New Hampshire end and one on the Maine end) were added to the bridge which listed the names of the various committees, commissions and boards association with the construction of the bridge.

1929-30

The entire bridge was painted with two coats between July 22, 1929 and July 1, 1930 by James S. Heyson of Waverly, Massachusetts for a contract price of $11,000.

1932

In late July 1932, "The Boston Bridge Works jacked the upstream Kittery sheave back into position on shaft and placed eight new dowels. Results were not satisfactory. (Movement of sheaves on this tower has been noted since December 1928.)" (Richardson 1940). This apparently was a continuation of the problem that delayed the opening of the bridge in 1922-23. The displacement of the sheaves with respect to the shaft was approximately $^{21}/_{64}$" but had increased significantly between June and August of 1932. The dowels were to keep the movement from occurring in the future.

1933

In October 1933, the bridge was closed to permit the installation of two new sheaves at the Kittery Tower. The work was completed by Boston Bridge Works. The work supervised by Waddell & Hardesty, the original engineers on the bridge (Richardson 1940).

1934-36

On December 15, 1934, work began on, "renewing electrical equipment at the bridge and installing sleet-eater for the trolley wires." This work continued through 1935, working periodically, until early in 1936. The work consisted of:

> Electrical sleet–eating device for trolley wires.
> Elimination of trolley wires at Kittery end of draw.
> Grounding of entire bridge.
> New operating controls.
> New switchboards
> New signal system with traffic lights & warning gong.
> Control of signal system & gate lights by use of electric eyes. (Two at each end of draw.) (The navigation lights were also controlled by these eyes, which were turned on with the gate lights.) (Richardson 1940)

1936

Two gears in the main gear train were replaced by New Hampshire Highway Department personnel on June 1-3, 1936 (Richardson 1940).

1940-41

The sheaves on the Portsmouth Tower were replaced between December 1940 and December 1941 under the supervision of Waddell & Hardesty. The Phoenix Bridge Company of Phoenixville, Pennsylvania performed the work for $33,690.09. In addition, they replaced the downhaul and uphaul ropes, approximate lengths 325' (Waddell & Hardesty 1942). Measurements had been made between 1938 and October 1940 of the displacement of the sheave with respect to the shaft. During this time it increased to as much as $^{42}/_{64}$" thus prompting the replacement of the sheaves and shafts. The new sheaves were fabricated by the Earle Gear and Machine Company of Philadelphia. The sheaves were doweled to the new shafts with six 3" diameter pins. New bearings were also installed.

It should be noted that these changes to Memorial Bridge and all changes previous to them fall within the National Register's fifty year guideline and although not part of the bridge's original design have likely achieved historic significance.

1944

On February 17, 1944 a drifting submarine from the Portsmouth Navy Yard struck the Kittery Span at a point about 60' from the draw end of the span. With the force of the collision, the submarine's conning tower was bent back and the sub slipped under the span and up-river. Two minutes later, a Navy tug, which was attempting to rescue the sub, also struck and was stuck under the bridge (apparently at the drawspan). Immediately thereafter, the bridge operators raised the lift and the tug was freed. According to the contemporary report, damage to the lift span was at the third panel from the north end of the lift span. Damage to the Kittery span was at the second panel of the east truss near the south end. Repairs, which were completed by May 2, 1944, included: a new end connection; repairs to the outstanding legs of the lower flange angles of the bottom chord; repairing or replacing the sidewalk panel and sidewalk railing(at the easterly truss of the Kittery span). According to the report, "No attempt was made to remove the permanent lateral deflection in the bottom chord and main diagonal members in the easterly truss of the northerly span or to any of the lower lateral members "although these members have been considerably distorted." (State Highway Department 1944)

1949

In 1949, the wooden planking on the lift span was replaced with open A7 copper bearing steel decking by USS I-Beams Lok Open floor. The deck with a thickness of 5" was to be open in the middle panels but on the end panels a 3½" I-Lok decking was to be used and armored with bituminous concrete with variable thickness so wearing surfaces met at the first panel point. At this time new sidewalks were construction and certain electrical fixtures replaced. It is likely that the hand railings were also replaced at this time.

1960

In 1960, the fender guards on the lift span piers were upgraded along with the Kittery Approach piers. The pier work consisted of concrete rehabilitation and pointing of the stonework.

At the same time, the decks on the flanking spans and Kittery Approach were replaced with 5¼" thick reinforced concrete with 2" asphalt wearing course. The Kittery Approach was on the existing 24" longitudinal "I" beams and 7" transverse "I" cross beams.

1962

The counterweight cables were replaced in 1962 after a life of almost forty years. The ropes were inspected frequently and greased on a regular basis such that when in 1945, "engineers of the American Steel and Wire Company (original installers) inspected the ropes voluntarily as a matter of interest. No frayed wires were found, core was found in good condition, as no flattening was evident at the sheaves where the load would cause it to show. Wires were shiny on being cleaned of lubricant" (letter June 19, 1961 Richardson to Morton). Fifteen years later, another inspection of the cables was made by American Bridge engineers. They reported on July 15, 1960, "In brief- again no frayed wires but serious flattening had begun to develop indicating core deterioration and collapse. Stated no cause for alarm but in view of the condition and its progressive features and as ropes had given more than twelve years service over that which is customarily expected of this type of service, they state that the end of their useful life was being reached. Recommended immediate replacement if wires started to break" (letter June 19, 1961 Richardson to Morton). H. H. Richardson determined "due to the length of time required to set up such a project and time to manufacture the required ropes, and due to the catastrophic results if any full failure should occur. I requested authority to set up replacement project" (letter June 19, 1961 Richardson to Morton). Hardesty & Hanover was retained to prepare plans and specifications for the replacement program and submit an estimate for approval of both states. The preliminary estimate was $126,000 but by May 26, 1960, the estimate increased to $128,000. Bids were received in 1961 but work was not started until 1962 when The Seaward Construction Company of Kittery, Maine executed the contract. The work also included modifications to the lifting girders on the lift spans. The contractor made use of the original tower supports to hold the counterweights in the up position while the old cables were disconnected from both the lift span and counterweights. This work was performed between March 6 and May 10, 1962.

1969

The uphaul and downhaul cables were replaced in 1969 for the second time. The operators of the bridge had been experiencing trouble with the cables for over three years with cables coming off their sheaves and switching positions on the sheaves.

1972

Electric controls on the two 100 HP motors were upgraded in 1972 to ensure that each motor was equally loaded in lifting the bridge. In the same year, Hardesty & Hanover did a complete inspection of the bridge and made recommendations for maintenance and rehabilitation.

1974

The need for an emergency method to raise or lower the bridge was considered once again in 1974. Waddell's original plans called for a back-up gasoline motor to be installed in the future. A hand operated capstan mechanism was installed that could be used in an emergency. In 1974, it was noted that the hand-operated equipment had been removed and that the gasoline motor had never been installed. Hardesty & Hanover designed the gasoline motor and methods to connect it to the gear train. They also suggested that if it was going to take too long for the motor to be installed they would develop plans to restore the manual capstan operating method.

1977

In 1975, plans for the "Reconstruction of Electrical and Mechanical Including Structural Supports" were approved by both states. The plans called for an expanded machinery house with a virtually complete replacement of all motors, gear drives, winding drum bearings, etc. and the addition of emergency power. The machinery house was expanded to the south by steel framing attached to the lift span truss lateral bracing at truss panel point 4 (panel point numbering increases from the south to the north starting at panel point 0 at the southerly end of the lift span.) This enlargement created an additional 6' x 7' space with an observation platform and storage space of 4' x 17'. All additional steel was bolted in place. A36 steel rolled shapes were used. The machinery house was changed from wood frame to a concrete deck and steel framed structure with 3" metal insulated panels. Special structural steel members were placed to support the new motor and drive mechanisms. Inside, the new lift machinery consisted of two new 100HP electric motors attached to a new gearbox, which engaged the drive shaft to the existing lifting drums. Span brakes were attached to the output shafts from the electric motors, and on the opposite side of the gearbox, emergency disc brakes were installed to control the speed of the gear train running to the lifting/lowering drums. A new propane powered L. P. G. engine was provided for emergency power. This engine tied into the main gear train with a reversing transmission that could raise or lower the span in an emergency. The operator's control panel was upgraded to provide the latest control technology. While the existing winding drums were retained, all other gearing, pinions, and bushings were replaced. The work was completed by Seaward Construction Company of Kittery for $524,500. Work started on April 5 and the bridge reopened on June 25, 1977.

1977

The city of Portsmouth rehabilitated the south approach and abutment in the summer of 1977. In late 1978 New Hampshire asked Maine to participate in a complete rehabilitation of the bridge based upon Hardesty & Hanover's 1972 inspection report. This proposed rehabilitation consisted of painting, truss member repair, and miscellaneous steel repair. Roadway bridge railing installation, new expansion joints at the bridge abutments, uphaul and downhaul rope replacement, replacement of sidewalk planking, etc. was also included. The estimated cost, including engineering and inspection was $880,000. No work was done, as Maine did not concur with the rehabilitation.

1981

Hardesty & Hanover inspected the bridge again in 1980 and found several items of critical concern. Their main concern was that the pulleys at the ends of the lift span over which the uphaul and downhaul ropes pass were not turning and that the cables were slipping over them causing particular wear. In the summer of 1981, both states approved the rehabilitation plans of Hardesty & Hanover for a rehabilitation of "tower and lift span structural steel including improvement work." The work is shown on a 38-sheet set of drawings dated June 1981 and consisted of the following:

Repair of Structural Steel

The repairing of all structural steel that had been bent from impacts and members weakened by rusting or rivet failure. Repairs consisted of:

Truss Web members

U1-L1 Span 2 (Lift)	East Truss
U5-L5 Span 3	West Truss
U1-L1 Span3	West Truss
U1-L1 Span 1	East Truss
U1-L1 Span 3	East Truss
U5-L5 Span 1	East Truss

These were all tension members where no diagonals connected to the lower chord of the truss. All members consisted of four angles with latticing. (Sheets 11-14)

Lower chord member replacement

L7-L9 Span 3 East Truss

The northerly 20'–$^3/_4$" end of this member was replaced with four steel angles, two 28" x $^9/_{16}$" plates with lattice bars. (Sheet 15)

Lower chord plates connecting with under deck lateral bracing

The repair was necessary at selected points on all three spans. It consisted of cutting away the existing plates and replacing with new plates bolted to the top angles of the lower truss chords. (Sheets 16, 17)

Upper chord repairs

This repair was necessary at selected points on all three spans. Generally, it consisted of replacing splice plates on the top surface of the chords and new plates to connect the top chord lateral bracing to the top chords. Some rivet replacement was also necessary using bolts. (Sheet 19)

Lift span floor beam repairs

Floor beams L2, L3 and L7 required the placement of $^3/_8$" plates to the webs of existing beams. These plates were bolted through the web and the stringers modified to connect to the thicker webs of the girders. Stringer connections were upgraded to include a seat angle and web stiffening angles extending from the seat to the lower flange of the girder. Rivet replacement on the lower twin angles was necessary in places on most girders. Bolts $^7/_8$" diameter were used for all rivet replacement. (Sheet 20)

Pavement repair and replacement

The concrete pavement on the tower spans was repaired and a two layer bituminous concrete pavement was laid down. The open metal grating on the lift span was repaired in places and replaced at the northwesterly end of the lift span between floor beams 1, 8, 9 and 10. New stringers were added in this location. (Sheet 22)

Expansion Joints

New expansion joints were installed at the southerly end of span 1 and at the northerly end of span 3 with fingerplates. An upgraded expansion plate was also added between the ends of the tower spans and the lift span consisting of a solid plate connected to the lift span riding on an "I" section cast into the ends of the tower spans. (Sheet 23)

Counterweight guide replacement and tower repair

Gusset plates were replaced on the northwesterly end of the lift span on the southerly face of the columns at three panel points. New counterweight guides and counterweight guide castings at the upper and lower end of the guide were installed. Seward Construction wrote to the New Hampshire District Construction Engineer, Jess Dennis, noting that they could find no one to cast the new guide castings and instead recommended repairing the existing castings. (Sheet 24)

Timber platform – top of tower

The existing wooden platform providing access to the sheaves and counterweight cables was replaced in its entirety. The existing railing was retained and new lighting was installed. (Sheet 26)

New expansion shoes on tower spans

Four new expansion shoes were placed: two at the southerly end of span 1 and two at the northerly end of span 3. (Sheet 27) The sheet also described the new operating ropes required to be installed. These ropes were to be 325' long with an open socket at one end.

Concrete pedestal repair

The concrete pedestals at the top of the lift span piers were chipped away back to the steel grillage upon which the bearings for the tower and lift spans sit. New concrete was placed and the entire top of the piers given an epoxy coating. (Sheet 28)

Concrete Repair Pier #1

Spalled concrete on the back wall of this pier was removed and new concrete added. The entire bridge seat was given an epoxy coating. (Sheet 29)

Fender System Repairs

New planking was added on the north face of pier #2 and the south face of pier #3. These were the surfaces facing the shipping channel. (Sheets 30, 31)

Bridge Rail

A new bridge rail (a railing located between the roadway and the structural elements of the bridge) was added to the center three spans. The bridge rail protected the bridge structure from vehicular accidents.

Miscellaneous Items

750 – 90# cast iron counterweight blocks were provided and holes drilled in the bottom of the steel counterweight boxes to attach the blocks.

New Ladders were to be provided to access the tops of the lifting girders at both ends of the lift span. The stairs to the machinery room were relocated.

Minor modifications to the control house/machinery room were made including carpeting, new windows, and doors. The area to be carpeted was first to have furring strips nailed to the concrete and plywood nailed to the furring strips. Additional insulation was added to all walls and ceiling of the control house.

Electrical wiring and controls were modified to control traffic during lifting of span. (Sheets 36 – 38)

1984

In 1984, the State of Maine rebuilt the roadway on Badgers Island and replaced selected granite and truncated pyramidal concrete piers with reinforced concrete cylindrical piers. All piers on column lines #1 to #9 on the east and west side, with the exception of #6 West and #9 east, (note:

pier numbers run from south to north) were replaced. The process was to shore up the girders, remove all concrete and stone above the footing, lag stay-in-place wood forms to the concrete footings and pour a new concrete footing to a point above mean high water. Three-foot diameter concrete columns were then poured on the extended footing up to the beam bearing elevation. On all other columns, wood stay-in-place forms were lagged to the existing concrete, enveloping the existing stone. Concrete was then poured up to mean high water and the existing truncated pyramids patched as necessary. The concrete deck was patched, a waterproof membrane placed and a 2" hot bituminous concrete pavement installed.

1985

In January 1985 one of the counterweight cables broke. A cable that was removed in the 1981 restoration was used temporarily until a new cable could be fabricated. The new cable had to be stretched before it could be installed according to Henry McCone of the New Hampshire Public Works and Highway Department.

1999

In 1999, new barrier gates were installed at mid-span of the flanking spans along with necessary controls. The existing barrier gates at the beginning and end of each flanking span were retained. CPM Constructors of Freeport, Maine were the contractors on this and the Sarah Long Bridge. Work was to be complete by June 2000.

Power Cables for the bridge were replaced on September 14 and 15, 1999.

On October 19, 2000 the bridge was closed for a week while a severed uphaul or downhaul cable on the bridge was replaced.

2001

In the summer of 2001, Cianbro Corp. of Pittsfield, Maine, replaced fourteen counterweight cables. The bridge was out of service for about a week and the contract was for $53,000

c. Conditions

Memorial Bridge is in excellent condition for a bridge of its vintage, having been well maintained over its eighty-year life. Problems with the bridge are typical for bridges of its type, material, and age. Although the bridge suffers from rusting of various elements of the truss and deck system and deteriorated concrete surface conditions, these conditions can be remedied. The operating machinery needs updating but continues to function adequately. The detailed discussion which follows is based on the inspection report completed as part of this project.

Deck Area

Open Grating (Lift Span): The lift span open steel grating is typically lightly rusted. There are a few isolated cracked welds for the grating and some loose sections. Numerous purlins supporting the deck are heavily deteriorated in the wheel line areas along the span.

Reinforced Concrete (Flanking Spans): The concrete decks of the flanking spans are in poor condition with popouts, spalls and honeycombing found in every span. The deck is directly supported by transverse purlins in all spans and is typically spalled around the curbline drainage scuppers. The deterioration to the purlins around the scuppers is so consistent along the outside edge of the deck that the deck itself has lifted away from the purlins by approximately ¼" (6 mm) due to impacted rust between it and the top flange. The deck now squeaks against the purlins with passing traffic in all of the approach spans, with the heaviest occurring at the east side of the bridge.

Wearing Surface:
The open grating of the lift span is slightly worn. The pavement has settled on both sides of the concrete header for the deck joint at the north abutment with minor associated hairline cracks, especially on the north side. The north flanking span pavement is settled with full width and length cracks in the patched area 20'-25' from the abutment.

Deck Joints:
The concrete header for the north abutment deck joint is worn on the top side with exposed aggregate and some minor cracks along the joint steel. The steel joints are lightly rusted. The steel fingers are slightly misaligned. The expansion joint support brackets at pier 4 are heavily deteriorated with heavy rusting throughout and up to 5" high holes at the bottom of the bracket at the floorbeam connection. The shim plates between the bracket and the support beam are not fully bearing at the bracket adjacent to stringer 7; the shim plates overhang the bracket by 1⅜" (35 mm).

Sidewalks:
The sidewalk timbers are heavily worn with checks, splits, and some decay to the top side, resulting in an irregular surface. The concrete curbs have numerous longitudinal and transverse cracks along the length of the north approach spans. There are numerous scrapes and chipped away segments at the edges of the concrete curbs and transverse header, mainly on the west side, for the north 20' of the bridge. The curbs and the sidewalks have settled 1¼" at the bridge concrete sidewalk header at the north abutment. There is vegetation growth at all north flanking span sidewalk joints.

Bridge Rail:
The bridge rail is moderately rusted throughout, with several areas of heavy to severe rusting and 100 percent loss. There are holes in the lower rail at the east side of span 1 (at L3 and L6); and at the west side of span 3 (at L5). At many of these locations the rail has separated from the rail post. The rail posts are typically heavily rusted at the bases of the posts along the exterior stringers; most have holes at this location. The northwest approach guardrail and endpost are ivy covered. There is no transition from the bridge rail to the endpost or guardrail.

Drainage:
The scuppers or downspouts in the flanking spans are typically heavily deteriorated and holed-through in all locations and leak onto the curbline stringers at midspan.

Superstructure
Purlins:
The lift span deck purlins are in fair condition with deterioration and holes found in the purlins above the two outermost roadway stringers – stringers S3 and S4 and stringers S8 and S9. The purlins in the concrete flanking spans are deteriorated around the downspouts and at other isolated locations.

Two to three purlins adjacent to the downspouts are typically deteriorated on the webs and flanges for the end 3' of the purlin. The deterioration to the purlins is so consistent along the outside edge of the deck that the deck itself has lifted away from the purlins by approximately ¼" due to impacted rust between it and the top flange. The deck now squeaks against the purlins with passing traffic in all of the flanking spans.

Stringers
Lift Span: Some of the lift span roadway stringers are in critical condition with severe rusting and numerous holed-through areas for the entire length of the members. There are numerous other roadway stringers (S3 through S9) that are heavily deteriorated and holed-through with flange losses at midspan and web losses at the floorbeam connections. The sidewalk stringers (S1, S2, S10 and S11) are moderately to heavily rusted throughout, with the heaviest rusting and losses occurring at the connections to the rail posts.

Flanking Spans 1 and 3: The outside roadway stringers (S3 and S7) are typically heavily deteriorated at midspan below the heavily deteriorated and leaking downspouts. The remaining roadway stringers (S4, S5 and S6) have other areas of heavy rusting and deterioration with flange losses at midspan and web losses at the floorbeam connections. The sidewalk stringers (S1, S2, S8 and S9) are moderately to heavily rusted throughout, especially at the flanges, with the heaviest rusting and losses occurring at the connections to the rail posts.

Floorbeams/Cross Girders
Lift Span: The floorbeams in the lift span are typically heavily rusted and deteriorated, with web holes and section losses throughout, especially floorbeams 1, 4, 6, 8 and 9. Some of these holes are around stringer connections. Most floorbeams have web losses and holes up to 7" high near the connections to the trusses. Portions of several floorbeams (1, 2, 4-6, 8 and 9) were previously repaired with bolted and welded web plates. Many welds were found in the tension zones of these fracture critical members.

Flanking Spans (Spans 1 and 3): The floorbeams for the flanking spans have isolated areas of heavy rusting and pitting with web holes, typically 3" high, near the connections to both trusses. Many of the floorbeam cantilevers supporting the sidewalk stringers have similar web holes up to 5" high and some top flange losses near the connections to the trusses. There are full height web losses and holes (up to 5" high) above both bearings for the floorbeam, with the heaviest losses above the east bearing. There are isolated areas of heavy rusting with losses to the remaining flanking span floorbeams.

Truss Members:
The steel truss lower chord members, upper chord members, and web members at the lower panel points in spans 1-3 are typically moderately to heavily rusted with pitting and section losses. The diagonals and verticals have several existing irregularly welded repairs, particularly at the lower panel points. Many of these diagonals and verticals are fracture critical members. At a few locations, the lower 2-2.5' of the outstanding legs of the diagonals were cut away at the lower panel point so that repairs could be made to the vertical gusset plates. The lower chords have miscellaneous irregularly welded batten plates and other repair plates. All of the lower chords are fracture critical members. Several of the lower chord lacing plates and batten plates are heavily deteriorated. The rivets at the top of the top chords typically have heavy head loss.

Towers:
The steel towers are lightly to moderately rusted throughout, with some areas of heavy rust. There is debris and vegetation growth on the counterweights for the lift span. The tower gusset plates and connection plates have areas of heavy rusting with some holed-through plates. Impacted rust between the tower members and the gusset plates has caused some of the gusset plates to bulge up to 1".

Bearings

Flanking Spans (Spans 1 and 3): The bearings for the trusses are lightly to moderately rusted throughout, with some areas of heavy rust. Some impacted rust was found between the pin nut, bearing, and vertical gussets at the east truss for span 3 at pier 2. The rocker bearing for the west truss of span 2 at pier 2 is out of plumb and appears to be frozen in the contracted position. The west vertical gussets at both bearings for span 3 at pier 3 have $^{3}/_{16}$" deep pitting around the pins.

Connections and Plates:

The west connection angle for stringer 7 at floorbeam 6 in bay 7 of span 1 is cracked for 75% of its height. The vertical gusset plates at the lower panel points at each truss in spans 1-3 are typically moderately to heavily rusted with some holed-through areas. Some are severely deteriorated with several holes. Several of the upper and lower lateral bracing gusset plates are heavily deteriorated. The bracket connecting the north flanking spans exterior sidewalk channels to the floorbeams is typically heavily deteriorated and holed-through.

Bracing:

The upper and lower lateral bracing in spans 1-3 are heavily deteriorated at several locations. The sidewalk stringer bracing for the north flanking spans is typically deteriorated and broken away at several spots, especially at the east side of the bridge. All of the sidewalk stringer bracing is broken at the east side of spans 4-10.

Substructure:

Pier 1 has hairline cracking, throughout and a ¼" wide vertical crack in the center of the stem. There are also various areas of spalling and delamination, including spalling that partially undermines a stringer bearing. The crack at the middle of the pier stem appears to be displaced, possibly the result of differential settlement. Since the pier is founded on rock it is presumed that this crack occurred early on in the life of the structure and does not present a situation that will continue to worsen.

d. Foundation Considerations

The existing bridge has three spans, 298.75', 302.5', and 298.75' long. The bridge foundation components consist of a south abutment, south pier, north pier, and north abutment. The available bridge plans consist of five design plans, dated 1920, and five record drawings of as-built conditions, dated 1923. Based on the plans, all four foundation units are concrete footings constructed on ledge. There is no evidence of steel reinforcement within the south abutment, while there is limited reinforcement within the south pier, north pier, and north abutment.

Very limited information is available from the existing plans regarding the subsurface conditions. The general anticipated stratigraphy is a thin alluvial deposit over bedrock with the bedrock surface ranging from less than 3' to 20' below the existing mudline. The bedrock surface appears to be the lowest beneath the center span of the bridge.

2. Portsmouth Approach

a. Description

The design of the approach is a one way continuously 1'–6" thick reinforced slab with edge beams. The approach consists of an embankment on what is now called Scott Avenue to a reinforced concrete retaining wall and abutment.

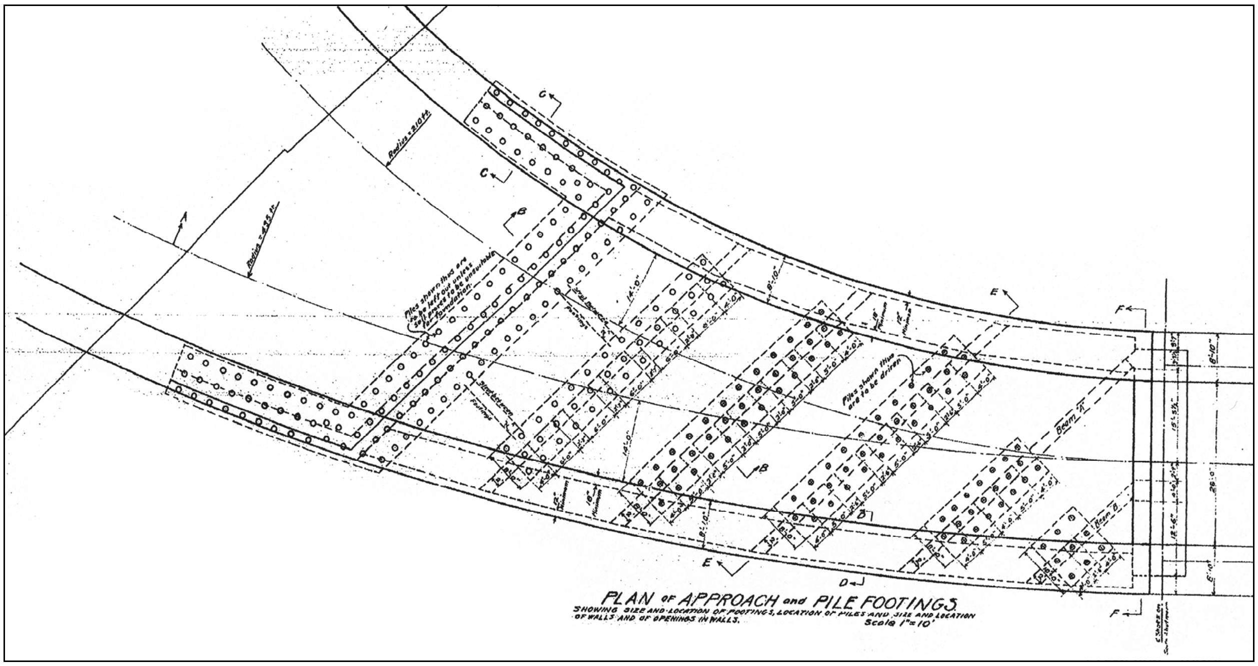

Figure E-15: Portsmouth Approach (see full drawing at O-19)

The height of fill varies from 0' to 16'. Five piers are 20' apart and parallel to the abutment. The length of these piers varies due to the curvature of the roadway. The northerly two piers were modified due to their proximity to the end of the southerly flanking span; these are shorter than the first three and have beams that extend from them to the southernmost pier of the main bridge to support the deck. From the abutment, a reinforced concrete deck on five spans extends to the first river pier. The deck is variable in width to allow traffic heading towards Portsmouth to swing to the right on an S curve to connect with Daniel Street. Traffic heading towards Kittery comes on to the bridge from State Street on a 435' radius curve. This results in the deck varying from a width of 50' at the abutment/retaining wall to 28' at the beginning of the first flanking/tower span. The abutment and five piers are set on wood piling with a 10' thick truncated pyramidal reinforced concrete footing. Two 4' x 2' columns and four 2'–6" x 2' columns spaced 5' apart and reinforced with ten ¾" square bars, extend to a haunched beam below the deck. The transverse beam spanning between the columns is 5' deep and 2' wide. The 1'–6" thick deck is reinforced continuously for its entire length with ¾" square bars at 4" on center on the lower face and ¾" bars in the upper face over the haunches on the same spacing. Every other bar is bent up from the lower tension face to the upper tension face and then back down to the next lower tension face. Additional 12' long top bars are placed between the bent up bars to provide the necessary tensile reinforcement over the haunches. Temperature reinforcing runs perpendicular to the main steel with ½" square bars on a spacing of 12".

The sidewalks are supported on 2' wide variable depth – 6' to 3' – cantilever beams formed as extensions of each pier beam. Between these cantilevers a 1'–6" wide edge beam 3'–3" deep, including a curb, follows the curvature of the deck. These beams were cast monolithically with the 6" thick reinforced concrete sidewalk slab. In addition, an outer 1' wide, 3' deep concrete beam was cast monolithically with the sidewalk slab and also followed the curve of the deck. The sidewalk width, out to out, was originally 8'–10" and had iron railings anchored to the concrete. Currently, railings on the part of the approach over the piers are aluminum and consists of three horizontal rails with posts every 6'–0" at the south end and every 7'–3" at the north end. The railing over the abutment consists of wooden posts and rails. The sidewalks on the approach consist of open metal grates supported by longitudinal I-beams and steel brackets to the north and plain concrete sidewalks to the south.

Running between the two most southern piers of the approach, the road running between Daniel and State Streets in this area is sharply curved. The other bridge bays are used as pedestrian crossings and for parking. The height of the underpass is 12'–2".

There have been significant alterations to the bridge over time, particularly to the sidewalks and railings. In April 1977, the approach went through a major rehabilitation. During this repair, the sidewalks were replaced above the abutments and the abutment itself was repaired with the upper level of concrete removed and replaced. This concrete was cast monolithically with the sidewalks. At this time also, the original 8'–10" concrete sidewalks on the portion of the approach over the piers which were cantilevered off the edge beams, were replaced with the current metal grate sidewalks. In 1950 there also was significant work done replacing sidewalks and curbing as well as railings. Photos from the opening of the bridge indicate that the original railings on the approach were pipe railings that matched those of Memorial Bridge.

b. Conditions

Deck and Superstructure

The bituminous wearing surface is considered to be in good condition. Minor cracks were observed throughout.

The deck and curbs are considered to be in poor condition. The shotcrete on the underside of the deck exhibits signs of cracking and delamination from the original structure. There are also large areas of efflorescence and rust staining on the soffit. There is water leakage and exposed rebar at the drainage pipe–deck interface.

The concrete along the curb line on both the east and west sides of the bridge exhibits several areas of significant spalling, large cracks, and exposed rebar. The remaining portions of the sidewalks exhibit varying degrees of concrete deterioration.

In 1977, the concrete sidewalks were replaced with open metal grates supported by three (3)-steel I-beams, equally spaced. The sidewalks are considered to be in satisfactory condition. The open metal grates exhibit no signs of deterioration. The steel I-beams show minor rusting on the top flanges and the top surface of the bottom flanges. At the south end, a concrete shelf cut into the abutment backwall supports the I-beams. Significant debris has built up on this shelf covering the ends of the I-beams. A bracket consisting of steel angles and channels attached to each pier cap also supports the sidewalk I-beams. The support bracket shows minor rusting and is covered with shotcrete at the piers. The support bracket attached to the east side of Pier 1, shows signs of collision damage.

An existing electrical conduit runs across the bridge along the soffit at the west fascia. A PVC drainpipe extends down from the deck at the east and west side of the south abutment and at the west side of the deck at Pier 2. The drainage pipe extends down to the base of the south abutment and pier column. The PVC drainage pipe exhibits signs of leaking near the deck and has caused deterioration of the surrounding concrete elements.

Substructure
The abutments, wingwalls, pier caps and pier columns are considered to be in fair condition. The footings of all substructure elements are below ground and were not inspected.

The south abutment exhibits numerous small cracks, efflorescence, rust staining, and large areas of delaminated shotcrete. The top of the southwest wingwall, at the first bridge rail post, has a large concrete spall. Minor erosion was observed at the end of the southwest wingwall. Both wingwalls exhibit cracking, delaminated concrete, efflorescence and areas of spalled concrete.

The north abutment exhibits large cracks, delaminated concrete, efflorescence, and areas of spalled concrete.

The pier caps and columns have numerous small cracks, efflorescence, rust staining, and exposed rebar. Collision damage to the western most column of Pier 1 and to the eastern most column of Pier 2 observed.

Roadway Approach Condition
The roadway approaches are considered to be in good condition. The south approach bituminous pavement exhibits minor cracking. The south approach sidewalk consists of granite curb with a bituminous sidewalk. The sidewalks exhibit minor cracks and areas of settlement. The south approach rail consists of timber rails and posts. The north approach is the Memorial Bridge.

Material Testing Findings
On October 6 and 7, 2003, New England Testing Company, Inc. extracted cores from the bridge deck, soffit, abutments, and piers. After the visual inspection was completed, areas of concern were noted, marked and a core was extracted. Each column was labeled and marked so the visual inspection and material testing coordinated with the individual findings (reference Figure 2 for the column identification codes). A total of twenty cores were drilled; five from the soffit, three from the abutments, seven from the piers and five from the top of the deck. Out of the twenty cores taken, only thirteen could be tested due to the poor condition of the concrete samples. The compressive strength test results ranged from 3012 to 4736 psi for the deck, 2676 to 4162 psi for the soffit, 2918 to 4622 psi for the abutments, and 2272 to3269 psi for the piers. The chloride ion content results indicate levels of chloride ranging from 1.4 lb/yd^3 to 2.8 lb/yd^3. These chloride ion content levels fall within the threshold of active levels.

Additionally, half-cell corrosion potential testing was performed on the concrete deck to determine the level of corrosion potential. The results indicate that approximately 25 percent of the bridge deck is “approaching active” or “active” state of corrosion potential. The remaining 75 percent of the bridge deck ranges from “threshold” to “normal”.

Condition Evaluation
The FHWA sufficiency rating for this bridge, based on the NHDOT 2001 Inspection Report, is 56.2 percent. The bridge is classified as Functionally Obsolete.

c. Foundation Considerations

Very limited information is available from the existing plans regarding the subsurface conditions. The general anticipated stratigraphy is a thin glacial deposit (glacial outwash or till) over bedrock with the bedrock surface approximately 5' to 15' below the existing grade. The bedrock surface appears to slope downward toward the river. Groundwater levels are anticipated to be slightly above adjacent river level.

Based on the above understanding of the existing conditions, our preliminary conclusions and recommendations are as follows:

New spans should be designed such that the foundations can be constructed between the existing foundations.

Use rock bearing spread footing for the south abutment.

If the bridge is closed, use drilled shafts with rock sockets for piers where the depth to bedrock and the groundwater table make spread footings impractical.

If the bridge is not closed, the viable pier foundation is basically the same; however, drilled shaft diameters and rock socket lengths would be reduced, due to the lower capacity equipment that could function below the approach.

3. Maine Approach

a. Description

The plan and profile of the approach structure from Badgers Island to the north pier of the flanking span is shown below. The depth to rock changed significantly over the length requiring some piers to be set on piles while others were set directly on bedrock

Concrete footings, generally 10' x 10' and of variable depths, were placed, either on piling or bedrock (see profile above), depending on the depth to rock, with their top surface placed at mean low water elevation. Courses of granite block on eight of the piers, backfilled with concrete, raised the piers to 2'–0" above the mean high water mark. From that elevation, truncated concrete pyramids ranging from 6' square at the base to 3'–4" square at the top and 12' high were built. The pyramids were topped with blocks of concrete 4' square and 1'–8" thick to reach the base plate elevations of built up crossing girders.

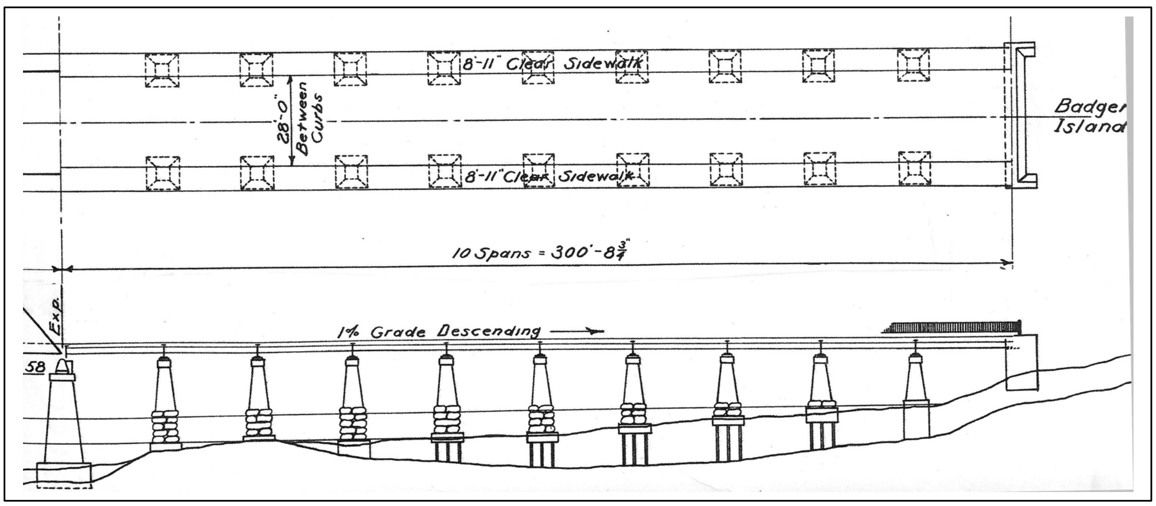

Figure E-16: Badgers Island – Kittery Approach Span

Plate E-3: Understructure of Kittery Approach showing 1984 rehab. Note single remaining truncated pyramid support, and new cylindrical columns.

The Maine Highway Department designed the ten-span structure to cover the 300'–$8\frac{3}{4}$" distance from the main bridge to the Island. The span lengths from south to north (left to right) are 32'–$2\frac{3}{8}$", eight at 29'–$8\frac{3}{8}$" and 30'–$11\frac{3}{8}$". The width of traveled way, between curbs, is 28'. The deck system consists of 36" deep transverse main floor girders built up with a web plate with double 6" x 6' x $\frac{3}{4}$" angles top and bottom supported by a pair of piers. Riveted to the webs of the girders are five (5) longitudinal 24" - "I" stringer beams weighing 77.9 #/ft at a spacing of 6'–6". Cross beams of 7" - "I" beams weighing 17.5 #/ft at a spacing of 2'–5" or 2'–$9\frac{1}{4}$" support the concrete deck. The deck consists of a $5\frac{1}{4}$" thick reinforced concrete slab topped with a 2" thick asphaltic concrete surface. Four lines of 4" - "T" beams were cast into the slab at the proposed location of tracks for interurban trolley cars in the event they were to run over the bridge. Sidewalks 8'–10" wide are supported on extensions of the main transverse floor girders. Two stringer beams support a 3" wood plank floor with iron railings.

b. Conditions

Deck Joints:
The expansion joint support brackets at the south side of the end floorbeam 1 for span 1 of the Kittery Viaduct (at pier 4 of the Portsmouth Memorial Bridge) are heavily deteriorated with heavy rusting and up to 5" high holes at the bottom of the bracket at the floorbeam connection. The shim plates between the bracket and the support beam are not fully bearing at the bracket adjacent to stringer 7; the shim plates overhang the bracket by $1\frac{3}{8}$" (35 mm).

Sidewalks:
There are three loose sidewalk planks on the west sidewalk in span 2 of the Kittery Viaduct.

Wearing Surface:
The pavement on the top of the deck in spans 1-10 of the Kittery Viaduct has 2' long transverse cracks randomly spaced at the centerline of the roadway and at the middle of the southbound lane. The pavement has settled on both sides of the concrete header for the deck joint at the north abutment with minor associated hairline cracks, especially on the north side. The north approach pavement is settled with full width and length cracks in the patched area 20'-25' from the abutment.

Bridge Rail:
There are holes in the lower bridge rail at spans 1, 2, 3, 5, 6, and 8 (two areas) of the Kittery Viaduct; and spans 3 and 9 of the Kittery Viaduct. The top rail has a hole at the east side of span 6 of the Kittery Viaduct. There are holes through the east rail posts for the Kittery Viaduct in spans 2 and 3 and over the north abutment in span 10. At many of these locations the rail has separated from the rail post. The rail posts are typically heavily rusted at the bases of the posts along the exterior stringers; most have holes at this location. The northwest approach guardrail and endpost are ivy covered. There is no transition from the bridge rail to the endpost or guardrail.

Superstructure
Overall, the concrete decks are in poor condition with numerous popouts and spalls, particularly around the scuppers. Additionally, impacted rust on various purlins has caused deck separation from the steel structure. This condition is prevalent throughout all spans of the structure.

Purlins:
The purlins in the concrete approach spans, including the Kittery Viaduct, are deteriorated around the downspouts and at other isolated locations.

Stringers:
North approach spans 1-10 of the Kittery Viaduct: The east roadway stringer (S7) is typically heavily rusted adjacent to the downspout with $\frac{5}{32}$" loss to the bottom flange. The remaining roadway stringers (S3-S6) are lightly rusted throughout. The exterior sidewalk channel stringers are heavily deteriorated with flange and web losses (typically $\frac{1}{8}$" loss to the top flange) at the east side of the bridge and at isolated locations at the west side, especially at the connections to the bridge rail posts.

Floorbeams/Cross Girders
Spans 1-10 of the Kittery Viaduct: The floorbeams for approach spans 1 and 3 have isolated areas of heavy rusting and pitting with web holes, typically 3" high, near the connections to both trusses. Many of the floorbeam cantilevers supporting the sidewalk stringers have similar web holes up to 5" high and some top flange losses near the connections to the trusses. The end floorbeam for span 1 of the Kittery Viaduct at pier 4 of the Portsmouth Memorial Bridge is pitted throughout with several web holes and flange losses at midspan. There are full height web losses and holes (up to 5" high) above both bearings for the floorbeam, with the heaviest losses above the east bearing. There are isolated areas of heavy rusting with losses to the remaining approach span floorbeams.

Bearings:
Spans 1-10 of the Kittery Viaduct: The top of the bearing surfaces (original plates) are typically pitted and painted over. There is heavy rust and impacted rust (up to $\frac{5}{8}$" at east bearing for pier 1) between the top original plate and the replaced bearing/masonry plate. The bearings for span 1 at pier 4 of the Portsmouth Memorial Bridge are heavily rusted. The bearing plate exposed is

virtually 100 percent deteriorated and is paper thin. Anchor bolt heads are typically 50 percent deteriorated and one anchor bolt at each bearing is 100 percent deteriorated. Similarly, both bearing anchor bolts for the west bearing at pier 9 are 100 percent deteriorated below the top plate of the bearing leaving no remaining anchor bolts at the bearing. Both bearing anchor bolts at the west bearing for pier 6 are 95 percent deteriorated. There is a missing anchor bolt at the east bearing for pier 8.

Bracing:
The sidewalk stringer bracing for the north approach spans is typically deteriorated and broken away at several spots, especially at the east side of the bridge. All of the sidewalk stringer bracing is broken at the east side of spans 4-10.

Substructure:
The reinforced concrete columns at piers 6, 8 and 9 have spalls, the original granite footing blocks are worn on the top side with deteriorated mortar joints, the pier 8 columns have cracks and heavy scaling on the exterior faces, the reinforced concrete north abutment exhibits similar defects as pier 1 on the main portion of the bridge, and there are some pedestals that exhibit spalls and undermining at the base.

c. Foundation Considerations

The existing approach consists of nine piers and one abutment, at approximate 30' spacing. Four of the piers appear to be supported by footings constructed directly on bedrock. The remaining piers are supported by piles driven to bedrock, with pile lengths ranging from approximately 6' to 13'. The piles appear to be embedded approximately 2' into the pile caps.

Very limited information is available from the existing plans regarding the subsurface conditions. The general anticipated stratigraphy is similar to the Memorial Bridge, with the bedrock surface ranging from ground surface to 13' below the existing grade.

Based on the above understanding of the project and existing conditions, our preliminary conclusions and recommendations regarding liquefaction and seismicity for the Kittery Approach are as follows:

1. We anticipate that the subsurface conditions include saturated, relatively loose sand deposits, which would be liquefaction susceptible during seismic loading. However, both foundation types are founded on bedrock, which provides adequate vertical support under seismic loading.
2. There is a significant concern regarding performance of the pile foundations under earthquake loading. Most of the pile foundations are shorter than is generally acceptable under current design codes. Also, the pile to cap connection is probably inadequate with respect to current codes.

4. Seismic and Scour Assessments

a. Seismic Assessment

A seismic evaluation has been conducted to assess the vulnerability of the main load carrying elements of the structure to a seismic event. In accordance with the NHDOT Bridge Design Manual, this bridge falls within Seismic Performance Category (SPC) B and is therefore evaluated for an earthquake consistent with a rock peak ground acceleration of 0.17g.

Discussion of Seismic Hazard at the Site

It is noted that there has been a significant effort put forth by the United States Geologic Survey (USGS) to develop more accurate estimates of seismic hazard throughout the United States. This National Seismic Hazard Mapping Project has resulted in the development of amended seismic hazard maps that represent significant revisions to the maps adopted by AASHTO (that were originally developed by the USGS in 1988 for the National Earthquake Hazard Reduction Program (NEHRP)). As is demonstrated in the figures below, there are significant reductions in seismic hazard associated with the 1996 USGS maps, with a peak ground acceleration for bedrock estimated at 0.06g (as compared to 0.15g in the 1988 maps).

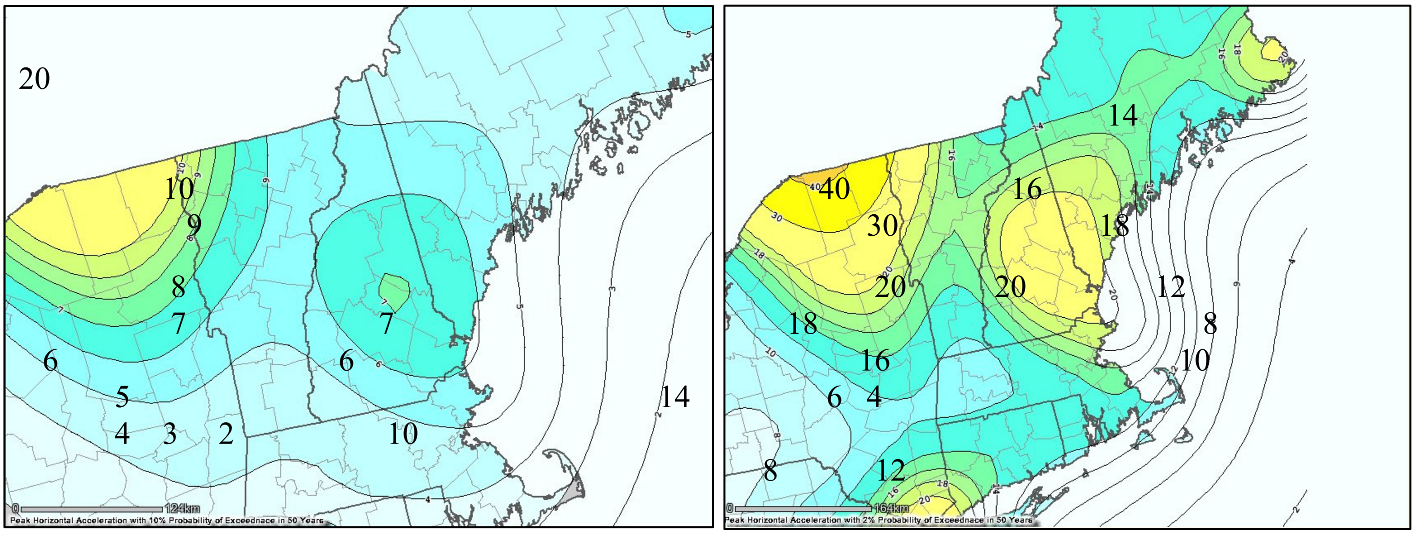

Figure E-17: 10% in 50 Year PGA (left) and – 2% in 50 Years PGA

The 1996 USGS maps have been adopted in the *Recommended LRFD Guidelines for the Seismic Design of Highway Bridges, NCHRP 12-49 (2001),* which currently serves as a guide specification prior to formal adoption and incorporation into LRFD. One major revision is the change in return period for bridge evaluation. These specifications recommend the use of a Maximum Considered Earthquake (MCE) for a safety evaluation (i.e. no collapse). This return period approximately corresponds to the 2 percent in fifty-year event that has a peak ground acceleration of 0.20g, somewhat in excess of the 0.17g specified in the current NHDOT Bridge Design Manual. However, changes in the shape of the response spectra reduce the impact of this increase in PGA, particularly for bridges on rock.

Given the above, we have conducted our evaluation in accordance with current NHDOT standards, i.e. with a rock spectra of 0.17g using the AASHTO spectral shape based upon a $T^{2/3}$. However, for final design, it is recommended that the seismic hazard for the site be revisited since seismic hazard may have a significant impact on overall retrofit costs.

Basis for Evaluation – Elements Evaluated

Given the potential for both intermediate and extensive levels of rehabilitation, we have evaluated the structure using the following guidelines:

- Remove and replace rocker bearings
- Tie together / restrain girders in multiple simple span structures
- Evaluate deck continuity at pier locations

- Evaluate the adequacy of confinement reinforcing at the columns/piers
- Design structure to resist all forces as required for a new structure**

**Note that this criterion applies only for structures where rehabilitation is considered extensive.

Analysis Methodology
Given the unusual nature of the main span units, single mode analysis techniques do not adequately capture behavior. Therefore, a 3D multi-mode analysis was conducted using GTSTRUDL. Movable bridges must be evaluated in the span up and span down condition, and this evaluation becomes particularly important when the bridge is in the span-up position for more than 10 percent of the time (if not, the seismic forces are taken as 50 percent of the forces in the span-up position). Given 4500 openings and an average cycle time of 10 minutes, we are slightly below the criteria and therefore are permitted the 50 percent reduction of force in the span up condition. For the approach spans, an equivalent single degree of freedom approach was conducted.

Substructure
The piers for the Memorial Bridge are unreinforced concrete and data on the strength of the concrete is not available. We have conservatively estimated the concrete strength at 3000 psi, though we would recommend coring to provide a definitive estimate of concrete strength. Given the magnitude of the shear forces introduced at the bearings, it is likely that localized shear failure will be anticipated below the pedestals (it is noted that the pedestals were rehabilitated in the early 1980’s and are somewhat less vulnerable, presuming the grillage beams installed in the original construction remain in good condition or were properly rehabilitated).

Plate E-4: Rocker Bearing (right)

Given the articulation of the structure, the most vulnerable pier is pier 2 since both span 1 and span 2 have a fixed connection to this pier. However, due to the massive size of these substructure units, they have sufficient shear and flexural strength to resist seismic forces, provided that the concrete strength approaches 3000 psi. However, it is likely that a large seismic event would cause significant damage to these unreinforced substructure units and it is recommended that the installation of grouted post-tensioning bars (similar to rock anchors) be considered to reduce the potential for damage and enhance the overall strength of the substructure units.

Bearings and Connections
The connection of the bearings to the substructure is inadequate to develop the seismic loads, with demands approximately double the capacity (C/D ratios range from 0.45 to 0.7) for fixed bearings. This is with the assumption that the anchor bolts are comprised of A7 steel (60 ksi ultimate strength) and are 2½" diameter (consistent with the original design drawings). In addition, rocker bearing pintels are inadequate to resist transverse seismic forces with similar C/D ratios. Note that these C/D ratios assume no section loss or deterioration to the anchor bolts.

Rocker bearings are seismically vulnerable to toppling and are typically replaced as part of a bridge seismic retrofit program. This combination of fixed bearings with inadequate connection strength to resist seismic forces combined with expansion bearings is a particularly vulnerable combination and will require retrofit.

Under ideal circumstances, it is preferable to replace rocker bearings with seismic isolation bearings in order to reduce the seismic forces transmitted to the substructure. However, due to the operational requirements associated with movable bridges, particularly vertical lift bridges where skew control is vital to the operation of the lift span, the use of flexible seismic isolation bearings is not recommended. A detailed evaluation of feasibility may demonstrate that some level of isolation may be achieved, without impacting bridge operations, particularly for the lift span, but it is unlikely that the tower spans can be seismically isolated effectively.

In any case, substantial strengthening measures will be required at fixed bearing locations. It is recommended that all rocker expansion bearings be replaced with bearings that are not seismically vulnerable, such as disk bearings.

Superstructure
For the superstructure, a limited number of diagonals, together with the bottom chords at the end panels, are vulnerable due to the substantial section loss associated with corrosion. Significant rehabilitation is anticipated for these members and it is anticipated that this will be sufficient to overcome any seismic deficiencies. All main load-carrying elements of the towers are not vulnerable, although CD ratios approach 1.0.

The span and counterweight guides are inadequate to resist seismic forces and are assumed to fail. Provided relative displacements between the counterweight/lift span and the tower are not significant, this does not represent a hazard to the bridge, although it will not be operational following a seismic event.

It is noted that the superstructure evaluation assumes that the current deck system will remain. If the grating in the lift span were rehabilitated to heavier concrete filled grid deck, it is likely that a number of tower elements will require retrofit due to the increase in counterweight size. Particularly, the rear legs of the towers will become vulnerable with any increase in the counterweight.

b. Scour Assessment

A scour assessment of the Memorial Bridge has been completed. This assessment included gathering and review of available data, assessment of the erodibility of rock formations, and an assessment of the stability of the Kittery Approach piers should scour of erodible soils occur at those locations. A comprehensive hydraulic and scour analysis was not included in the scope of work.

Geology / Foundations
The bridge consists of two concrete abutments, three concrete piers and eighteen reinforced concrete columns. The abutments and three concrete piers bear directly on ledge. The eighteen approach pier columns are supported on piles that also bear on the bedrock.

Rock Scourability
The Federal Highway Administration (FHWA) has issued a Memorandum titled "Scourability of Rock Formations," dated July 19, 1991, which gives interim guidance for assessing rock scourability. The memorandum provides various methods to assess rock scourability such as Rock Quality Designation, Unconfined Compressive Strength, Slake Durability Index, among others. Sufficient geotechnical data however, is not available to assess the scourability of the existing rock formations based on the FHWA methodology. The lack of documented scour of bedrock at the bridge suggests that the rock is not scourable.

History of Scour / NBIS Coding
Historical scour was investigated by reviewing available bridge inspection reports including the in-depth structural inspection and underwater inspection completed as part of this project. Routine inspection reports from December 1998 and March 2001 were also reviewed.

During our review of the previous reports only one mention was made of scour. The December 1998 report included a photograph of one of the Kittery Approach foundations with the footing undermined. By reviewing the original bridge construction plans, and comparing those plans to the photograph, there is an indication of several feet of scour at this location.

It should be noted that since only one underwater inspection was reviewed there is little historical data to indicate any longer term patterns of scour.

Adjacent Bridges
In assessing an individual bridge, it can be helpful to review records for adjacent bridges. Two bridges are located just upstream of the Memorial Bridge, the Route 1 bypass bridge and the I-95 bridge. An attempt was made to obtain hydraulic and scour reports and/or inspection reports for those bridges. However, no reports were available.

NBIS Coding
The FHWA's National Bridge Inspection Standards (NBIS) includes a coding for Scour, Item 113. Based on a review of available records the coding for the Memorial Bridge and the two upstream bridges are as follows.

Bridge Name	NBIS #	Item 113 Coding
Route 95	021702580012800	Foundation determined to be stable for assessed scour conditions.
Rte 1 Bypass	021702510010800	Foundation determined to be stable for assessed scour conditions.
Memorial Bridge	021702470008400	Foundation determined to be stable for assessed scour conditions.

Figure E-18: NBIS Coding for Scour

Piscataqua River Hydraulic Characteristics

The Piscataqua River is tidally influenced. Based on available data the tides are of the semi diurnal type, two high and two low tides occurring each lunar day. Per NOAA data (at Seavey Island just downstream of the Memorial Bridge) a mean tide range or 8.1 ft and a spring tide range of 9.4 ft is reported. The mean tide level is 4.4 ft.

Per the Federal Emergency Management Agency's Flood Insurance Rate Map (FIRM) the Memorial Bridge is located within an A Zone (A2) with a 100-Yr base flood elevation of 9 (NGVD 1929).

Very little river discharge or river current information was readily available. Maximum currents southwest of Badger Island in September 2003 were reported by NOAA as 4.2 knots (7.1 ft/s). A description of the Piscataqua River, obtained from an internet search, had references to currents as high as 4 knots (6.7 ft/s) although the source and validity of that current is unknown.

Conclusions

The bridge has been previously assessed, by others, as part of the NBIS coding program as "scour stable," presumably because it was considered to be founded on non-erodible bedrock. The bridge was constructed in the early 1920s and, based on available data, has not experienced significant scour issues since its construction. Based on inspection reports some scour has occurred at one of the Kittery Approach piers, this condition should be monitored, and countermeasures considered during any major improvements to the facility. Possible repairs or countermeasure might include constructing formwork around the undermined area and injecting concrete to fill the void and/or installing countermeasures such as riprap, gabion mats, or concrete arming units at the base of the pier(s).

The Piscataqua River appears to experience high currents. The waterway opening at the bridge is large and subject to tidal flows. The primary concern with respect to scour is local scour at the piers. The bridge should continue to be monitored for scour as part of the bridge's regular inspection program. During underwater inspections measurements of the mudline should be taken at the piers to document mudline elevation changes over time.