Long Island Rail Road Bridge Infrastructure

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The Long Island Rail Road (LIRR) bridge infrastructure comprises 396 railroad bridges. Most of the bridges were built in the early 1900s, and some were constructed as early as the 1890s. Because of their age and exposure to ever-increasing static and dynamic loading, many of these structures have reached or are nearing their useful service life. The LIRR's $2.0 billion, 10-year (1982–1991) Capital Improvement Program provided an opportunity and challenge to priority rank some of these bridges and program them for rehabilitation or replacement. Three of the projects are reviewed, and the LIRR bridge data base, load rating program, and bridge management process are discussed.

The Long Island Rail Road (LIRR) system comprises 396 railroad bridges. The system is subdivided into 15 branches (corridors) for bridge identification purposes. The various branches of the LIRR are shown in Figure 1. One hundred ninety bridges are located in the boroughs of Queens and Brooklyn in New York City. The remaining 206 bridges are on Long Island; Nassau and Suffolk counties have 116 and 90 bridges, respectively. The Montauk Branch has the largest number (148) of bridges, whereas the Bushwick Branch has the distinction of having the smallest number (2) of bridges. Table 1 gives the number of bridges, branch route miles, and bridge miles. The total route mileage on the LIRR system is 335.4 mi, of which nearly 12 mi are bridges. This translates to bridges being 3.6 percent of the system route miles. Table 2 gives the various types and numbers of bridges by branch.

Most of the bridges were built in the early 1900s, and some were constructed as early as the 1890s. Because of their age, exposure to weather, exposure to steam locomotives in the past, and increasing static and dynamic loading, many of these structures have reached or are nearing their useful service lives.

The LIRR has mixed operating conditions because the same bridges have to accommodate high-speed light passenger (electric) trains, heavy diesel passenger trains, and occasional slow, heavy tonnage freight trains (*1*). The LIRR operates about 850 trains on an average weekday, of which 700 are electric trains and 150 are diesel trains. The rolling stock ranges from the 270,000-lb (67.6-kip axle load) heaviest diesel locomotives to 100,000-lb (26.2-kip axle load) electric M-3 trains.

Nearly 60 percent of the LIRR's main-line tracks are electrified by third (contact) rail (*1*). The running rails are also used as negative returns for power. Some of the old steel bridges exhibit significant corrosion. However, whether stray currents are one of the contributing factors has not been quantified or studied. In reinforced and prestressed concrete structures, there is no apparent evidence of stray current damage (corrosion). The deterioration present in some of the older reinforced concrete bridges and viaducts appears to have been caused by poor drainage, the age of the structure, lack of maintenance, inadequate concrete cover over reinforcement, and deicing salts used for snow and ice mitigation in viaducts with passenger station platforms.

The new bridge structures are designed to the American Railway Engineering Association (AREA) Cooper Railway Loading, at present E-80. However, in the case of repair and strengthening schemes, the procedure is to conduct repairs to bring the structure or the component to at least its "as-built" condition.

REVIEW OF BRIDGE PROJECTS

In 1982 the Long Island Rail Road, with funds from its Capital Improvement Program or operating budget, embarked on various bridge projects. Three of the major projects are reviewed here.

Manhasset Viaduct

Description

The Manhasset Viaduct is located west of Manhasset Station on the Port Washington Branch of the LIRR. A view of the viaduct is shown in Figure 2. The Manhasset Viaduct is a single-track, open-deck structure built in 1897–1898. The structure has a tangent alignment.

The viaduct has 15 spans that, except at the viaduct ends, are approximately 75 ft above ground. The original construction entailed riveted deck girders supported by stone abutments and steel bents on concrete footings. The west approach span is 90 ft long. The tower spans are 30 ft long, and the intermediate spans are 54 ft long. Figure 3 shows the elevation of the Manhasset Viaduct.

According to the design drawings, the east end of all 54-ft spans and the west end of the 90-ft span are free to move longitudinally; hence the structure is not subject to thermal forces (*2*).

The towers are fully x-braced in both the longitudinal and transverse direction; hence the columns support axial loads only. The tower legs are inclined 1:6 in the transverse direction to provide resistance to lateral forces. The longitudinal bracing is designed for tension and compression, whereas the

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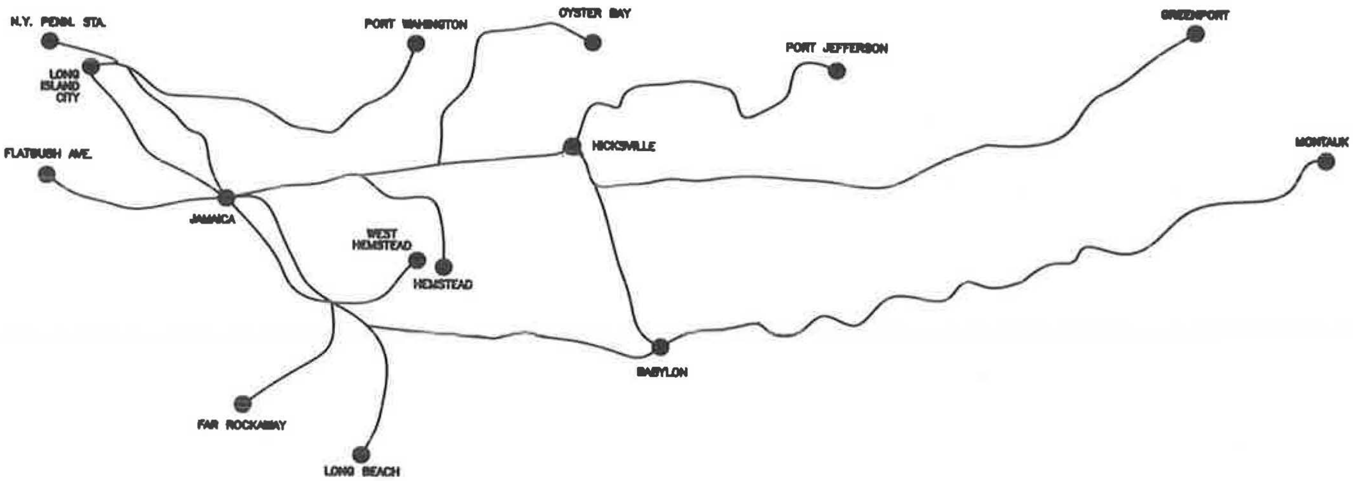

FIGURE 1 LIRR system map.

TABLE 1 INVENTORY OF LIRR BRIDGES

BRANCH	NUMBER OF RAILROAD BRIDGES	LENGTH OF BRANCH IN ROUTE MILES	TOTAL LENGTH OF BRIDGES IN ROUTE MILES
MAIN LINE	94	94.3	1.49
MONTAUK	150	115.8	6.54
NORTH SHORE	1	2	
MONTAUK CUT OFF	10	1	0.21
ATLANTIC	27	15.8	2.03
PORT WASHINGTON	25	16.3	0.59
BAY RIDGE	21	13	
WEST HEMSTEAD	5	4.6	0.44
HEMSTEAD	8	4.9	0.05
FAR ROCKAWAY	4	5	0.02
BUSHWICK	2	1.8	0.03
OYSTER BAY	15	14.3	0.22
PORT JEFFERSON	19	32.5	0.30
LONG BEACH	9	6.9	0.02
CENTRAL	6	7.2	0.06
BRANCH	396	335.4	12.0

transverse bracing is effective for carrying tension only because of its slenderness.

In 1938 the tower over Shore Road was modified by the addition of four vertical columns braced longitudinally and transversely as is the original tower. The new columns support the adjacent 54-ft spans leaving the original tower to carry only the 30-ft span within the tower. The new and old towers share any longitudinal forces, whereas the original tower is primarily effective in carrying transverse forces because of its inclined legs.

The project was programmed in different phases given the funding available at the time:

Scope of Work	*Year*
Report of condition survey	1980
Report of inspection, repair, and painting—Phase I	1983
Report of inspection, repair, and painting—Phase II	1983
Final design for renovation and painting—Phase III	1986
Construction	1987–1989

Findings of 1983 Inspection

Girders The girders had corroded significantly along the top and bottom flanges and on the upper surfaces of lateral connection plates. Webs of the girders were generally in good condition, although some localized rusting was present in areas where paint was not adhering tightly.

The amount of deterioration varied widely from point to point, but all girders had areas where the top cover plate was significantly reduced. At locations on the girders where the timber ties had previously been shifted, conditions were also variable. In the worst cases, the deterioration under the ties was considerably more extensive than in the adjacent spaces; there were deep craters around the rivet heads.

Towers The bent caps between girders were generally deteriorated at the outstanding leg of the bottom flange angle. However, previous maintenance repairs has remedied the deficiency by the addition of plates welded and spliced across the bottom of the cap beam.

Columns were in excellent condition and are stronger than originally built because of the addition of cover plates.

Footings and Abutments Footings for Bents 13 and 14 were badly deteriorated. At the east abutment, the girder ends were recessed into the backwall. The girder is intended to be free to expand and contract at the abutment. However, the backwall was restricting that movement, forcing both expansion and contraction to occur at the expansion end of Span 13-14.

TABLE 2 TYPES OF BRIDGES ON VARIOUS BRANCHES

BRANCH	BRIDGES	THRU PLAT. GIRDER	DECK PLAT GIRDER	I BEAM	TRUSS	CONC. BRICK ARCH	PRE-STRESS STRUCTURE	TIMBER OR TRESTLE
MAIN LINE	94	54	20	13	–	5	1	1
MONTAUK	150	88	25	13	2	5	16	1
NORTH SHORE	1	–	–	–	1	–	–	–
MONTAUK CUT OFF	10	3	6	1	–	–	–	–
ATLANTIC	27	16	8	3	–	–	–	–
PORT WASHINGTON	25	18	3	1	–	1	–	2
BAY RIDGE	21	8	11	–	1	1	–	–
WEST HEMSTEAD	5	4	–	–	–	1	–	–
HEMSTEAD	8	5	1	1	–	1	–	–
FAR ROCKAWAY	4	4	–	–	–	–	–	–
BUSHWICK	2	1	–	–	–	–	–	1
OYSTER BAY	15	7	2	1	1	4	–	–
PORT JEFFERSON	19	9	4	5	–	1	–	–
LONG BEACH	9	2	3	1	–	–	–	3
CENTRAL	6	6	–	–	–	–	–	–

FIGURE 2 View of Manhasset Viaduct.

Load Rating

Girders and columns of the viaduct were rated, according to the AREA Manual (Chapter 15, Part 7, Existing Bridges, 1984), for their as-built and as-inspected conditions (*3*).

Ratings were computed on the basis of open-hearth steel. If its nitrogen content is below 0.004 percent, or if it is not more than 0.012 percent and the phosphorus content is below 0.04 percent, steel can be considered open hearth. The steel samples tested fell into the latter category. A summary of the ratings is given in Table 3.

The as-inspected ratings were governed by the 30-ft spans with ratings as low as E-51, followed by the 54-ft spans with a lowest rating of E-63. The 90-ft span and columns all rated higher than their as-built conditions. The 30- and 54-ft spans

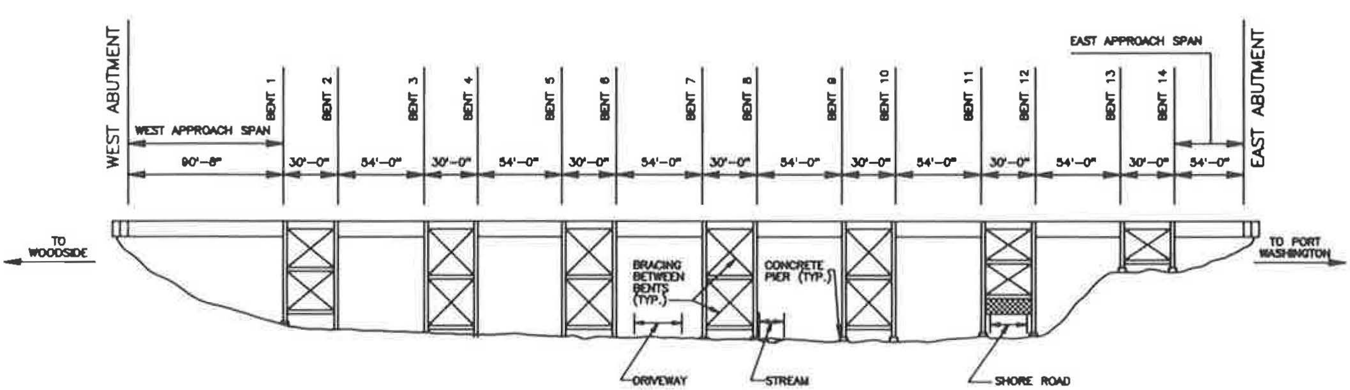

FIGURE 3 Elevation of Manhasset Viaduct.

TABLE 3 SUMMARY OF RATINGS

GIRDERS	AS–BUILT	AS–INSPECTED
WEST APPROACH SPAN	E73	E90
THIRTY–FOOT SPANS		
1–2	E63	E51
3–4	E63	E54
5–6	E63	E51
7–8	E63	E53
9–10	E63	E55
11–12	E63	E54
12–13	E63	E54
FIFTY–FOUR FOOT SPANS		
2–3	E69	E63
4–5	E69	E67
6–7	E69	E67
8–9	E69	E64
10–11	E69	E66
12–13	E69	E67
14–15	E69	E67
COLUMNS		
BENT 1	E93	E138
BENT 2	E98	E151
BENT 3–10	E98	E194
BENT 11–12	E106	E106
BENT 11–12 (NEW)	E142	E142
BENT 13–14	E92	E171

possibly could be subject to fatigue, whereas the 90-ft span should not be affected by fatigue.

Strengthening Schemes and Recommendations

Some of the criteria used in formulating repair and strengthening schemes follow:

- To the maximum extent possible, train operations are to be maintained without interruption.
- Strengthen members to rate as high as or higher than their as-built condition.
- Design new members according to the allowable stresses of present design standards.
- High-strength bolting is preferred to welding for field connections, especially in tension zones.
- Cost-saving approaches consistent with good design are to be pursued.

Three strengthening schemes were investigated:

Scheme 1: Adding Plates Below Existing Flanges This scheme requires welding plates to the girder between the flanges. Plates are added top and bottom for the 30-ft span, but bottom only for the 54-ft spans. Strengthening would bring the overall rating to E-65.

Scheme 2: Prestressing This scheme entails applying a prestressing force along the bottom flanges of the girder. A single high-strength bar attached below the bottom flange and stressed to approximately 100 ksi would be sufficient to raise the ratings of the 30- and 54-ft spans to their as-built level. Recently, this technique has been used on highway bridges in Iowa. The prestressing force can increase girder capacity; however, it does not reduce the stress range, which is regarded as an important parameter for susceptibility to fatigue cracking. The prestressing force does not significantly increase the section modulus of the girder. The overall strengthened rating using this methodology would be E-70.

Scheme 3: Span Replacement This scheme entails replacing spans one at a time during weekends, when the trains terminate at stations located east (Manhasset Station) and west (Great Neck Station) of the viaduct. Commuters would need to be bused.

New girder spans would be designed to carry E-80 loading and would rate in the E-110 range. They can be designed to require no maintenance for many years by using special protection on the top horizontal surfaces. The Canadian Pacific Railroad has found zinc metallizing topped by a vinyl coating to be effective treatment for top flanges of girders and other horizontal surfaces that hold water and debris.

Galvanizing was considered for the new girders. However, because of size limitations of galvanizing tanks, it was not practical to galvanize the 54- and 90-ft girders.

Because of the high cost of field repairs and the limited strength gains achieved in Schemes 1 and 2, the replacement of complete spans can be financially competitive, providing many long-term benefits, if it can be done without undue interruption of railroad traffic.

Cost Considerations and Ramifications

Preliminary construction cost estimates prepared in 1983 follow:

Scheme	*Cost ($)*
1	2,184,000
2	1,234,000
3	1,480,000

Scheme 1 is uneconomical. Scheme 2, utilizing the prestressing approach, is a relatively simple and economical technique that is the least costly of the three alternatives. However, Scheme 3, entailing span replacement, is within a reasonable range of the cost of the prestressing scheme, considering that a much higher rating is obtained by this scheme. In addition, it can provide a maintenance-free superstructure for many years.

Scheme 3 was the recommended scheme, and design documents were prepared and contracts awarded for construction in late 1987.

English Kills Drawbridge (fixed)

Description

The English Kills Drawbridge (presently fixed) carries single-track freight service over English Kills on the Bushwick Branch in Brooklyn, New York. The existing steel bobtail swing bridge was designed as a temporary bridge in 1888 and was rehabilitated between 1907 and 1927. The bridge is presently fixed in place; however, closed-position clearances are 46 ft horizontally and 9 ft (low tide) and 4 ft (high tide).

The support structure comprises a stone masonry pivot pier and two stone masonry rest piers, all assumed to be supported on timber cribbing and timber piles.

The superstructure has two deck plate girders that are 97-ft-long longitudinal steel girders, with diaphragms and top and bottom lateral bracing, that provide two unequal spans of 67 ft 6 in. across the channel and 29 ft 6 in. for the bobtail.

Inspection Findings

An in-depth inspection was performed in August 1982. Some of the major findings follow:

- Stone masonry rest piers and pivot piers were in fair to poor condition.
- The two built-up longitudinal girders were in poor condition. Webs of both girders were heavily deteriorated with $^3/_{16}$- to $^1/_4$-in. losses and many small holes in the web of the south girder. The majority of severe losses to the top flange occurred under railroad ties.

Structural Analysis and Rating

A structural analysis of the bridge was conducted to determine the operating rating of the structure using AREA Manual procedures for both as-built and present conditions. The following allowable stresses were used: steel, medium open hearth; yield stress = 30 ksi; tension = 0.8; yield stress = 24 ksi; and shear = 18 ksi.

In addition to being rated for Cooper E loading, the longitudinal girders were analyzed using the estimated dead load of an LIRR freight train with diesel engine impact. Table 4 gives the results of the rating analysis. Substructure plans were not available, so the substructure was not rated.

The results of the structural analysis and evaluation of the load-carrying capacities of the main structural components of the bobtail swing indicated that the span is only marginally adequate to carry Cooper E-36 loading. This is satisfactory for the L-2 type of freight train the LIRR presently uses. However, the bridge does not meet the current AREA Cooper E-80 loading. The unknown history of loading and repair by welding with considerable variations in loading conditions makes the girders especially susceptible to fatigue-related problems and fracture mechanisms. Therefore the reliability of the calculated capacity is additionally suspect.

The bridge was again inspected in depth and rated in 1985 and 1987. The findings and the loading rating follow:

- The bridge, as a whole, was found to be in a state of deterioration similar to that revealed by the 1982 inspection.
- The measured loss of section to the critical point of the main girders was $^1/_4$ in. at the top cover plate, compared with the $^3/_{16}$-in. loss recorded in 1982.
- In calculating the rating of the bridge for live load capacity, this additional deterioration was taken into account but did not change the computed rating.

Cost Considerations and Ramifications

Studies were conducted for different types of bridge structures. Preliminary construction cost estimates ranged from $250,000 for a timber trestle to $1.27 million for a steel plate girder bridge.

At present, the LIRR provides freight service to one customer over this bridge. The bridge is presently being inspected more frequently (three to four times per year). The decision on the bridge is pending, subject to availability of funds and justification of proposed expenditures.

Reynolds Channel Trestle

Description

The Reynolds Channel Trestle is a single-track open-deck timber structure located over Reynolds Channel connecting Island Park to Long Beach, as shown in Figure 4.

The trestle was originally built around the turn of the century as a wooden swing bridge in the location of the current bridge. That wooden bridge was replaced by the present steel swing bridge in 1927. The trestle is 1,230 ft long not counting the movable bridge portion, which is 65 ft long. The trestle, in addition to normal maintenance, received extensive rehabilitation, particularly of its superstructure, during the 1970s (*4*).

The trestle and bridge provide access to the Long Beach Station, the Long Beach Branch's terminal station. A total of 88 trains consisting of 6 to 10 cars traverse the trestle each weekday.

The bents typically consist of five to nine wooden piles with a pile cap. The piles are randomly spaced timbers 12 to 14 in. in diameter.

The tops of the piles are connected to a 14-in.-square by 14-ft-long timber cap by steel spikes (drift pin) driven through the bent cap and into each pile (one each). The trestle has 112 bents. All of the timbers used in the structure have been treated with a preservative.

Four timber stringers (10 × 14 in.) are paired under each track rail and rest directly on the bents. Figure 5 shows a cross section of timber trestle.

Findings of Inspection and Rating

Findings In 1983 an in-depth inspection was conducted of the pile foundations, bents, and superstructure. Material samples from both the substructure and the superstructure were taken for laboratory testing. Some of the major deficiencies observed follow:

1. The most serious deterioration of the trestle structure was observed in the piles. During underwater inspection of piles, including probing, dry rot was observed in almost one-half of all piles. This deterioration is generally confined to the tidal zone (+2.1 to −1.8 ft above and below Mean Sea Level); the daily cycle of immersion and drying promotes the incidence of dry rot. Fortunately, the piles incapable of supporting a full or even a partial load were randomly distributed throughout the structure.
2. The pile caps are subjected to partial submersion on a daily basis.
3. Portions of the trestle appear to have sustained surface damage from fire.
4. Rotting, deterioration, and absence of lateral bracing were prevalent. A total of 152 of the possible 224 cross braces are missing, loose, split, or rotted.
5. The prevalence of dry-rotted piles suggests that the cut ends of these piles were either not treated or inadequately treated.
6. Numerous instances of out-of-alignment failed piles missing a "V" section were noted. No significant instances of marine borer damage were observed.

TABLE 4 RATING TABLE

SHEAR			A	B	C	D	
LONGITUDINAL GIRDER (ASSUMED SIMPLE SUPPORT)	ALLOWABLE STRESS (KSI)	AREA $(IN)^2$	ALLOWABLE SHEAR (K)	DEAD LOAD SHEAR (K)	LIVE LOAD & IMPACT SHEAR COMPACITY (K)	COOPER E60 LIVE LOAD & IMPACT SHEAR (K)	COOPER E RATING
AS-BUILT							
66'-0" SPAN	18.0	13.5	243	18.2	225	235	57
24'-6" SPAN	18.0	22.4	403	6.7	396	131	60+
PRESENT							
66'-0" SPAN	18.0	13.5	243	18.2	225	235	57
24'-6" SPAN	18.0	11.2	202	6.7	195	131	60+

MOMENT			A	B	C	D	
LONGITUDINAL GIRDER	ALLOWABLE STRESS (KSI)	SECTION MODULUS $(IN)^3$	ALLOWABLE MOMENT (K-FT)	DEAD LOAD MOMENT (K-FT)	LIVE LOAD & IMPACT MOMENT CAPACITY (K-FT)	COOPER E60 LIVE LOAD & IMPACT MOMENT (K-FT)	COOPER E RATING
AS-BUILT							
66'-0" SPAN							
SIMPLE	24.0	1362	2724	300	2424	3436	42
CONTINUOUS	24.0	1362	2724	240	2484	2749	54
24'-6" SPAN							
SIMPLE	24.0	746	1492	42	1450	700	60+
CONTINUOUS	24.0	746	1492	34	1458	560	60+
PRESENT							
66'-0" SPAN							
SIMPLE	24.0	1200	2400	300	2100	3436	36
CONTINUOUS	24.0	1200	2400	240	2160	2749	47
24'-6" SPAN							
SIMPLE	24.0	370	740	42	698	700	59
CONTINUOUS	24.0	370	740	34	706	560	60+

MEMBER	ALLOWABLE STRESS (KSI)	AREA $(IN)^2$	ALLOWABLE SHEAR (K)	DEAD LOAD SHEAR (K)	LIVE LOAD & IMPACT SHEAR COMPACITY (K)	COOPER E60 LIVE LOAD & IMPACT SHEAR (K)	COOPER E RATING
AS-BUILT							
PIVOT GIRDER	18.0	42.0	756	25	731	275	60+
PRESENT							
PIVOT GIRDER	18.0	21.0	378	25	353	275	60+

MOMENT			A	B	C	D	
MEMBER	ALLOWABLE STRESS (KSI)	SECTION MODULUS $(IN)^3$	ALLOWABLE MOMENT (K-FT)	DEAD LOAD MOMENT (K-FT)	LIVE LOAD & IMPACT MOMENT CAPACITY (K-FT)	COOPER E60 LIVE LOAD & IMPACT MOMENT (K-FT)	COOPER E RATING
AS-BUILT							
PIVOT GIRDER	24.0	1012	2024	62.5	1962	688	60+
PRESENT							
PIVOT GIRDER	24.0	506	1012	62.5	950	688	60+

COMBINED MOMENT & SHEAR			A	B	C	D	
MEMBER	AREA $(IN)^2$	SECTION MODULUS $(IN)^3$	ALLOWABLE DIAGONAL TENSION STRESS (KSI)	DEAD LOAD DIAGONAL TENSION STRESS (KSI)	LIVE LOAD & IMPACT DIAGONAL TENSION STRESS CAPACITY (KSI)	COOPER E60 LIVE LOAD & IMPACT DIAGONAL TENSION STRESS (KSI)	COOPER E RATING
AS-BUILT							
PIVOT GIRDER	42.0	1012	24.0	1.1	22.9	11.8	60+
PRESENT							
PIVOT GIRDER	21.0	506	24.0	2.2	21.8	23.6	55

$$E \text{ RATING} = \frac{(A-B) \times 60}{D} = \frac{(C) \times 60}{D}$$

"PRESENT" MEMBERS INCLUDE SECTION LOSSES AND ASSUME RIVETS PROVIDE INTEGRITY OF SECTION. RELIABILITY OF SECTION INTEGRITY IS SUSPECT; PARTICULARLY IN AREA ADJACENT TO COUNTERWEIGHT.

FIGURE 4 Reynolds Channel timber trestle.

Load Rating The purpose of this analysis was twofold. First, it was intended to provide an indication of the structural strength of the trestle as it existed. Second, it was to determine the structural strength the trestle could possess if fully rehabilitated.

During this analysis, it was found that the bents could not support the projected longitudinal force acting on the trestle. It was therefore necessary to conduct the analysis by factoring in the rails' transference of that longitudinal force to the banks at each end of the trestle. AREA permits the use of this assumption for the design of piles by reason of the continuity of the rails from embankment to embankment, except in the section of swing bridge with the subsequent frictional resistance of the rails and ties over the embankments. The embankments are acting to stabilize the trestle longitudinally. The Cooper E ratings for the five selected bents follow:

Bent No.	*Piles*	*Cap*	*Bracing*
S-27	E-17	E-25	3-19
S-76	E-15	E-21	L/D allowable
N-4	E-22	E-25	E-24
N-6	E-0	E-0	Missing
Baseline	E-12	E-17	

The pile cap capacity of the baseline bent was found to be low because most of the vertical loads apply to the center piles and, consequently, are not shared equally among all of the piles.

The following ratings of the remaining bents were made by comparing baseline bent results with the field inspection report:

Cooper Rating	*No. of Bents*
E-22 to 25	23
E-17 to 22	34
E-15 to 17	23
E-12 to 15	15
Less than E-12	7
	102

The current loading of the trestle with LIRR electric trains (M-3) is equivalent to E-17 Cooper loading. The characteristics of wood structures and the basically sound

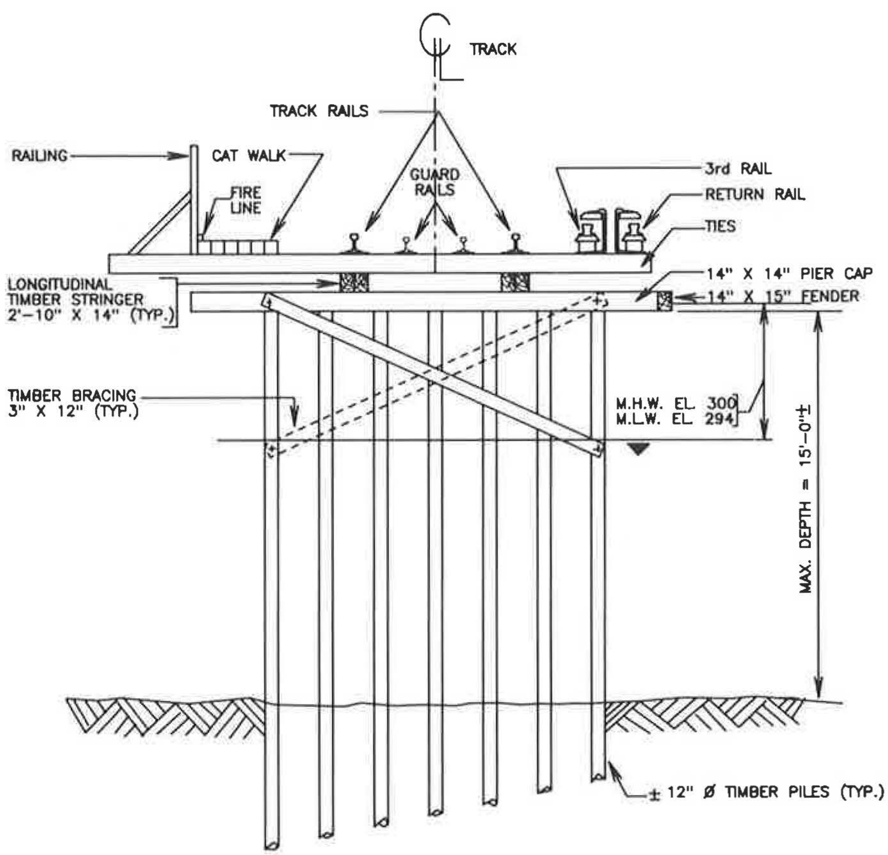

FIGURE 5 Cross section of timber trestle.

condition of the superstructure, coupled with the relatively even distribution of the bents rated below E-17, have apparently permitted the trestle to continue to function.

The stringer analysis was completed using the assumption that the stringers have partial continuity over the support. Given this assumption, based on field observation, the stringers can support approximately Cooper E-33 loading.

In summary, the timber trestle superstructure, which was rehabilitated during 1971–1980, was found to be in good shape.

However, in the vicinity of those bents rated below Cooper E-12, there were signs of structural damage to the stringers and the pile caps. In these instances, both needed to be replaced with new members. The substructure needed extensive rehabilitation to bring it up to a rating for a Cooper E-25 loading. This involved a majority of the bents. However, the absence of foundation data dictated caution in relying on the ratings obtained. Therefore, even after the trestle was rehabilitated, it was recommended by the consultant that the LIRR not use this trestle for loads greater than E-20 loading. The trestle was restricted to light electric trains and speeds not to exceed 5 mph.

Rehabilitation Scheme

The field investigation, test results, and design analysis indicated that the substructure required extensive rehabilitation. The rehabilitation scheme entailed driving new piles and replacing existing pile caps and stringers with new members. It was estimated that this scheme would cost $550,000 in 1984 dollars.

Project (management) Decision

Based on further management review, the decision was made to allocate funding from the LIRR 10-year, $2.0 billion Capital Improvement Program for a replacement bridge. The design was to be based on current AREA Cooper E-80 loading with prestressed concrete piles, prestressed concrete beams, precast deck, and direct fixation track. The engineer's construction cost estimate for a new concrete bridge was $11 million. However, the lowest responsive bid received was $14 million. After review and analysis of the bids received, it was concluded that the relatively high bids resulted from

- Reduced market competition as a result of the abundance of bridge projects in the metropolitan area,
- Unique design features of the bridge, and
- Tight project completion time and Coast Guard restrictions on channel opening and closing during certain timeframes.

It was decided that, with funding available, the project must proceed. The new bridge is under construction in 1988.

BRIDGE DATA BASE AND RATING PROGRAM

Background Information

One of the goals of the Civil Engineering Department is to automate various tasks that have been heretofore performed manually. The explosive growth of low-cost, high-powered microcomputers (*5, 6*) presented an opportunity and challenge to computerize bridge information that existed in various Engineering Department documents (Bridge and Building Record Book, valuation maps, engineering drawings, New York State Bridge Inspection Reports, etc.).

Bridge Data Base Program

In early 1986, the Civil Engineering Department obtained a number of IBM personal computers (PCs). Available IBM software, Data Ease, and dBASE III programs were used to formulate an in-house program for bridge data base information.

The salient features of the program include the following items (descriptors), shown in Figure 6, that provides various types of information:

1. Identification, which includes information on railroad branch, LIRR bridge identification number, New York State bridge identification number (if applicable and available), and feature crossed (highway, roadway, waterway).
2. Structural data, which include year built, date of original design drawings, and date of as-built drawings, depending on the information available.
3. Type of structure, which includes structural configuration (e.g., through plate girder, prestressed concrete, open deck/solid deck). In addition, number of spans, number of tracks, and length of spans between abutments are given.
4. Condition rating: The New York State standard Bridge Inspection and Condition Report is used to conduct in-depth inspection, photograph, document deficiencies, and record the condition rating of a structure on a scale from 0 (bridge condition beyond repair, danger of immediate collapse) to 9, which is new condition. Figure 7 is a typical inspection report form.
5. Load rating and capacity: Based on the AREA *Manual of Railway Engineering*, calculations are performed and various components are rated and listed in terms of railway Cooper E loading, with the weakest member governing the capacity of the bridge structure.
6. Structural or safety flag (if applicable).
7. Maintenance and repair history: If available, includes date and type of repairs performed and date of last painting.

Bridge Load Rating Program

A majority of LIRR bridges have through-plate girders, deck-plate girders, or I-beams. These three are used in 349 bridges of the total 396, which translates to 88 percent of all bridges in the system.

Most of the plate-girder bridges are built-up sections with riveted connections. In performing load rating analysis, one of the tedious, time-consuming, and costly tasks is calculation of the moment of inertia of a section. An in-house program was written in BASICA 2.0 and compiled using IBM BASIC Compiler Version 1.0. The program consists of two modules, Moiner.exe and Sectdraw.exe. Minimum hardware requirements are 640K RAM, two floppy drives, a graphics monitor,

LIRR BRIDGE NO.	COUNTY	STRU TYPE	BIN (NYS) (DOT)	NO. OF SPANS	LENGTH OF SPANS	LOCATION	CONDI. RATING	RATING DATE	RATED BY	LOAD RATI	CLEARANCE	PAINTED (T/F)	PAINT DATE	REPAIRS	REHABILITATION	FLAG CONDI	COMMENTS
BRANCH NAME : PORT JEFFERSON																	
65-0-257	NASSAU	TPG	7036780	3		BARCLAY ST.- HICKSVILLE	6	02/22/85	NYS/CONSUL			F	/ /				PART OF VIADUCT
65-0-259	NASSAU	TPG	70950	1		BETHPAGE RD- HICKSVILLE		/ /				F	/ /				PART OF VIADUCT
65-0-280	NASSAU	HTPG	7060840	1	56'	JERICHO TPKE.- SYOSSET	5	09/18/85	NYS/CONSUL			F	/ /				
65-0-310	NASSAU	IB		1	21'	WHITNEY LANE-COLD SPRING HARBR		/ /				F	/ /				I-BEAM CONCRETE ENCASED
65-0-313	SUFFOLK	TPG	70952	2		WOODBURY RD-COLD SPRING HARBOR		/ /				F	/ /				
65-0-323	SUFFOLK	TPG	70953	2		W.ROUGUES PATH-COLD SPRING HAR		/ /				F	/ /				
65-0-347	SUFFOLK	HTPG	7037000	1	71'	NEW YORK AVE - HUNTINGTON	6	08/01/86	NYS/CONSUL		12'	F	/ /				
66-0-384	SUFFOLK	IB	70954	1	28'	STONY HOLLOW RD- DIX HILLS		/ /				F	/ /				
66-0-416	SUFFOLK	DPG	70955	5	201'	CHEESE HOLLOW RD-E. NORTHPORT		/ /				F	/ /				
66-0-424	SUFFOLK	IB	7059030	2	105'	SUNKEN MEADOW PKWY-KINGS PARK	6	07/15/86	NYS/CONSUL			F	/ /				
66-0-463	SUFFOLK	DPG	7060810	12	420'	JERICHO TPKE- SMITHTOWN	6	06/16/86	NYS/CONSUL			F	/ /		Rebuilt In 1937		
66-0-470	SUFFOLK	HTPG	7060830	1	85'	MAIN ST.- SMITHTOWN	6	09/17/86	NYS/CONSUL			F	/ /				
66-0-490	SUFFOLK	TPG	01833	1		ST.JAMES RD(RT.25A)- ST. JAMES		/ /				F	/ /				
66-0-520	SUFFOLK	DPG	70956	5	181'	LONGHILL RD.- STONY BROOK		/ /				F	/ /				
66-0-537	SUFFOLK	TPG	70957	2	181'	NICHOLLS RD.-STONY BROOK		/ /				F	/ /				
66-0-541	SUFFOLK	IB	70958	1	27'	BENNETT RD.- STONY BROOK		/ /				F	/ /				
66-0-549	SUFFOLK	IB	70959	1	27'	DEPOT RD.- SETAUKET		/ /				F	/ /				
66-0-552	SUFFOLK	DPG	70960	5	171'	OLD TOWN RD.- SETAUKET		/ /				F	/ /				
66-0-565	SUFFOLK	CBOA	70961			DARK HOLLOW RD-PORT JEFFERSON		/ /				F	/ /				
66-B-466	SUFFOLK	IB	2261130	3	94'	EDGEWOOD AVE/BLYDENBURG RD.	5	07/09/86	NYS/CONSUL			F	/ /	REPAIR 3-7-71			
66-B-414	SUFFOLK	DPG	3364550	3	137'	PULASKI ROAD - NORTH PORT	4	07/09/86	NYS/CONSUL		17'	F	/ /				SAFTY CONCRETE PAVING
66-B-568	SUFFOLK	TS	2261340	5	80'	SHEEP PASTURE RD - SETAUKET	4	07/10/86	NYS/CONSUL		21'	F	/ /				REMOVAL OF SAFTY FLAG

TOTAL NO. OF BRIDGES	HALF THRU PLATE GIRDER (HTPG)	DECK PLATE GIRDER (DPG)	THRU PLATE GIRDER (TPG)	I - BEAM (IB)	CONC. BEAM OR ARCH (CBOA)	TIMBER STRUC. (TS)	PRESTRESSED CONC. BEAM (PB)
22	3	5	6	6	1	1	0

PORT JEFFERSON Branch Report Is Complete ....

FIGURE 6 Bridge data base report.

TP 360d (7/81)

NEW YORK STATE DEPARTMENT OF TRANSPORTATION
BRIDGE INSPECTION AND CONDITION REPORT

SHEET 3 OF 14

REGION (1–2): [ ] [ ] COUNTY

BRIDGE IDENTIFICATION NUMBER (3–9): 7 7 0 9 7 3 0

FEATURE CARRIED: LIRR-Valley St. East FEATURE CROSSED: Rockaway Ave.

INSPECTED BY: LDS, MP, RG, JB TITLE: Designer-Structures Field Egr.

DATE: MO. 03 (13–14) DAY 25 (15–16) YEAR 87 (17–18)

	SPAN NO. (10)	(11)	(12)	DECK ELEMENTS: WEARING SURFACE (19)	MONO DECK SURFACE (20)	CURBS (21)	SIDEWALKS & FACIAS (22)	RAILINGS & PARAPETS (23)	RAILING PAINT (24)	SCUPPERS (25)	GRATINGS (26)	MEDIAN (27)	RECOMMENDATION DECK (28)	SUPERSTRUCTURE: STRUCTURAL PRIMARY MEMBER (29)	SECONDARY MEMBER (30)	PAINT (31)	JOINTS (32)	RECOMMENDATION (33)	(34)
Exp. jt.	0	1	1	8	8	8	5	5	5	8	8	8	5	5	5	4	5	3	4
	0	1	2	8	8	8	5	5	5	8	8	8	5	5	5	5	5	8	5
	0	1	3	8	8	8	5	5	5	8	8	8	5	5	5	5	5	8	5
	0	1	4	8	8	8	4	5	5	8	8	8	5	5	5	5	5	8	5
Exp. jt.	0	1	5	8	8	8	3	4	5	8	8	8	4	5	5	5	5	8	5
	0	1	6	8	8	8	4	5	5	8	8	8	8	5	5	5	5	3	4
	0	1	7	8	8	8	4	5	5	8	8	8	4	5	5	5	5	8	5
	0	1	8	8	8	8	5	5	5	8	8	8	5	5	5	5	5	8	5
Exp. jt.	0	1	9	8	8	8	3	4	5	8	8	8	4	4	5	5	5	8	4
	0	2	0	8	8	8	4	4	5	8	8	8	4	2	5	4	5	2	3

SPAN NO.	PIER: BRG. ANCHOR BOLT, PADS (35)	PEDESTALS (36)	TOP OF PIER CAP OR BEAM (37)	STEM SOLID PIER (38)	CAP BEAM (39)	PIER COLUMNS (40)	FOOTINGS (41)	EROSION OR SCOUR (42)	PILES (43)	RECOMMENDATION (44)	UTILITIES: LIGHTING STANDARDS (45)	LIGHTING FIXTURES (46)	SIGN STRUCTURE (47)	UTILITIES (48)	UTILITIES SUPPORT (49)	RECOMMENDATION (50)	CONTRACTOR CODE (116)	(117)	REC. CODE (118)	(119)	TX CD (120)
011	2	8	5	8	5	5	9	8	8	2	5	5	8	3	5	4			1	6	3
012	8	8	5	8	5	5	9	8	8	5	5	8	8	5	5	8			1	6	3
013	8	8	5	8	5	5	9	8	8	5	5	5	8	5	5	5			1	6	3
014	8	8	5	8	5	5	9	8	8	5	5	8	8	5	5	5			1	6	3
015	8	8	5	8	5	5	9	8	8	5	5	5	8	5	5	5			1	6	3
016	8	8	5	8	5	5	9	8	8	5	5	8	8	5	5	5			1	6	3
017	8	8	5	8	5	5	9	8	8	5	5	5	8	5	5	5			1	6	3
018	8	8	5	8	5	5	9	8	8	5	5	8	8	5	5	5			1	6	3
019	8	8	5	8	5	5	9	8	8	5	5	5	8	5	5	5			1	6	3
020	3	8	5	8	5	5	9	8	8	5	5	8	8	5	5	5			1	6	3

REMARKS: 29 Hole in slab adjacent to expansion joint. (Span 20) Ballast falling through.

FIGURE 7 Typical bridge inspection form.

an IBM PC–compatible computer, and a printer for hard copies.

Additional programs are being formulated to include computation of moments and shears for moving loads and further automation of stresses and final rating calculations. Figure 8 shows the LIRR bridge management process.

Data Base Benefits

Some of the benefits that can be derived from full implementation of the data base and load rating program include

- Reduced engineering man-hour costs and improved efficiency gained by automating various tasks previously conducted manually;
- Availability of instant information and thus capability to review lists of all deficient bridges on various branches and focus on bridges with inspection condition rating of 3 or less;
- Continuously update information (data base) from in-house inspection reports, consultants' inspection reports, or New York State inspection reports;
- Revise and periodically update bridge load rating and capacity when such calculations are formulated;
- Review lists of any structural or safety flags and take appropriate corrective action;
- Exchange with and provide to other agencies timely information when requested;
- Respond expeditiously to a freight shipment by analyzing the stress condition of the bridge or bridges on the branch using the freight load and axle spacing and comparing them with known capacity of the bridge structure or structures in the data base;
- Effective management tool to priority rank deficient bridges by conducting more frequent inspections; reducing train speeds (if necessary); restricting train loads (if required); and budgeting monies for bridge strengthening, rehabilitation, or replacement.

CONCLUSIONS

The LIRR bridge infrastructure of 396 railroad bridges presents a challenge in terms of priority ranking resources and managing the system so that the structures are safe, reliable, and maintain their load-carrying capacity. The $2.0 billion, 10-year (1982–1991) Capital Improvement Program has, for the first time, provided an opportunity to focus on existing deficient bridges, trestles, and viaducts that have either reached or are nearing their useful service lives. Various bridge structures on different corridors are in the design phase or have been inspected, studied, designed, and contracted for strengthening,

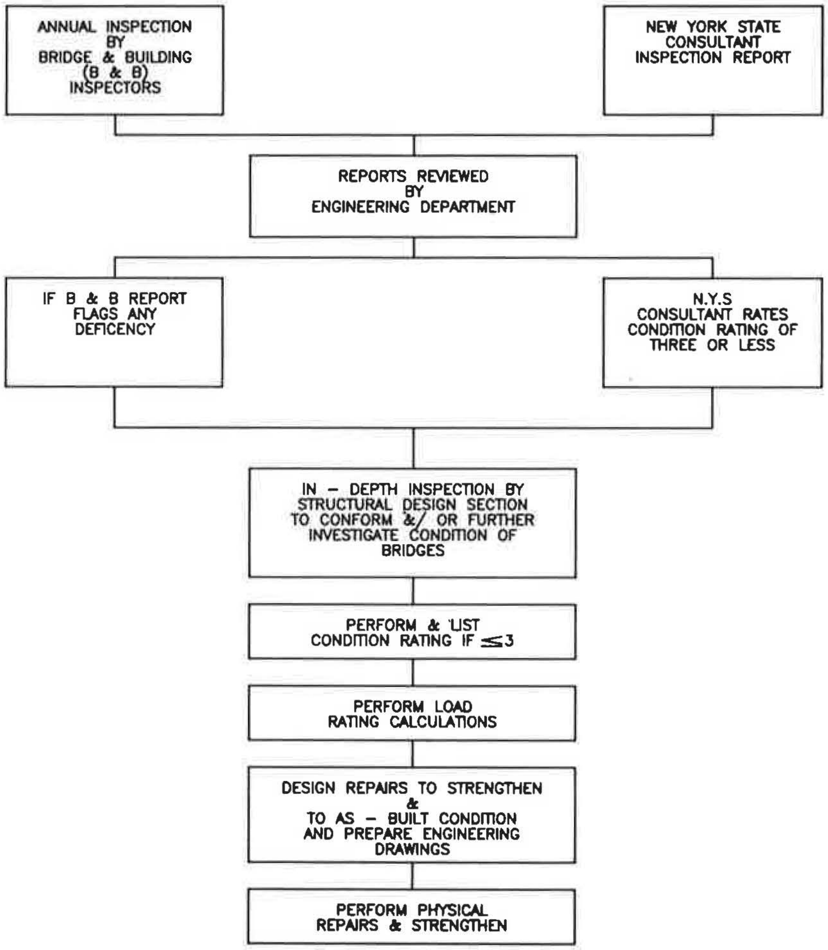

FIGURE 8 LIRR bridge management process.

rehabilitation, or replacement with the assistance of either outside consultants or in-house engineering personnel.

The bridge data base and rating program will provide an effective management tool to priority rank and manage deficient bridges.

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