QUEENSBORO BRIDGE: SEISMIC CONDITION ASSESSMENT AND RETROFIT CONCEPTS

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ABSTRACT

The Queensboro Bridge connecting the boroughs of Manhattan and Queens is a critical transportation link in New York City. It is a two-level, five span truss bridge with extensive approaches and approach ramps. The total length of the Main Bridge spanning the East River is 3,725 ft. The structure, which opened to traffic in 1909 and is a registered U.S. landmark, carries a daily traffic of approximately 147,000 vehicles.

This major east coast bridge underwent an in-depth seismic condition assessment that combined the efforts of seismologists, geotechnical, and structural engineers. The study serves as a prototype for the seismic condition assessment of other bridges built at the beginning of the century or earlier. This paper reports on the issues of superstructure evaluation, proposed retrofit measures, and other findings regarding the expected seismic response of a bridge that represents the design practice of another era.

KEYWORDS

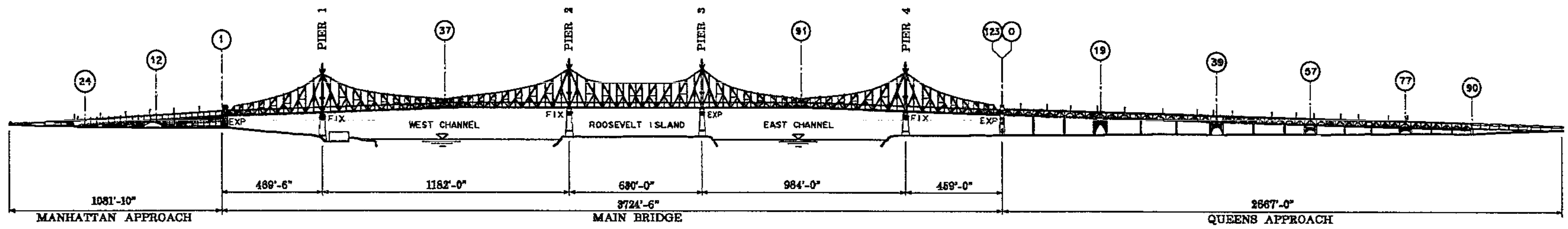
Queensboro Bridge; long span; seismic condition assessment; east coast seismicity; retrofit measures.

BRIDGE DESCRIPTION

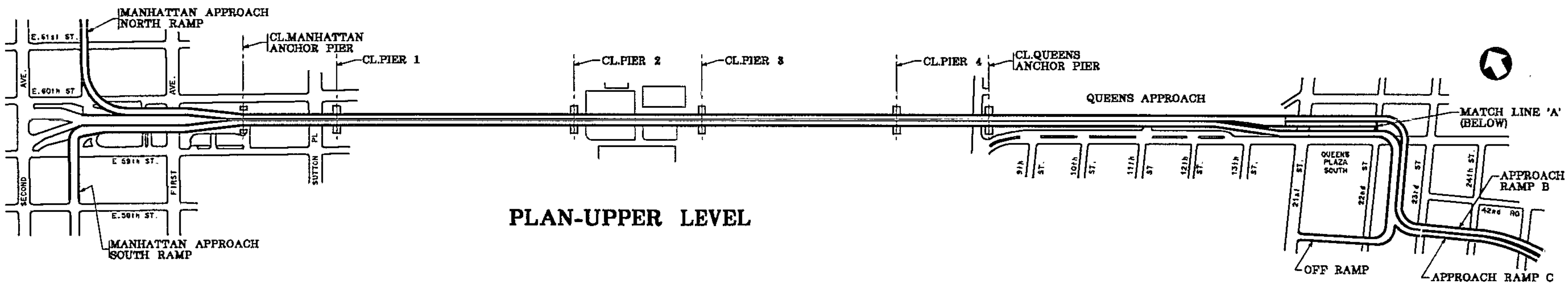
The Queensboro Bridge, Fig. 1, consists of four different structures with a variety of structural systems and forms resulting in distinct responses:

- *The Manhattan Approach*, a two-level stringer and multiple column bent structure with extensive masonry encasements.
- *The Main Bridge*, a two-level heavy long span truss structure with suspension bridge features supported on masonry piers.
- *The Queens Approach*, a two-level parallel chord truss structure supported on steel towers and bents.
- *The Approach Ramps*, a system of steel viaduct structures of one and two levels.

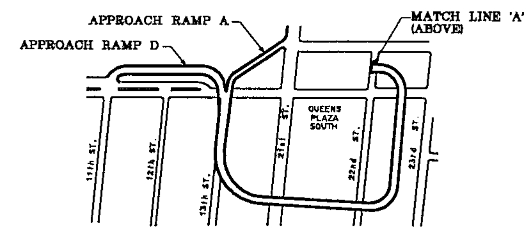
The Manhattan Approach, is a steel structure and has a total length of 1081 ft. The upper roadway is accessed by the north and south ramps. The lower roadway framing is enclosed by a facade of granite stones and terra cotta tiles forming masonry arches and infill walls. Underneath the lower roadway, the enclosed

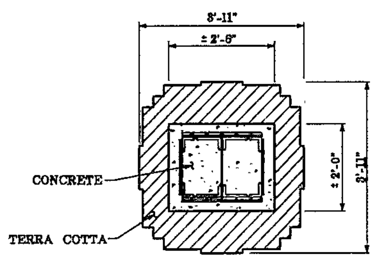


ELEVATION

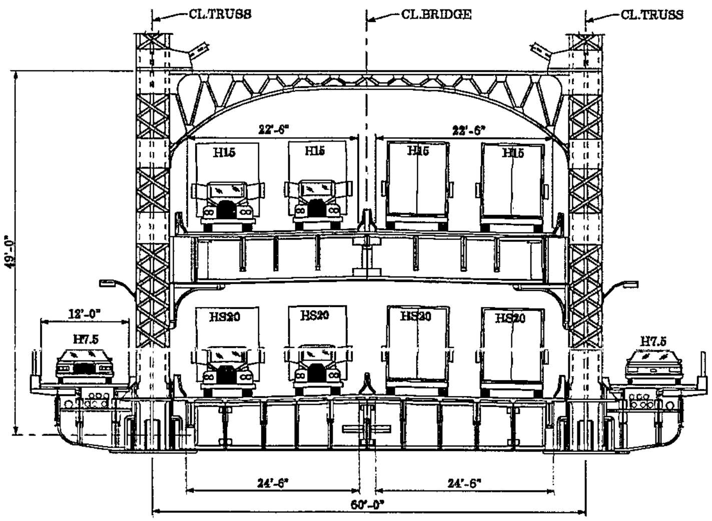


PLAN-UPPER LEVEL





TYPICAL COLUMN SECTION
MANHATTAN APPROACH



CROSS SECTION

STEINMAN	CITY OF NEW YORK DEPARTMENT OF TRANSPORTATION BUREAU OF BRIDGES	*SEISMIC INVESTIGATION OF QUEENSBORO BRIDGE*	GENERAL ELEVATION, PLAN AND CROSS SECTION	FIGURE 1

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space has a ceiling of historic tiled Guastavino shells, bearing the name of the engineer who invented the method of construction of shells with tiles. The columns of the lower level are encased in concrete and a masonry veneer, Fig. 1. They are supported on foundations consisting of individual tall unreinforced concrete pedestals bearing on hardpan or rock at various levels up to 25 ft. below the ground surface.

The Main Bridge was designed with suspension bridge features such as towers and trusses passing through the towers. It consists of the Manhattan, Queens, and the Roosevelt Island portions which are connected at the middle of the two channels of the East River with vertical links and transverse shear keys. The main tension members of the trusses are made of eyebars and the main compression members of riveted plates and lacing bars. The four towers, Fig. 2, are about 200 ft. tall, 60 ft. wide at the top where they are connected with the truss diagonals and top chords, and 95 ft. wide at the bottom.

At each end, the bridge is supported on links that allow longitudinal displacements. At Piers 1, 2, and 4 the bridge is supported on fixed bearings and at Pier 3 on sliding bearings. Transverse loads are resisted at all piers through shear keys. The piers consist of a 16 ft. thick spandrel beam from unreinforced concrete and steel I beams which is supported on two legs from unreinforced concrete or limestone. These massive structures have a granite block veneer and they are founded on rock.

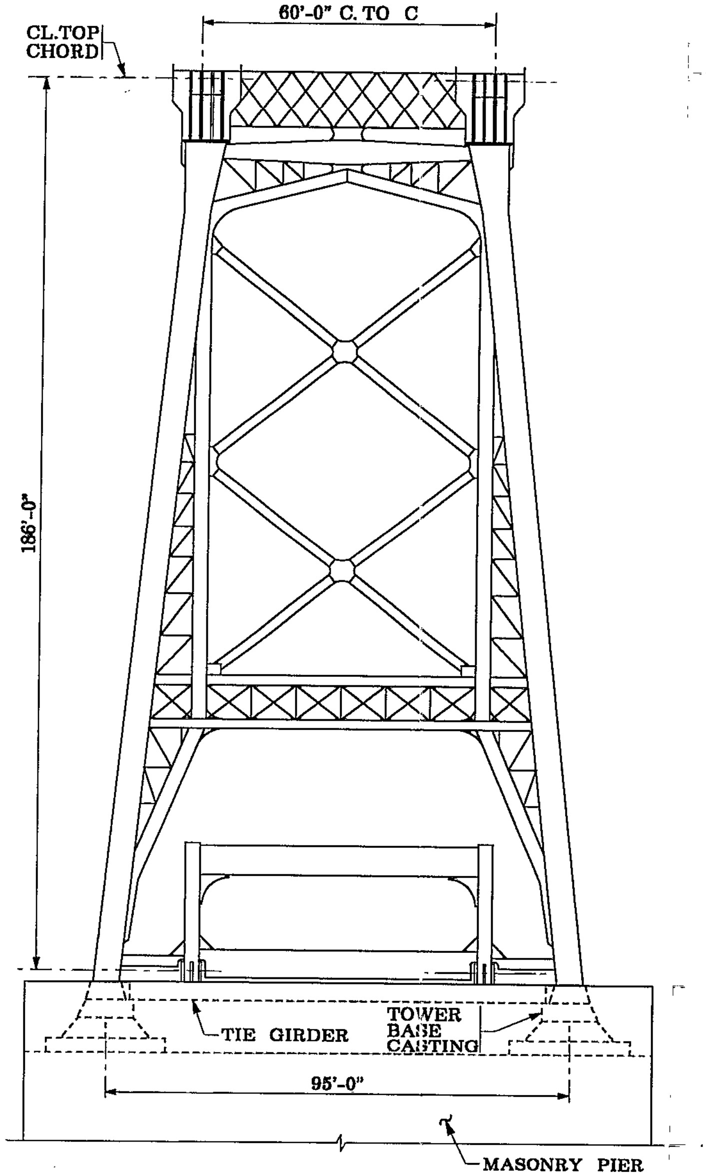

Fig. 2. Tower Elevation.

The Queens Approach is a 2,236 ft. through truss structure long consisting of simply supported and continuous spans varying in length from 97 ft. to approximately 166 ft. The superstructure is supported with bearings on steel towers and bents that include elaborate arch shaped truss articulations bracing the columns. All main members are riveted plates and angles connected with lacing bars. As in the Manhattan Approach, each column is founded on individual tall column-like unreinforced concrete pedestals bearing on rock up to 50 ft. below the ground surface. Thus, for many columns of the Queens Approach, the concrete part below ground is longer than the steel part above ground.

The Approach Ramps are an extensive system of steel viaducts connecting the upper roadways of the Queens and Manhattan Approaches with local streets. Their framing system consists of stringers and floorbeams supported on columns. The viaducts are mostly founded on concrete filled steel pipe piles or on H-piles.

GROUND MOTIONS

A seismic hazard analysis was performed by Lamont Doherty Earth Observatory, (NCEER, 1995), based on the seismicity of the Manhattan Prong and portions of the adjacent Newark Basin and Hudson Highlands.

Straight line fits to the observed seismicity were made using the Gutenberg-Richter frequency-magnitude relation

$$\log n = a - bM$$

where M is magnitude of earthquakes, n is the rate of events normalized to number of events per year per km^2, and a and b are parameters. This relation allows the representation of magnitude-distance combinations of events consistent with the established hazard levels as follows:

- *A low to moderate hazard level,* established as a seismic event with a 10% probability of exceedance in 50 years or approximately 500 years return period. This is similar to the hazard level of the AASHTO Specifications.
- *A high hazard level,* established as a seismic event with a 10% probability of exceedance in 250 years or approximately 2,500 years return period.

Magnitudes 5, 6, and 7 were used to establish envelope spectra for the above hazard levels and subsequently synthesize ground motions, (Horton, S. P., 1994). The rock level envelope spectra for the Queensboro Bridge site are shown in Fig. 3.

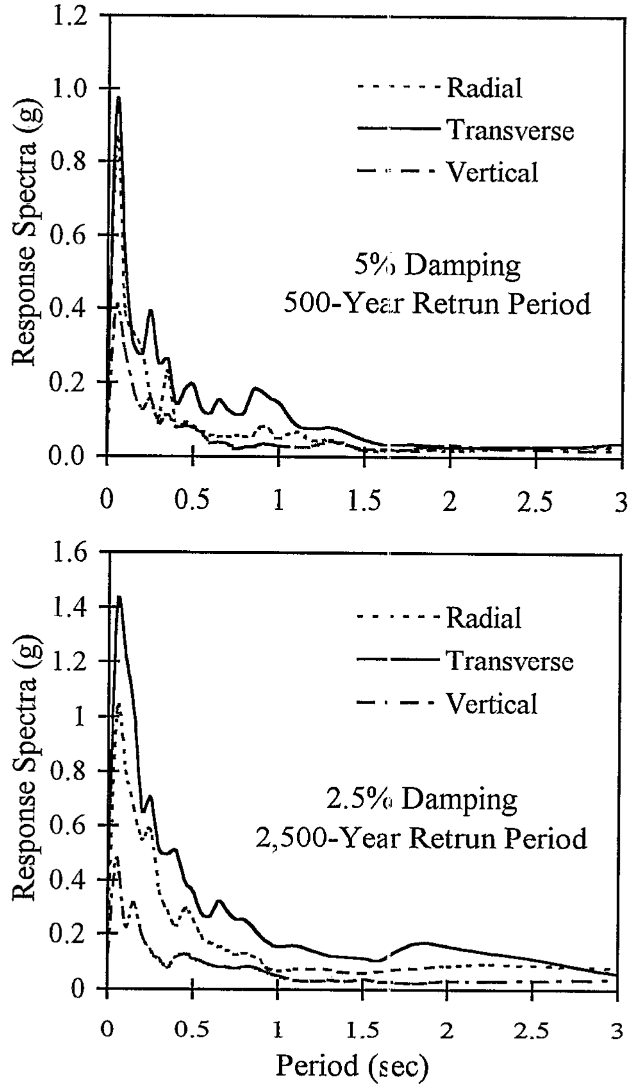

Fig. 3. Envelope Response Spectra

PERFORMANCE CRITERIA

The bridge performance goals were set as follows:

- For the low to moderate hazard level or Operational Basis Earthquake (OBE), the goal is for the bridge to remain functional after an earthquake. Specifically, the bridge shall be capable to carry normal traffic almost immediately after an earthquake, with minimal or no damage to structural elements.
- For the high hazard level or Structural Safety Earthquake (SSE), the goal is for the bridge to be capable to carry limited traffic within days and full traffic within months after an earthquake. Any damage shall be repairable without complete closure for extended periods.

The performance of structural components was determined on the basis of the capacity over demand ratio (C/D) for all potential modes of failure (FHWA, 1994). The lowest C/D ratio value of each component indicates the controlling mode of failure. This C/D ratio is defined as follows:

$$C/D = \frac{R_c - \Sigma Q_i}{Q_{EQ}}$$

where R_c is the nominal ultimate displacement or force capacity of a component for the mode of failure under consideration

ΣQ_i is the sum of displacement or force demand for loads other than earthquakes such as dead and live loads

Q_{EQ} is the earthquake displacement and force demand.

For a multi-axial stress state, the C/D ratio for a component's forces can be expressed as

$$C/D = \frac{1 - \Sigma Q_i / R_c}{Q_{EQ} / R_c} = \frac{1 - \Sigma SR_i}{SR_{EQ}}$$

where ΣSR_i is the sum of stress ratios, e.g. as per AASHTO Specifications, of force demands for other than earthquake loads to the capacity of the component

SR_{EQ} is the stress ratio for earthquake force or deformation demand to the capacity of the component.

Decisions on which components are to be retrofitted were based on the likely consequences from their damage, especially on the performance of entire structural systems with damaged components. In this regard, the structural members were classified as primary with no redundant load path, primary with redundant load path, and secondary.

ANALYSIS

The Queensboro Bridge seismic condition assessment was performed using the multi-mode response spectrum method with uniform support excitation. This procedure has its limitations, and a time history approach could have produced more accurate results. However, it is believed that in this phase of identification of vulnerabilities the response spectrum procedure was generally sufficient.

While most structural components of the Queensboro Bridge have adequate strength to resist the seismic demands, several components are vulnerable and can be expected to be damaged at low levels of seismic loads. This is typical behavior for bridges that were not designed for seismic loads. If early damage of such components would not result in structural failure, in most cases it would be the cause for considerable nonlinear response. Examples of vulnerable components at low levels of seismic forces are as follows:

- *Manhattan Approach.* The masonry elements, which provide significant stiffness to the steel components, but have limited strength in bending. The unreinforced concrete foundations of columns, which are up to 20 ft. long and were designed as compression members only.
- *Main Bridge.* The masonry piers, which have no reinforcement and resist lateral loads through their own weight and the weight of the superstructure.
- *Queens Approach.* The truss bearings, which have limited shear capacity. The anchor bolts of the column base anchorages, which could yield under the bending moment demands. The unreinforced concrete foundations of columns, which are similar to those of the Manhattan Approach but up to 45 ft. long.
- *Approach Ramps.* The floorbeam connections to columns, which are a part of the framing system resisting lateral loads. The anchor bolts of column base anchorages, which could yield as those of the Queens Approach.

In the analysis, early damage of key structural components was accounted for in the following manner:

1. If early damage could cause global loss of stability, the bridge was analyzed and evaluated assuming that retrofit measures were applied at the vulnerable components.
2. If early damage would not cause global loss of stability but induce strong nonlinear response, several iterations of analysis and evaluation were performed to determine a secant stiffness for final analysis.

Three large computer models were prepared for analysis, one each for the Main Bridge, Manhattan and Queens Approaches. Eighteen smaller size models were necessary for an equal number of portions of the Approach Ramps. Numerous variations of these models were used to perform dead load analyses, modal analyses, etc. Portions of the Queens Approach Ramps were not analyzed and the assessment of their condition was based on interpolation of the results of similar adjacent portions. The analyses were conducted with ground motions at the top of the foundations' level derived from procedures described in a companion paper at this conference entitled Seismic Investigation of the Queensboro Bridge, New York City: Geotechnical Aspects.

The computer models for the Main Bridge, and the Manhattan and Queens Approaches were verified using field measurements of ambient vibrations that provided mode shapes and frequencies of the structures. Table 1 compares the measured with the ten lowest computed frequencies for the Main Bridge using a computer model with fixed supports. The frequencies show excellent agreement. The field measurements also produced lower bound estimates of the expected damping values for each vibration mode.

Table 1. Computed vs. Measured Frequencies of Main Bridge

Mode No.	Comp. Freq. (Hz)	Meas. Freq. (Hz)	Perc. Diff. %	Mode Description
1	0.35	0.38	8	Transverse
2	0.45	0.46	3	Vertical
3	0.49	0.48	1	Transverse
4	0.54	0.56	3	Vertical
5	0.74			Local
6	0.81	0.79	2	Transverse
7	0.82	0.85	3	Vertical
8	0.86			Local
9	0.88	0.88	0	Transverse
10	0.91	0.91	0	Torsional

SEISMIC RESPONSE

The Manhattan Approach is a relatively stiff structure, particularly at its lower level, where the masonry components contribute significantly to the overall stiffness. This is shown in Table 2, where the frequencies of the Approach before and after a damaging earthquake are compared. The change in frequencies occurs due to the reduction in flexural rigidity at critical locations as shown in Fig. 4. The flexural rigidity is strongly affected by the size of bending moments, Fig. 5. It is observed that cracking of the masonry components occurs even at relatively low levels of seismic loads. After initiation of cracking, there is a considerable drop in flexural rigidity which is followed by a more gradual reduction as the bending moment increases further.

Table 2. Computed Frequencies of Manhattan Approach

Mode No.	No Masonry Cracking		With Masonry Cracking	
	Freq. (Hz)	Mode Description	Freq. (Hz)	Mode Description
1	2.57	Transverse	0.97	Longitudinal
2	2.69	Vertical	1.31	Longi. & Trans.
3	3.56	Transverse	1.38	Longi. & Trans.
4	3.70	Transverse	1.56	Transverse
5	3.96	Vertical	1.74	Transverse
6	5.15	Longitudinal	1.86	Longitudinal
7	5.29	Longitudinal	1.95	Longitudinal
8	5.71	Transverse	2.16	Transverse
9	6.57	Trans. & Longi.	2.69	Vertical
10	6.99	Verti. & Longi.	3.58	Transverse

Damage to the masonry components is not anticipated to cause a serviceability or a safety problem for the bridge or for traffic. However, the vulnerabilities of the fifty tall unreinforced concrete foundations may have a more serious effect on the structure due to an earthquake. The steel superstructure has only a limited number of vulnerabilities that are relatively easy to retrofit.

The Main Bridge is a flexible structure in the transverse and vertical directions as can be seen from the frequencies in Table 1. In the longitudinal direction, the bridge is stiffer and the seismic demands higher. Longitudinal forces acting on this 150,000 kips heavy superstructure are resisted by the fixed bearings at Piers 1, 2, and 4 and by the four towers. Since the towers are longitudinally flexible offering little resistance, the fixed bearings carry the majority of the forces. Thus, the bearings and the masonry piers of the Main Bridge are key components for the stability of the bridge. These components were found to be vulnerable due to longitudinal shear in the bearings and bending moment in the piers. A good part of the seismic bending moment acting on the piers is due to their own weight.

The response of the bridge to seismic loads is complex. One feature of this response is the interaction between superstructure and piers. As massive rigid structures, the piers tend to vibrate in frequencies that are much higher than the frequencies of the superstructure. The fundamental frequency of vibrations of Pier 1 as a stand-alone structure, for example, is 1.6 Hz, which is different from the frequency of longitudinal vibrations of the superstructure of about 0.58 Hz. It appears that the superstructure acts as a support of the piers in the longitudinal direction, reducing the seismic loads due to their weight and the weight of the superstructure. To accurately determine the seismic demands of the piers and bearings, it was necessary to consider a high number of modes to include those of the piers. A better approach is to perform a time history analysis with a sufficiently small step of integration.

The stiffness of the Queens Approach resembles that of the Manhattan Approach after cracking of the masonry components, Table 3. This response is affected by the varying depth of rock and type of underlying soils, which include a soft layer of peat. The effect of the soil layers in this area is shown in Fig. 6, which compares the AASHTO spectra type I and III with the site-specific spectra at the rock level and top of the foundations. Fig. 6 shows that the peaks of the spectral accelerations, which are a characteristic of the east coast spectra, are shifted towards a range of periods that coincides with important structural periods of the Queens Approach. The resulting resonance effects are responsible for large seismic loads on the Queens Approach.

In addition to retrofitting of the foundations as discussed for the Manhattan Approach, this portion of the bridge requires considerable

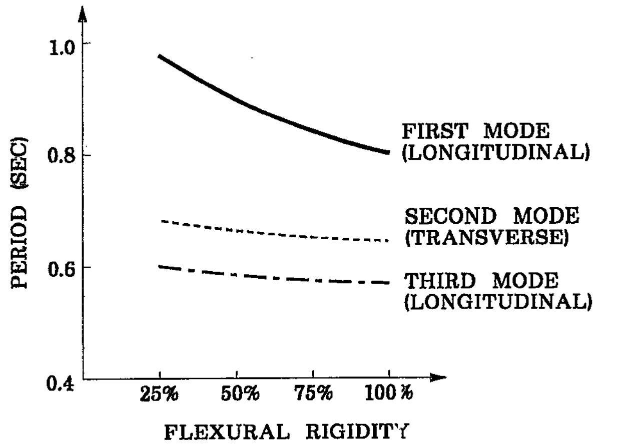

Fig. 4. Period of Vibrations vs Flexural Rigidity at Column Bases.

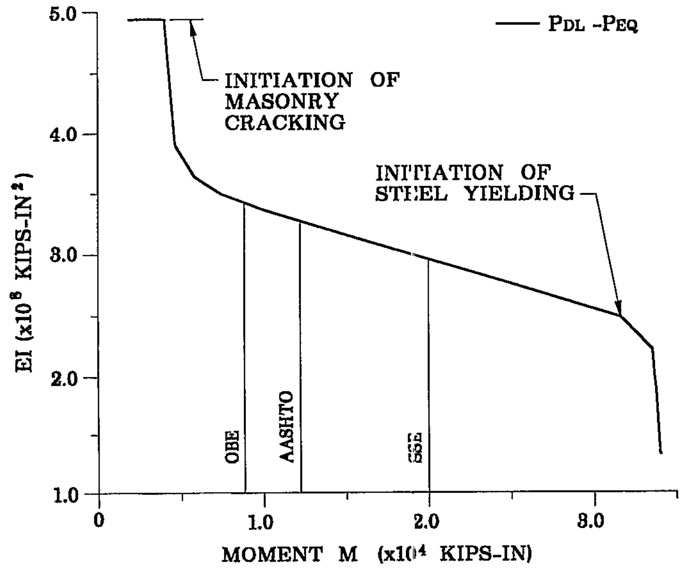

Fig. 5. Effect of Bending Moment on Flexural Rigidity of Columns.

Table 3. Lowest Frequencies for Eastern Half of Queens Approach

Mode No.	Freq. (Hz)	Period (Sec)	Mode Description
1	0.87	1.15	Longitudinal
2	1.13	0.89	Longitudinal
3	1.34	0.75	Longitudinal
1	1.02	0.98	Transverse
2	1.39	0.72	Transverse
3	1.51	0.66	Transverse
1	3.47	0.29	Vertical
2	3.89	0.26	Vertical
3	5.38	0.19	Vertical

retrofitting of the steel components. The work includes strengthening of the substructure towers and bents, reconfiguration of the support system with isolation bearings, and installation of shear transfer keys between adjacent portions of the superstructure. In lieu of isolation bearings, damping devices were considered and were found in preliminary studies to be beneficial for the response of the bridge (Constantinou, M., 1994).

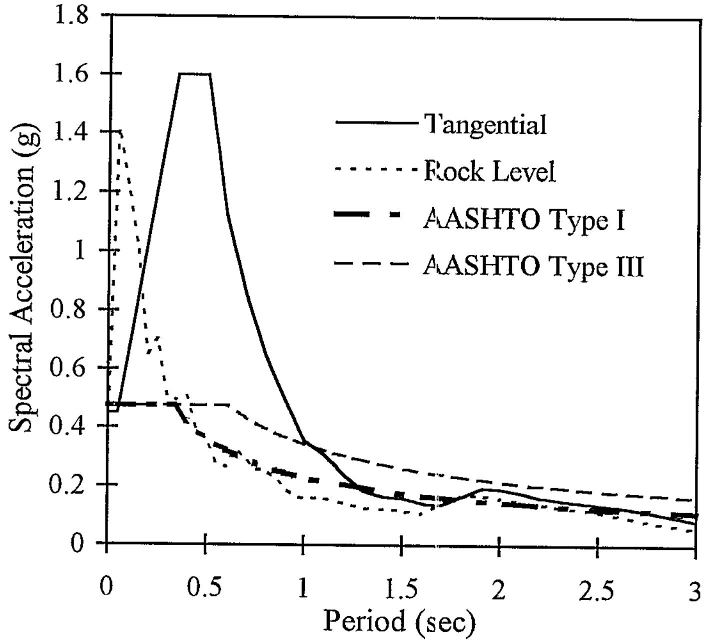

Fig. 6. AASHTO vs. Site Specific Spectra

The Queens Approach Ramps are more flexible in the longitudinal than the transverse direction. Judging from the design details of critical connections of the ramps, it is quite clear that the primary design loads were vertical. Most columns and their connections with the foundations, floorbeams, and stringers are vulnerable and cannot resist bending moments to achieve frame action that could provide stability against lateral loads. Even if the connections between columns and stringers are strengthened to resist bending moments, the longitudinal frame action would be of questionable effectiveness due to the flexibility of the stringers. Thus, longitudinal loads must be essentially resisted by cantilever action of the columns fixed at the foundations.

CONCLUSIONS

The vulnerabilities of the Queensboro Bridge were determined for the expected seismic loads in New York City and the desired level of performance of this critical transportation link. The response of the bridge is typical for older bridges that were not designed for seismic loads.

For the first time on a major bridge on the east coast, this seismic condition assessment integrated the available knowledge on seismic, geotechnical and structural issues. During the course of this work, it became clear that the current seismic provisions in specifications and retrofitting manuals require updating to include more information on the realities of the east coast. This includes provisions for steel members, semi-rigid connections, composite members, performance requirements, and ground motions.

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