SEISMIC DESIGN OF TWO NEW YORK BRIDGES

Beile Yin, Ph.D., PE, F. ASCE, Hardesty & Hanover, LLP, New York, USA

ABSTRACT

The Third Avenue Bridge and Willis Avenue Bridge are two adjacent bridges each carrying one directional traffic across Harlem River in New York City. The replacement of 100-year old Third Avenue Bridge is complete. The new 5-lane crossing consists of 17 approach spans and a 107-m long swing span for an overall bridge length of 457m. Including on-grade approaches, the overall project length is 1067m. Construction of the $118.8 million bridge replacement project began in 2001 and completed in 2005.

The Willis Avenue Swing Bridge over the Harlem River is being replaced under a $612 million project which is massive in scope as it extends over a mile in length, passes over the Harlem River and an adjoining railyard and provides connections between two major highways as well as three major arterial streets. The new alignment not only dramatically improves the alignment from that of the 100 year old existing bridge but also facilitates maintaining 70,000 vehicles per day of roadway traffic as well as maintaining navigation on the river. The project centerpiece is a new four lane, 106 meter long swing span.

This paper focuses on the seismic analysis and seismic design of the two bridges. Challenges faced by the designers of the new bridge, and solutions developed to meet these challenges will be introduced. Since similarities exist between these two bridges, more detailed description will be on one of them, i.e. Third Avenue Bridge, and the other one is only in brief.

Keywords: Seismic, Design, Swing Bridge, Harlem River, Pivot Pier, Pivot Bearing, Truss, New York, Non-linear

INTRODUCTION These two bridges, Third Avenue Bridge and Willis Avenue Bridge, have served as a vital part of New York City's infrastructure for over 100 years. Spanning the Harlem River, the bridges are essential components in the critical system of crossings that link the boroughs of Manhattan and the Bronx over this navigable waterway and is two of seven Harlem River drawbridges owned and operated by the New York City. Having been originally designed to carry trolleys and horse-drawn carriages, the burden of carrying New York City traffic for over 100 years has taken its toll on the existing structure, which can no longer accommodate modern demands.

To address the problems plaguing the structure - traffic congestion; substandard geometry that has led to high accident counts; deteriorating components; inadequate live load capacity; inadequate seismic capacity; and obsolete, deficient mechanical and electrical systems - the replacements of the two aging bridges are deemed necessary. The $118.8 million Reconstruction of the Third Avenue Swing Bridge, began in 2001 and completed in 2005, involves six stages of work, and includes complete substructure and superstructure replacement of the ramps, approach spans, and swing span, including the mechanical and electrical systems and control house. In total, the reconstruction project encompasses 1067m of structure.

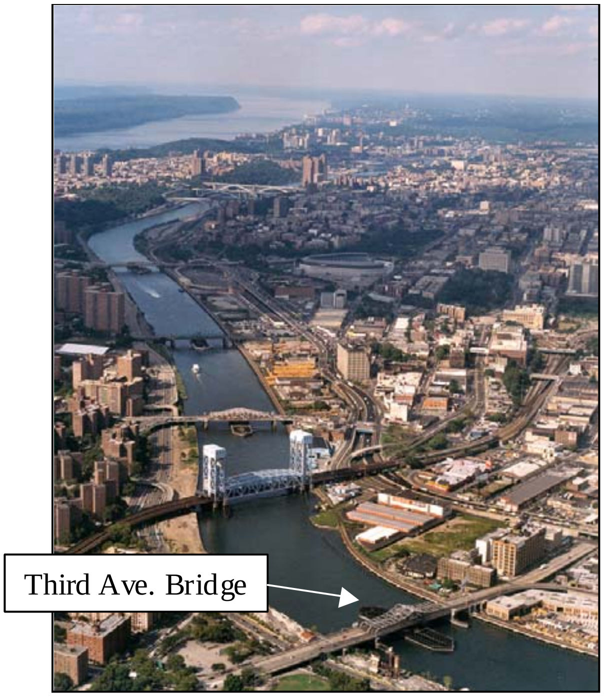


Fig. 1 View of Harlem River Looking North

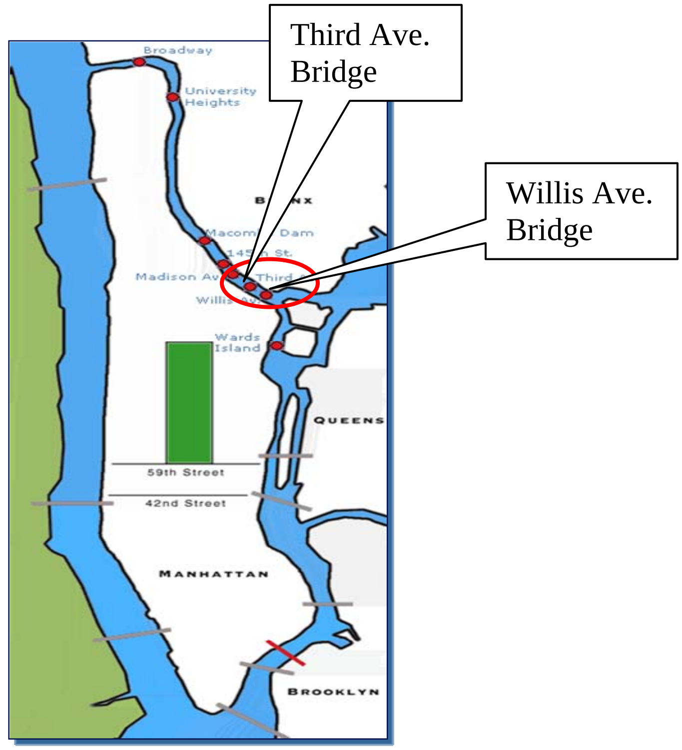


Fig. 2 Map of Harlem River

The Willis Avenue Swing Bridge over the Harlem River is being replaced under a $612 million project which is massive in scope as it extends over a mile in length between two boroughs of the City, passes over the Harlem River and an adjoining rail yard and provides connections between two major highways as well as three major arterial streets. The new alignment not only dramatically improves the alignment from that of the 100 year old existing bridge but also facilitates maintaining both 70,000 vehicles per day of roadway traffic as well as maintaining navigation on the river. Beginning in 2007 for construction, the project centerpiece is a new four lane, 106 meter long swing span.

THE THIRD AVENUE BRIDGE In contrast to the high volume of vessels that traveled beneath the bridge and through its draw span for several decades after its construction, current boat traffic on the Harlem River is relatively light. Currently, less than 10,000 vessels pass beneath the closed span annually. The span swings open only less than 30 times a year, mostly for routine maintenance. For the first 30 years or so of its existence, the span opened roughly 3000 times a year, with up to 9000 vessels passing through the drawn span annually. During this same period, another 50,000 to 75,000 boats passed beneath the closed span each year.

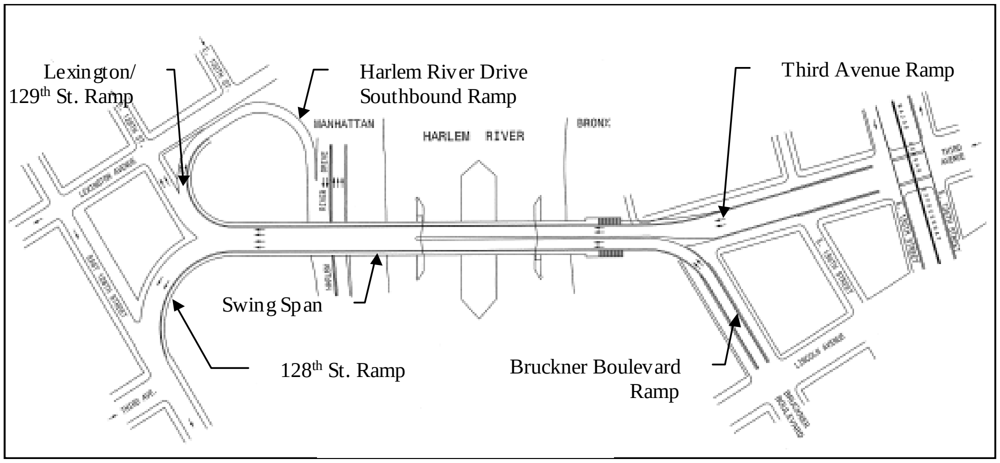


Fig. 3 General Plan of Third Avenue Bridge

General Configuration design of the new Third Avenue Bridge represents a significant improvement over the existing structure. In addition to addressing all the substandard and deficient conditions described above, the traffic capacity of the new bridge increases from 4 to 5 lanes in comparison to the existing structure, and the horizontal clearance of each of the navigation channels increases from 30m to 35m. To achieve vertical clearance requirements throughout, the profile of the replacement structure has been raised roughly 1.5m above the existing bridge. Additionally, the design increases the width of the sidewalks located on each side of the new bridge to 2.4m. In total, the new structure will measure 457m between abutments, and will consist of 18 spans, representing a significant reduction from the current 41 spans for roughly the same bridge length. Adding the nearly 610m of on-grade approaches, the overall project length is roughly 1067m.

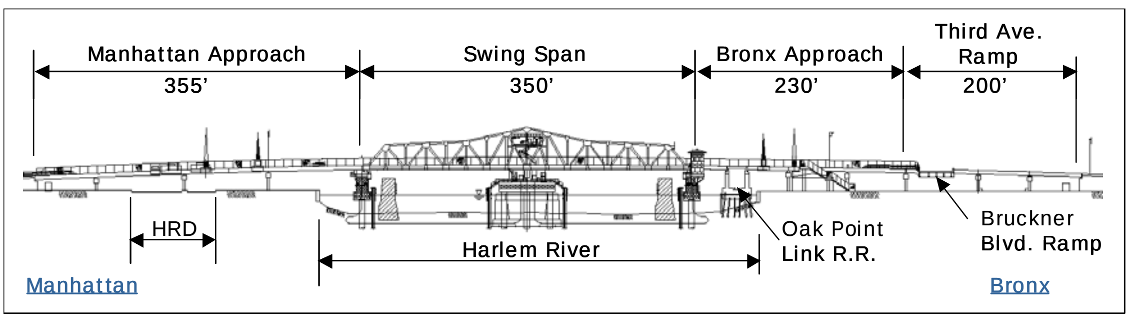
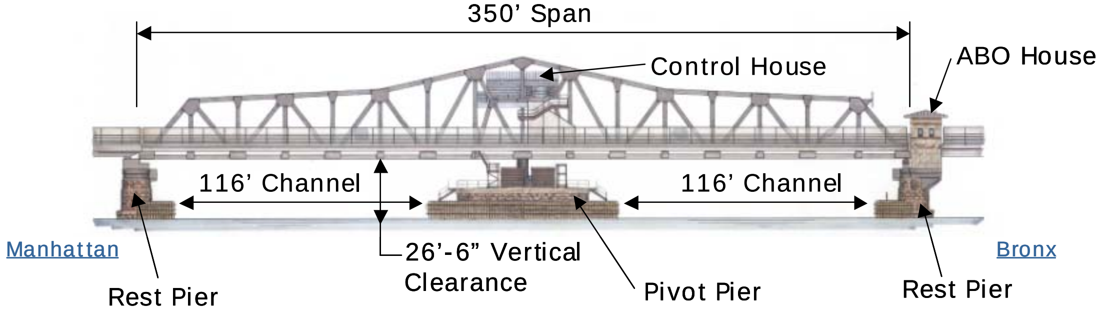
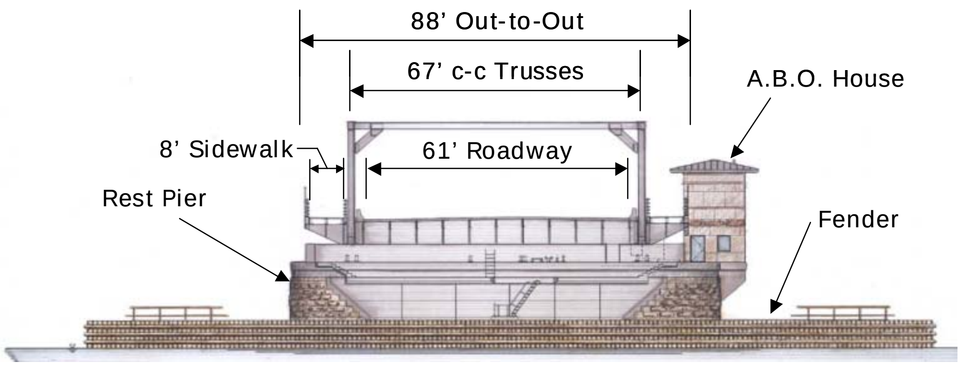


Fig. 4 General Elevation of New Third Avenue Bridge

Swing Span The main feature of the new bridge is the movable span. The design calls for a 107-m long, 26.8-m wide through truss swing span, operable from a control house located above the roadway at the center of the span. When drawn, the span will provide unlimited vertical clearance for the two equal 35-m wide channels. In the closed span position, the design provides a minimum vertical clearance of 8.1m, a 0.2m improvement over the existing bridge. Unlike its rim-bearing predecessor, the new span will be a center bearing swing, supported on a single center bearing on which the span will rotate when opening and closing. To best utilize the current channel configuration, the design locates this center pivot coincident to the center of the existing span.



(a) Elevation



(b) Section Through Swing Span at East Rest Pier

Fig. 5 Rendering of New Third Avenue Bridge Swing Span

Swing Span Substructure The substructure of the new swing span will consist of the same components as the existing bridge – a pivot pier at the center of the span, and a rest pier at each end of the span. The new pivot pier resembles a tabletop, with a 30.5x 18.3m reinforced concrete cap that is 3.4m thick and supported by ten 1.8-m diameter drilled shafts. The drilled shafts consist of ¾" thick steel casings, filled with reinforced concrete and socketed into rock beneath the river bottom. It is anticipated that the length of these shafts will be at least 30m. By locating the shafts at the perimeter and center of the pivot pier, the concrete cap will span over the existing center pier. This allows the existing pivot pier, which consists of a 100-year-old granite-faced concrete ring founded on a massive timber caisson, to remain in-place. Not only does this arrangement eliminate the need for costly demolition of the existing pier, but takes advantage of its hollow center by locating the drilled shafts clear of the sure-to-be impenetrable existing caisson. This arrangement also allows for ideal positioning of the center four shafts to carry the 2700 ton dead load of the new swing span concentrated at this location beneath the span's center bearing. By installing the six drilled shafts that are located beyond the existing pier and beyond the limits of the existing bridge prior to span float-out, significant construction time will be saved during the channel closure stage. The rest pier design consists of reinforced concrete shafts that are founded on drilled shafts similar to those utilized at the pivot pier. In addition to supporting the ends of the swing span under live load, the rest piers support the approach spans that flank the swing span, the end lift machinery, and the sockets for the swing span centering lock machinery.

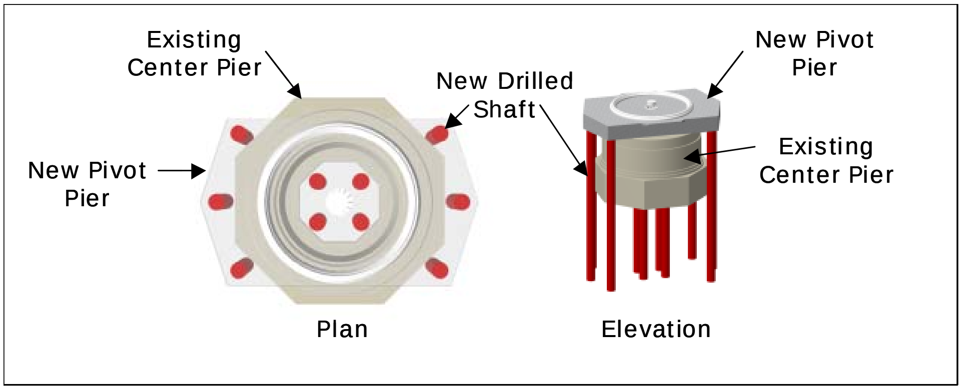


Fig. 6 Rendering of Pivot Pier

Swing Span Superstructure and Machinery Two parallel Warren trusses represent the main load-carrying members of the swing span superstructure design. Unlike for the three-truss arrangement of the existing span, traffic traveling across the new span can weave unimpeded. The new trusses each consist of 16 equally spaced panels and are braced together at the top by a system of sway frames and portals and at the bottom by the floorsystem. The individual truss members are welded steel boxes, ranging in dimension from 50cm x 50cm to 50cm x 60cm .

The steel floorsytem consists of parallel stringers spaced at just over 1.8m with floorbeams that span between the trusses. The floorbeam spacing matches the 6.7-meter truss panel spacing. The floorsystem directly supports the steel grating bridge deck, which will be filled with concrete for half its depth to create a smooth and durable riding surface. The key element of the floorsystem is the pivot girder, which not only serves as the floorbeam at the truss center panel point, but more importantly carries the full dead load of the span from the truss directly to the center bearing. The pivot girder is a 1.5-m wide, 4.6-m deep box girder that carries a cantilevered load of roughly 1350 tons at each of its ends.

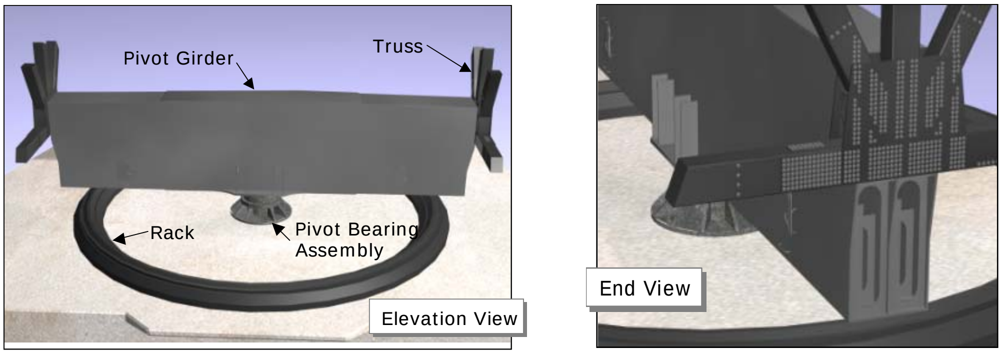


Fig. 7 Views of the Pivot Girder

For the limited paper length the mechanical and electrical issues will not be discussed in this paper.

Approaches The design of the Third Avenue ramp consists of three simple spans that range in length from 16 to 23m, totaling roughly 60m. With span lengths ranging from 44 to 79ft, the 5-span Bruckner Boulevard ramp totals 110m. The Bruckner Boulevard ramp arrangement includes three simple spans, two of which are curved, and two continuous spans. The remaining five spans on the Bronx side of the bridge comprise the Bronx approach, which carries traffic from the ramps to the swing span, and which spans a total distance of 70m. Included within the Bronx approach is a shallow 6-m span above the Oak Point Link Railroad.

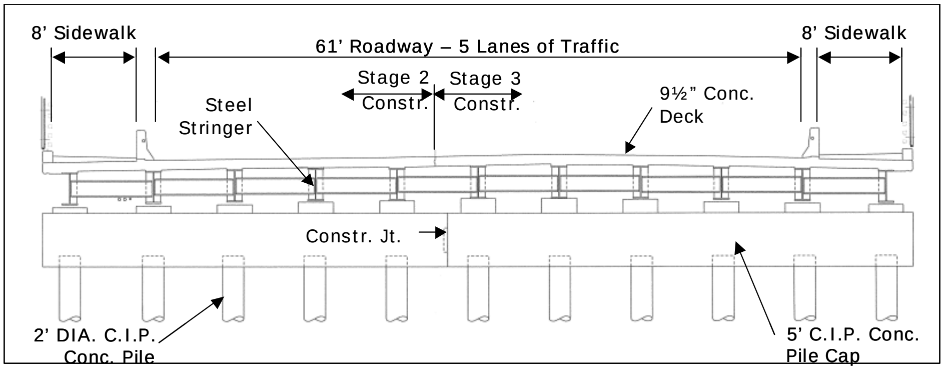


Fig. 8 Typical Section Through the Manhattan Approach

SEISMIC CRITERIA

- The Importance Classification (IC) is **Critical** for this bridge.
- The bridge is analyzed for **two earthquake levels**: a lower (functional) event having 10% probability of being exceeded in 50 years (500 year return period); and an upper (safety) event having a 2% in 50 years probability of exceedance (2500 year return period).
- Seismic Performance Category (SPC) is chosen as **B**.
- Site-specific soil effects for this bridge, including stiffness coefficients at ground level, spectrum and time-history ground acceleration, are obtained from our subconsultant Dr. Mishac Yegian.
- Structural damping is **0.05** for spectral method. For time-history, **Raleigh damping** is used.
- **Multi-mode Spectral** Method is used for the seismic analysis of the bridge, except the swing span. The **Time-history** Method is used for the swing span.
- For spectral method, the 30% rule has been used to account for the directional combination. Longitudinal and transverse earthquake loadings are applied respectively and the resulting forces and displacements combined by the **100%+30%** rule. Vertical earthquake loading was taken into account by increasing 30% member forces due to dead load.
- For time-history method, three directional loadings (**L+T+V**) are applied simultaneously to obtain the maximum forces and displacements. That is, 100% longitudinal, 100% transverse 100% vertical loadings are applied.
- Group Load = **1.0 (D+B(N/A)+SF(N/A)+E(N/A)+EQM)** (AASHTO IA)
- The member forces and displacements are calculated by combining the respective response quantities from individual modes by **SRSS** or **CQC** method. The later is used when the bridge has closely spaced modes (within 10%) to obtain the final response.

- Response Modification Factors (**R**)

Superstructure:	1;	
Wall-Type Pier:	2.5 for 2500-year	1.5 for 500-year;
Multiple Column Bent:	2.5 for 2500-year	1.5 for 500-year;
6 ft diameter Caisson:	1.5 for 2500-year	1.0 for 500-year
Connections:	1.0 for 2500-year	0.8 for 500-year;

  Unless otherwise noted
- The swing span is analyzed for closed position using full seismic load. One half the seismic load is used for open position.(AASHTO Standard Specifications for Movable Highway Bridges,1988)
- Hydrodynamic effect is very small. Therefore it is not included in the modeling. Usually, this effect is calculated by adding a mass equivalent to the displaced water volume. Since this added mass would be less than 1% of the total model, it is negligible.

SEISMIC GEOTECHNICAL INVESTIGATIONS Geotechnical earthquake engineering analyses were performed in support of the design of the replacement of the Third Avenue Bridge and its foundations. To evaluate the subsurface soil conditions and the dynamic soil properties, extensive field and laboratory exploration programs that included crosshole tests at two locations were implemented. The design level rock records were selected from the set of motions made available by the NYCDOT. Using these records multiple one-dimensional wave propagation analyses were performed, and the ground surface motions were generated for use in the soil-structure interaction (SSI) analysis of the bridge. In the dynamic response analysis of the bridge, the SSI effects were introduced through the use of foundation springs and dashpots at the bases of the bridge supports. The maximum drilled shaft shear forces and moments under the seismic loads were then computed and checked against the shaft capacities.

Soil Profile a comprehensive subsurface field exploration program has been implemented to define the soil conditions along the axis of the bridge. A total of 24 soil borings were made. In addition, at two locations (DHB-11A and DHB-13C), one in each approach of the bridge, crosshole tests were performed to obtain reliable estimates of the in-situ shear wave velocities of the soils and the bedrock.

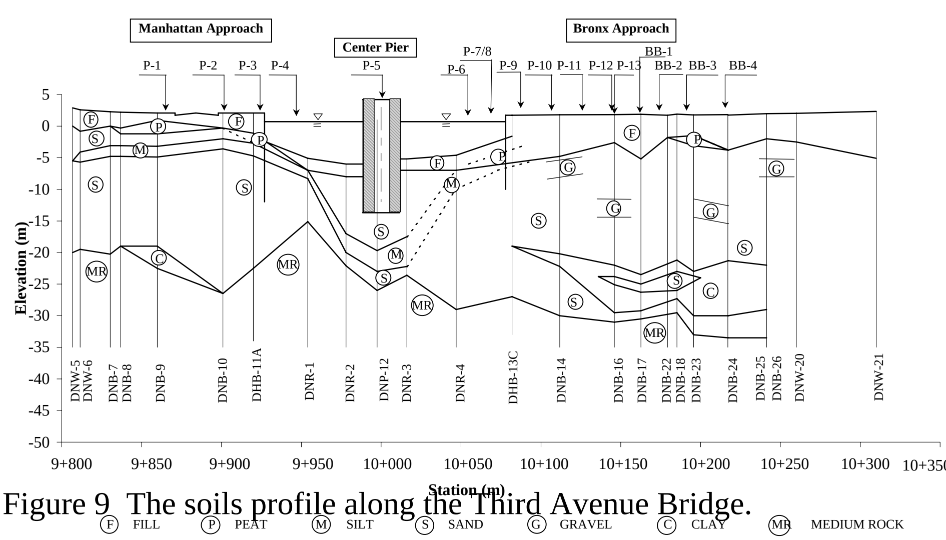


Figure 9 The soils profile along the Third Avenue Bridge.

Shear Wave Velocities Figure 10 shows the subsurface soil profile at one of the locations of the crosshole tests and the SPT N-values recorded. Included in the figure are the measured shear wave velocities $V_{s,}$ obtained from the crosshole test. For purposes of comparison, the V_s values for the soils at the site were also computed using the SPT-N values and the empirical procedures of Sykora and Koester (1988) and Seed et al. (1986). Clearly, the empirical procedures for this site overestimate the shear wave velocities of the soils by a factor of 1.5 to 2. The verestimation is most likely due to the presence of some gravel in the sand layer.

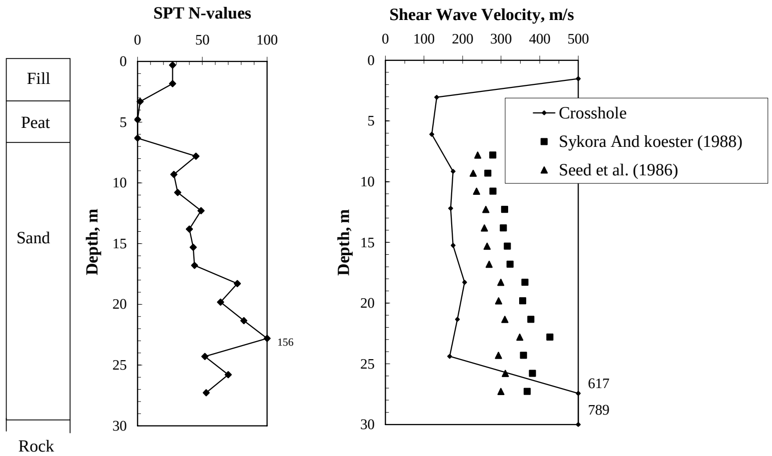


Figure 10 Comparisons of measured and estimated shear wave velocities.

Rock Motion The NYCDOT adopted a set of seismic guidelines that provided two levels of rock motions associated with 2500- and 500-year events. These hard rock spectra were established using a probabilistic seismic hazard analysis in which the likelihood of seismic events occurring in the region around New York City, as well as the resulting rock accelerations, were statistically combined. The RQD values of the rock cores retrieved from the boreholes made along the Third Avenue Bridge show that the rock at the bridge site is soft to medium hard, with shear wave velocities of about 2500 fps. Thus, the hard rock motions were scaled up by a factor of 1.25, and then used in the generation of the ground motions for input in the SSI analysis of the bridge.

Ground Motions The effect of the local site conditions upon the rock motion propagating through the soil profile was investigated using the theory of wave propagation. The computer program PROSHAKE was used to perform the site response analyses. The nonlinear soil behavior was considered through the use of strain-dependent shear moduli and damping ratios.

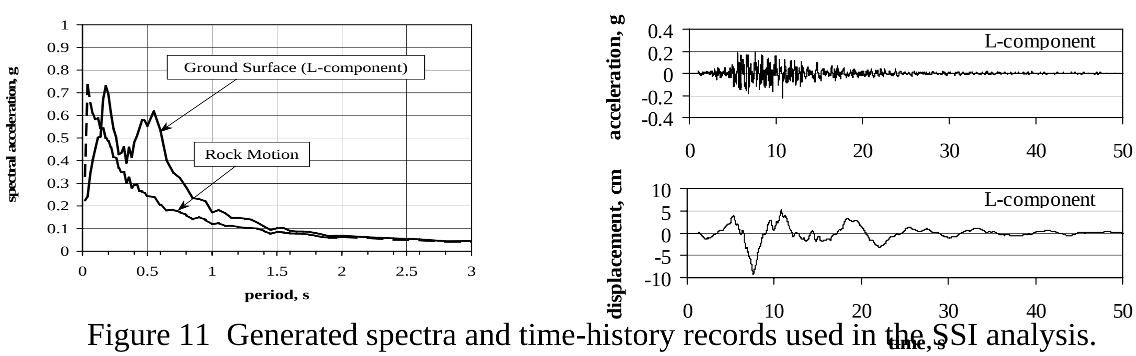


Figure 11 Generated spectra and time-history records used in the SSI analysis.

Foundation Impedances For The Center Pier In the dynamic response analysis of the bridge, the SSI effects were incorporated by introducing foundation springs and dashpots at the bases of the bridge supports. The coefficients defining the springs and dashpots depended on the foundation type, soil properties, soil strain levels induced by the seismic loads, and the frequency of excitation.

Due to the presence of the existing center pier caisson, the new retrofit-shafts will have nonsymmetrical lateral resistances. When the relative motion of a retrofit-shaft is *toward* the existing center pier caisson, its lateral stiffness will be larger than if the relative motion were *away* from the existing pier. This nonsymmetrical resistance was considered in the calculation of the lateral stiffness of the 6-ft diameter retrofit-shafts by using different p-multipliers in the computer program LPILE.

Since the analysis of the center pier was to be performed in the time–domain, damping coefficients for the retrofit-shafts were also provided for use in the SSI analysis. Radiational damping of energy from the retrofit-shafts to the surrounding soil was assumed to be negligible due to the presence of the existing center pier caisson. Thus, damping coefficients were computed taking into consideration only the energy loss due to soil internal damping.

Maximum Seismic Drilled Shaft Loads The SSI analysis of the retrofitted center pier was performed following the time-history approach. Three components of the ground motion computed from the site response analysis together with the foundation stiffness and damping coefficients were used, and the maximum forces and moments in each of the ten retrofit-shafts were computed at the mudline elevation.

SEISMIC STRUCTURAL ANALYSIS

Assumptions have been adopted in the numerical model as follows:

- The bridge is divided into three (3) sections in modeling and also reported in three corresponding volumes: Manhattan Approach, Swing Span and Bronx Span.
- In Manhattan approach, Spans 1 through 4 are modeled as one structure and Rest-Pier 4 is modeled as a separate structure. Because Pier 4 is relatively much stiffer than other three piers within this approach.
- In Bronx approach, viaduct spans 9 through 13 and ramp spans BB1 through BB5 are included in one model. Span 6, 7,and 8 are included in another model to simulate the railroad pier condition. No model is necessary for Pier 6. Because it is similar in condition with Pier 4 and apparently Pier 4 is the control case when using the same design for both two piers.
- For the Manhattan and Bronx spans, the superstructures are modeled with stringers as beam elements and concrete decks as shell elements. The multi-column bents are modeled as beam elements from ground level up.
- For Pier 4, only substructure is modeled. The superstructure weight is superimposed on the top of the substructure model. The substructure consists of pier wall, pile cap and six large diameter caissons, all modeled as beam elements with proper section properties. Each caisson is supported at riverbed level by springs with site-specific stiffness coefficients.
- Only **elastic** behaviors of the superstructure and substructure are considered. The elastomeric bearings that connect the bent and superstructure are in general modeled as elastic beam members. To account for the **bilinear elastic** behavior of the elastomeric bearings, an elastic property equivalent to the bilinear property was provided to give realistic seismic forces and displacements.

- At the ground connections, springs with geotechnical reaction stiffness coefficients are provided using the results from geotechnical investigation.

In addition to the general assumptions listed above, there are some exclusive features for the swing span. The following assumptions have been adopted in the modeling of the swing span.

- The superstructure swing span is modeled as a truss.
- The stringers and floor beams of the swing span are modeled as beam elements. The concrete deck is modeled as shell element. The density of the masses has been adjusted to match the manually calculated dead weight.
- The control house is modeled as a concentrated mass added in the middle of the corresponding truss members.
- The substructure of the swing span consists of 10 large diameter caissons connected with a thick concrete cap. This bench type substructure is quite stable to resist lateral seismic loading.
- Existing center pier will be truncated and remain in the place. Ten (10) new caissons are concrete piles cased in steel shell with diameter of 6 ft. Four (4) caissons will be placed in center of the existing pier and six (6) outside of the existing pier.
- The 10-Proposed caissons of 6-ft diameter are modeled as beam elements supported at the ground (riverbed) level with soil springs. Due to the existence of the old pier, the caisson will subject to different resistance when pushing toward and away from the existing pier. The pushing toward resistance is three times of that of pulling away. To account for this difference, soil springs are defined as bi-linear springs, and at bottom of each caisson, four (4) of this kind springs are attached.
- The 10’ thick concrete cap is modeled as shell elements.
- The links between truss superstructure and center pier and rest-piers featured non-linear properties that are exclusive for swing bridge. When the bridge is in the closed position, the links consist of center pivot assembly, center wedges, end-lift devices and end-centering devices. When in open position, the links only consist of center pivot assembly and balance wheels.
- The Center Pivot Assembly is modeled as two vertical beam elements linked together with a pin type joint. These two beams have large moment of Inertia and shear area. At the pin joint, the rotational degree of freedom about horizontal axes was released. But a nominal rotational stiffness about vertical axis was provided to account for rolling friction.
- The center wedge is a compression only member, which is modeled as an element with bi-linear elastic property.
- The end-lift device sustains a 100 kips vertical pre-compression in the closed position. Applying this 100 kips up-loading to the end of the superstructure truss, the corresponding up-displacement was obtained. The bi-linear elastic property then was built-up for modeling this element.
- The centering-lock device is a 6”x10” steel bar cantilevered 3’, which provides significant lateral resistance. It is modeled as a member taking only transverse load, which possesses the same stiffness as the real steel member.

WILLIS AVENUE BRIDGE Willis Avenue Bridge is an important crossing over Harlem River connecting Manhattan and Bronx. The seismic analysis of the Willis Avenue Replacement Bridge commenced in 1999. This project was placed on hold in 2001 and resumed in 2005. This bridge is defined as critical bridge and analyzed using a two level approach in accordance with the NYCDOT Seismic Design Guidelines. The first level that conforms to a functional event, is defined as having 10% probability of being exceeded in 50 years, or having 500 years return period. After such event, the bridge shall be fully accessible to normal traffic immediately (allow a few hours for inspection) with only minimal, easily repairable damage to non-primary structural elements. The second level that conforms to a safety event, is defined as having 2% probability of being exceeded in 50 years, or having 2500 years return period. During such event the bridge shall not collapse and provide limited access for emergency traffic within 48 hours, and full service within a few months.

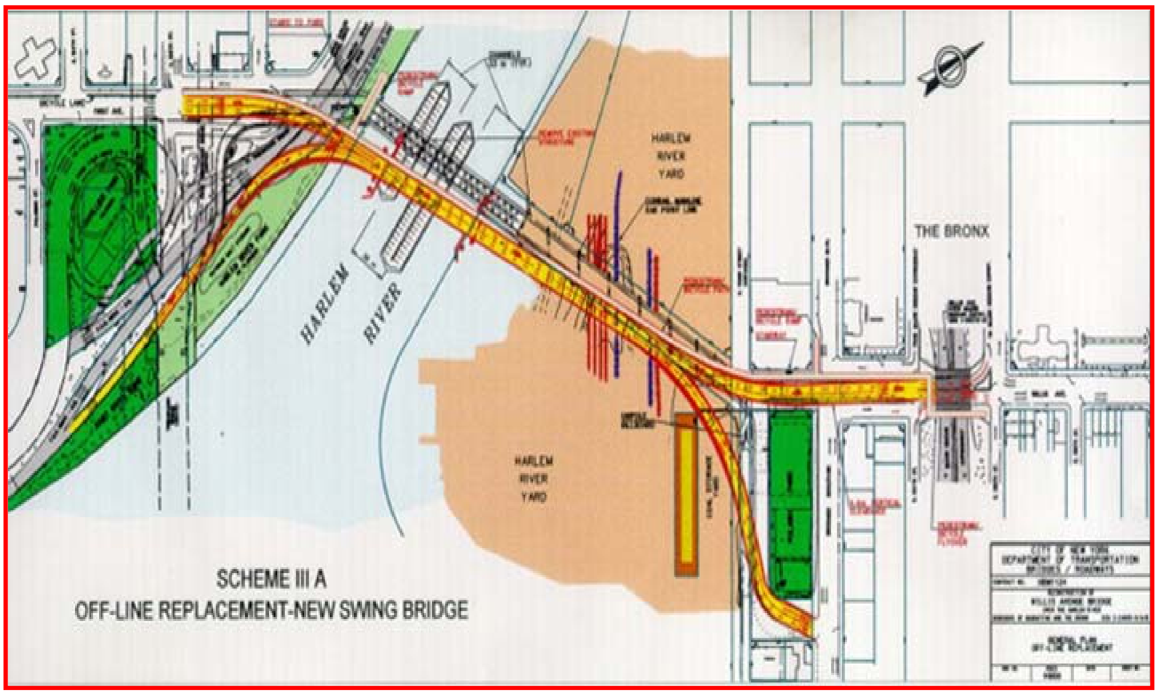


Figure 12 Layout of Willis Avenue Bridge

The bridge separates in a horizontal plane into two ramps on each side of swing span in Manhattan and Bronx Boroughs, see Figure 16 for the general plan of the bridge. On the Manhattan side the First Avenue approach and FDR Drive approach merge together as they approach the swing span. After the swing span, the bridge passes over the Harlem River Yard then splits into two ramps on Bronx side. One continues straight onto the Willis Avenue approach, where there is access to a northbound entrance ramp to the Major Deegan Expressway and the other becomes the Bruckner Boulevard Ramp. The bridge was divided into five sections for analysis. They are: Manhattan approach including First Avenue Approach and FDR Drive approach, Swing Span river crossing, Harlem River Yard Spans, Willis Avenue Approach and Bruckner Blvd. Ramp. These five portions were analyzed independently and are presented as five sections.

The program used for analysis was the ADINA finite element program, Version 8.3. and the SAP2000 non-linear finite element program. The SAP 2000 program was only used for Bruckner Blvd Ramp analysis. And the ADINA program was used for the rest sections of the bridge.

CONCLUSIONS

1. Earthquake-resist infrastructure is a demand from our real live and becomes as mandatory in engineering practice. However, any excessive requirement costs too much and would be not economically acceptable.
2. This paper demonstrates the practice of seismic engineering in the metropolitan of New York.
3. Non-linear characteristics should be serious considered in seismic analysis and design for important infrastructure particularly the highly non-linearly behaved structure.
4. Site specific ground motion and ground impedance along with the soil-structure interaction shall be included in the seismic engineering for a more reliable and realistic results.