disciplines as we move from the current code-based design state-of-practice into simulation-based designs.

Fig. 9: GVDA site with superimposed rendering of the test structure.

REFERENCES


FOUNDATION RETROFIT OF THE THIRD AVENUE BRIDGE IN NEW YORK

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ABSTRACT

The Third Avenue Bridge over the Harlem River links Manhattan to the Bronx. The pin bearing swing span bridge was constructed in 1898 and is in need of complete reconstruction to eliminate structural deficiencies and meet current traffic requirements as well as satisfy the seismic safety guidelines set forth by the NYCDOT. The new center bearing swing bridge, when reconstructed, will have a swing span of about 108 meters. Most of the piers and their foundations will be replaced. The critical center pier will be retrofitted using ten 6-ft diameter drilled shafts socketed into bedrock. Several major challenges regarding the design and construction of the replacement bridge were encountered. The existing center pier is founded on a hollow cylindrical caisson, which is partially embedded in the soil profile and does not rest on bedrock. This pier was found to be inadequate to carry the design seismic loads. The swing span mechanical system has minimal tolerance for permanent settlement and tilt of the foundation.

This paper presents the details of the new bridge and its site conditions, followed by the seismic geotechnical engineering analyses performed to develop the foundation retrofit design for the center pier.

INTRODUCTION

The existing Third Avenue Bridge has served as a vital part of New York City’s infrastructure since its construction in 1897. Having been originally designed to

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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 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 NYCDOT Division of Bridges moved ahead with plans to replace the aging bridge in 2001.

One of the major deficiencies of the existing bridge is its lack of capacity to resist seismic loads, which was not a consideration of the designers over a hundred years ago. The substructure, which includes masonry construction at the river span piers, abutments, and several other piers, has minimal (if any) ability to accommodate a seismic event. In addition, bearing details for superstructure members are generally seismic-prone.

This paper presents seismic geotechnical engineering aspects of the design of the retrofitted foundation for the center pier of the swing span.

**SWING SPAN**

The main feature of the new bridge is the movable span shown in Figure 1. 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.

The substructure of the new swing span will consist of the same components as the existing bridge—a pivot pier (center pier) at the center of the span, and a real pier at each end of the span. The new center pier resembles a table top, with a 100 ft x 60 ft reinforced concrete cap that is 11 ft thick and supported by ten 6 ft diameter retrofit drilled shafts.

The retrofit 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 100 ft. By locating the drilled shafts at the perimeter and center of the center pier, the new concrete cap will span over the existing center pier. This allows the existing center 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 retrofit shafts clear of the sump-to-be impenetrable existing caisson. This arrangement also allows for ideal positioning of the center four shafts to carry the 6 million pound dead load of the new swing span concentrated at this location beneath the span's center bearing.

By installing the six retrofit 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.

![Figure 1 The new Third Avenue Bridge and retrofit shafts of the center pier.](image)

**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 and 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. A number of foundation retrofit designs were evaluated. For each alternative, the foundation springs and damping were computed and the SSI analysis of the bridge was performed. The maximum shear forces and moments under the seismic loads were then compared and checked against the shaft capacities.

The details of the various analyses performed are presented in the following subsections, with a primary focus on the retrofit foundation of the center pier.
Soil Profile

Hardesty & Hanover implemented a comprehensive subsurface field exploration program 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 2 presents the soil profile as well as the locations of most of the boreholes and of the foundation piers that were evaluated in the seismic geotechnical investigations.

Figure 2 The soils profile along the Third Avenue Bridge.

The soil profile along the river crossing where the center pier of the bridge is located consists primarily of dense to very dense sands and clayey silts. Bedrock is encountered at about elevation -27 m at the center pier location. The rock is weathered granite mica Schist to the west of the pier and weathered marble beneath and to the cast of the pier. The average RQD of the rock is about 60%, indicative of soft to medium hard rock consistency.

Shear Wave Velocities

Figure 3 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 overestimation is most likely due to the presence of some gravel in the sand layer.

Figure 3 Comparison of measured and estimated shear wave velocities.

Based on the comparisons that were made between the measured and empirically estimated \( V_s \) values, the procedure of Seed et al. was modified to allow the use of the crosshole results in the estimation of shear wave velocities at other locations along the bridge. The modification that was introduced in Seed's approach was changing the coefficient of the parameter \( K_1 \) from 20 to 10. Using the reduced values of \( K_2 \), the estimated shear wave velocities for the two crosshole locations were in good agreement with those measured in the crosshole tests. Thus, the estimations of the shear wave velocities at other locations along the bridge were made using Seed's method with the modified parameters.

Rock Motion

In 1998, the NYCDOT adopted a set of seismic guidelines that provided two levels of rock motions associated with 2500- and 500-year events. Figure 4 shows the acceleration response spectra for the hard rock motions of the two events.

Figure 4 Hard rock spectra provided in the 1998 NYCDOT seismic guidelines.

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.
Hard rock, which is prevalent in the northeastern United States, has a shear wave velocity that is typically larger than 5000 fps. NEHRP (2000) classifies hard rock as Soil Profile A. It is well recognized that seismic waves propagating from hard rock to softer weathered rock can be amplified. For this reason, whenever the rock encountered at a bridge site is considered to be more soft than hard the NYCDOT seismic guidelines recommend magnification of the hard rock motions by a factor of 1.25.

The ROD 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 provided in the NYCDOT seismic guidelines 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 5: Generated spectra and time-history records used in the SSI analysis.](image)

Figure 5 presents a typical response spectrum and time history record of the computed longitudinal component of the motion in the free field, in the region of the center pier, for the 2500-year event. This longitudinal, transverse and vertical components of the computed motions used as input in the SSI analysis of the swing span of the retrofitted bridge.

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 piers. The coefficients defining the springs and dashpots depended on the foundation type, and properties, soil strain levels induced by the seismic loads, and the frequency of excitation.

The existing foundation of the swing span consists of a hollow cylindrical caisson that is partially embedded in the dense sand layer. This foundation was deemed inadequate to resist the seismic loads and to safeguard against excessive permanent displacements. After exploring a number of alternatives, a retrofit design that included ten 6-ft diameter drilled shafts socketed into the bedrock was selected. Four of the ten shafts will be installed in the hollow section of the existing pier, and the remaining six will be on the exterior, as shown in Figure 1.

A three-dimensional dynamic finite element analysis of the entire soil/refit shafts/existing caisson/bridge structure model was not within the scope of the project. In the 3-dimensional bridge superstructure analysis, the retrofit-shafts above the mudline were modeled as structural elements. At the base of these elements at the mudline elevation, foundation springs and dashpots were placed to account for the soil/ shaft-bridge interaction. The three components of the input ground motions obtained from site response analysis of the free-field soil profile were then specified at the base of the foundation springs and dashpots.

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 forward 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.

Due to the nonlinear soil behavior, the size of the seismic load on the retrofit-shafts will affect the stiffness coefficients. Since the seismic loads on the retrofit-shafts were not known, the stiffness coefficients were computed as a function of lateral force and shaft bending moment at mudline. Figure 6 presents the stiffness coefficients for three sets of retrofit-shafts, differentiated by their locations as inside, center, and side of the existing pier.

Since the drilled shafts at the mudline elevation can rotate as well as translate, the soil-coupling stiffness of the retrofit-shafts were important and were included in the SSI analysis. These stiffness plots were used by H&H in an iterative way, adjusting the stiffness coefficients in the SSI analysis according to the computed forces and moments in the springs. In the SSI analysis, each retrofit-shaft was assigned two different stiffness coefficients depending on the direction of the support motion relative to the existing center pier caissons - towards or away.

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.

Figure 6 Unsymmetrical stiffness of retrofit-shafts due the presence of existing center pier caisson.

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.

The LPILE program that was used to calculate the drilled shaft stiffness coefficients was also used to analyze the response of a typical retrofit-shaft under the 2500-year event seismic loads. In the stiffness calculations, p-multipliers were used to account for the stiffening effect of the existing pier as the shaft moved towards or away from it. In the drilled shaft load analysis, when the shaft movement was assumed to be towards the existing center pier caisson, the shaft loads below the mudline were larger. Thus, all the shaft load analyses using LPILE assumed movement towards the existing center pier caisson, resulting in conservative estimates of maximum shaft forces and bending moments.

Figure 7 presents a summary table of retrofit-shaft maximum forces and bending moments below the mudline. Similar to the stiffness computations, the ten retrofit-shafts were divided into three groups according to their locations relative to the existing center pier caisson. These forces and moments were then used in checking the structural integrity of the retrofit-shafts under the combined loads in the longitudinal and transverse directions of the bridge.

<table>
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<th>Shaft Location</th>
<th>Representative Shaft</th>
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<th>f_y (max)</th>
<th>f_z</th>
<th>m_y</th>
<th>m_z</th>
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<td>446</td>
<td>286</td>
<td>5692</td>
</tr>
</tbody>
</table>

V: Vertical; L: Longitudinal; T: Transverse

Figure 7 Maximum retrofit-shaft forces and moments under the 2500-year event.

SUMMARY

Seismic geotechnical investigations were performed in support of the design of the Third Avenue Replacement Bridge in New York. Crosshole tests were performed to obtain reliable estimates of the shear wave velocities of the soils and bedrock. The design level rock motions were selected from the set of motions provided by the NYCDOT seismic guidelines. The ground motions were computed taking into account the spatial variability of the soil conditions. The center pier of the existing bridge was deemed inadequate under the design seismic event. A foundation retrofit措施 consisting of ten 6-ft diameter drilled shafts was designed to carry the new swing span of the replacement bridge. The effects of the interaction of the retrofit-shaft foundation with the existing center pier caisson were included in the stiffness and drilled shaft load calculations.

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