Paper No. 23

Structural Elements

Seismic Analysis And Retrofit Of Roosevelt Island Vertical Lift Bridge

Beile Yin, PhD, P.E.
Nicholas J. Altebrando, P.E.
Frank M. Marzella, P.E.
Hardesty & Hanover, LLP

HEAVY MOVABLE STRUCTURES, INC.

TENTH BIENNIAL SYMPOSIUM

OCTOBER 25 - 28, 2004
The Omni Orlando Resort at ChampionsGate

Abstract

This paper addresses the necessity for rehabilitation of Roosevelt Island Bridge—classified as a critical structure—in order to meet the latest seismic requirements of NYCDOT and AASHTO. A full-scale finite element model of this bridge was built in order to perform seismic evaluation, and a two-level, nonlinear time history analysis was carried out to estimate the seismic behavior of the structure. The analytical results of the non-linear model of the bridge and the seismic evaluation procedure indicated that a few elements of the bridge required strengthening in order to meet the current NYCDOT seismic criteria and specifications. A practical retrofit procedure for the inadequate components was developed: foundations for two piers were widened, some steel members and bearings were replaced, and grout injection was incorporated to improve slope stability.

Introduction

Movable bridges are an important part of transportation infrastructure around the world. As movable bridges continue to be built, existing bridges must be maintained, retrofitted, and upgraded to meet the transportation requirements of the new millennium. Many existing bridge structures built before the 1960s were poorly designed and poorly detailed for seismic actions. Field reconnaissance reports, compiled from data taken from past earthquakes, indicate that mitigation measures could have prevented many bridge failures. The need exists to evaluate the seismic behavior of bridge structures and to determine appropriate retrofit methods that will improve the seismic responses of those structures.

A century of service since its construction in 1951, Roosevelt Island Lift Bridge is one of many bridges that now need rehabilitation to meet the latest seismic requirements of NYCDOT and AASHTO. The Roosevelt Island Bridge is the only road link between Roosevelt Island and the borough of Queens, New York, and is thus, classified as a critical bridge for seismic design. A complete nonlinear dynamic analysis was performed to evaluate the seismic characteristics, and to determine if any of the structural components of the bridge were susceptible to damage and the earthquake's impact on the overall structure. This paper outlines seismic analysis requirements, criteria and modeling assumptions followed in performing the seismic evaluation in accordance with the various codes for movable bridges, and the retrofit strategies considered for Roosevelt Island Bridge.

For lift bridges, the important items that need to be considered in seismic analysis are the counterweights, the towers, the lateral bracing of the lift span, and the foundations. The towers of a lift bridge serve mainly two functions. The first is to act as a guide to the span as it moves up and down, and the second is to support the counterweight laterally, which is paramount in seismic analysis. Each counterweight is approximately equal to half the weight of the span. Mostly the governing case for seismic analysis is the closed position of the bridge (i.e. the counterweight raised), thereby providing the masses a lever arm equal to the height of tower that amplifies the seismic forces. Counterweights slide vertically in the counterweight guides as the lift span raises and lowers. The counterweight guides are slender vertical members and usually fail under a seismic event. Failure of guides is not a great concern and is acceptable to most of the bridge owners since their failure actually isolates the structure from much of the load that would otherwise be transferred to the remainder of the structure.

Bridge Description

Roosevelt Island Bridge's main span, a vertical lift, presently carries one inbound lane, one outbound lane, and a sidewalk (see fig. 1). The 418-ft long lift-span was constructed from a Warren truss with a 5-in. open steel deck. The total length of the bridge is 936.5ft including the flanking spans and approach spans on the east and west side of the main span. There are

Fig. 1. Roosevelt Island Vertical Lift Bridge

two vertical towers, one on each side of the lift span, that carry the lift span and the counterweights. These two vertical lift towers rest on concrete piers. The piers supporting the lift span towers are concrete walls. The piers supporting the flanking spans are concrete multi-column piers, while the piers supporting the approach spans are steel hammerheads. The foundations of all piers are on bedrock except one of the east pier that is supported on steel H-piles.

Performance-Based Design Criteria

The performance goals based on AASHTO and NYCDOT seismic design criteria-guidelines were followed to assess the seismic performance of the bridge. These guidelines were used to frame performance-based criteria for the bridge. Criteria stipulated that the structure must comply with specified performance levels for two distinct seismic levels defined as the low level seismic event with a return period of 500 years, and the high level seismic event with a return period of 2,500 years. The performance levels were further classified into three damage levels: *No Damage*, *Minimum Damage, and Significant Damage*. *No Damage* was permitted to structural and non-structural members and no flexure cracking in concrete members for perfectly elastic behavior. For a 500-year earthquake return period, *Minimum Damage* was permitted to non-structural members and limited narrow flexure cracking in concrete. For a 2,500-year earthquake return period, *Significant Damage* in terms of extensive cracking and major spalling of concrete was permitted. Also, permanent deformation and local buckling of members was permitted without any collapse. In addition to these damage limits, criteria placed restrictions on lateral deformation. From our engineering experience, for a 500-year earthquake, the lateral deformations were limited to 0.75in. for foundations, 1.5in. at the top of piers of movable spans, and 4.5in. at the top of towers. For a 2,500-year earthquake, the lateral deformations were limited to 1.5in. for foundations, 3in. at the top of piers of movable spans, and 9in. at the top of towers.

For each damage level, criteria further specified compliance requirements by limiting either force or material strain levels. Compliance with the *No Damage* level was achieved when the seismic demands did not exceed the nominal capacity of members as determined by AASHTO. At the *Minimum Damage* level, strains in concrete were limited to less than 0.003 and strains in steel were permitted to 0.003. At the *Significant Damage* level, ultimate strains were permitted in concrete and steel members. In some cases, strains in concrete were allowed up to 0.004.

Bridge Modeling

A three-dimensional, nonlinear dynamic analytical model of the Roosevelt Island Vertical Lift Bridge was built in SAP 2000 Nonlinear Version 8.1.5 (see fig. 2). The bridge superstructure was primarily constructed from beam and shell elements. Shell elements were used for modeling the deck for flanking and approach spans, and for metal sheet covering on the top of the towers. The existing steel and rocker bearings were modeled with linear link elements. The 5-in. open roadway grating on the lift span was considered to have no structural stiffness, however, the grating floor mass was included in the model. The counterweight was modeled as lump mass with both translational and rotational inertia. Also, the counterweight sheaves (large pulleys over which the cables passing from the counterweight to the span are draped) are massive objects located at top of the towers, and these sheaves were modeled as lumped masses.

Non-Linearity

All non-linearity in the bridge is contained in the area around the counterweight. A gap of 3/8-in. is allowed on each side of the counterweight between the counterweight guide billet and the counterweight guide (see fig. 3). It is important to model small gaps because the forces are reduced for most of the duration of the earthquake due to the isolation effect. However, when contact is made between the counterweight and its guide, the velocity of the mass is high, and the ensuing impact causes larger forces to develop than if the counterweight is fully restrained. This gap between the counterweight and the counterweight guide is modeled with Non Linear Link Gap elements in SAP.

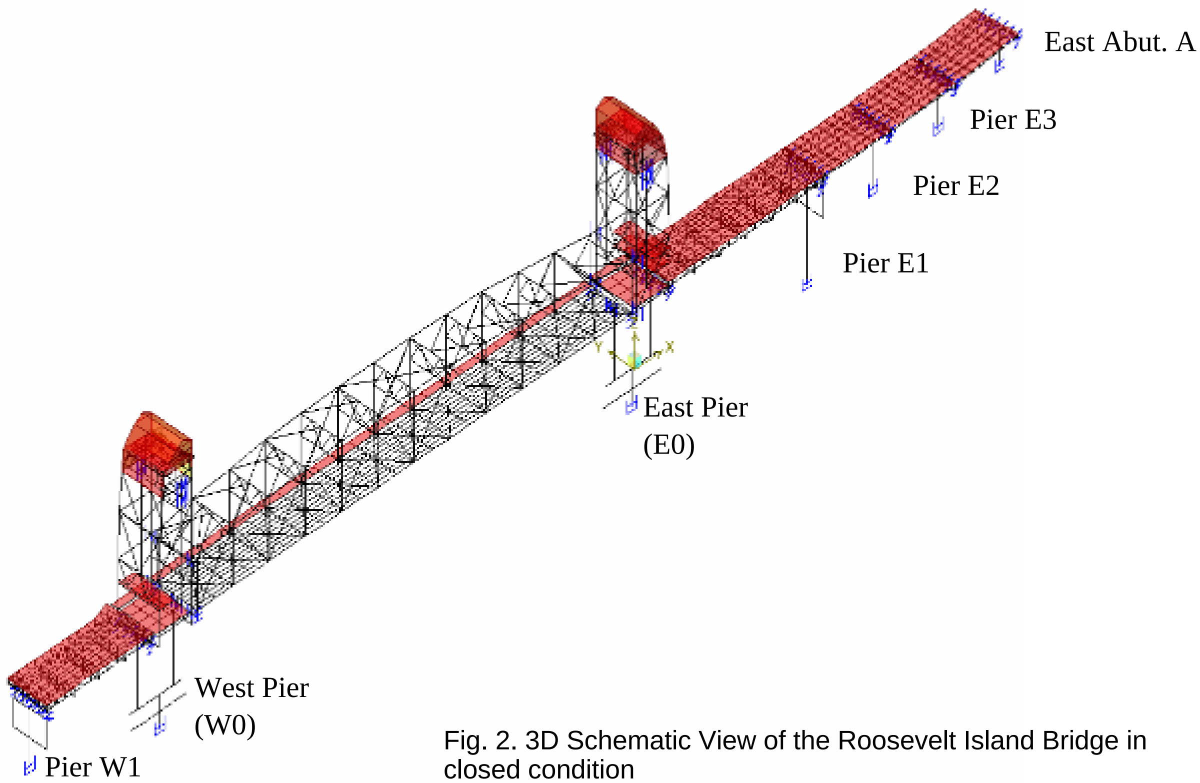

Fig. 2. 3D Schematic View of the Roosevelt Island Bridge in closed condition

Additional non-linearity was modeled at the interface of the lift span and the pier. Approximately 5% of the dead load of the span rests on the bearings, or shoes. These shoes have no uplift capacity, and during a seismic event, could lift off. Gap elements were utilized to model this effect. The locking and centering device is not a tight fit, and there are small gaps before force is developed in all directions. These were also modeled with gap elements. Other non-linearities exist in the real structure; however, they are considered insignificant and not modeled. The areas primarily affected by this non-linearity are the tower and the approach and flanking spans.

Ground Motions

The effect of local site condition upon the rock motion propagating through the soil profile was investigated using the theory of one-dimensional wage propagation. The rock motions that accompany the *1998 NYCDOT Seismic Guidelines* were used to select the appropriate horizontal and vertical components of the design rock motions. In this analysis, vertically propagating shear waves were considered, and the horizontal components of the motions at different depth within the soil profile were computed. Computer program PROSHAKE was used to perform the site response analysis. Nonlinear soil behavior was considered through the use of strain-dependent shear moduli and damping curves for sands and clays. In all the analyses, the input rock motion was specified at rock outcrop.

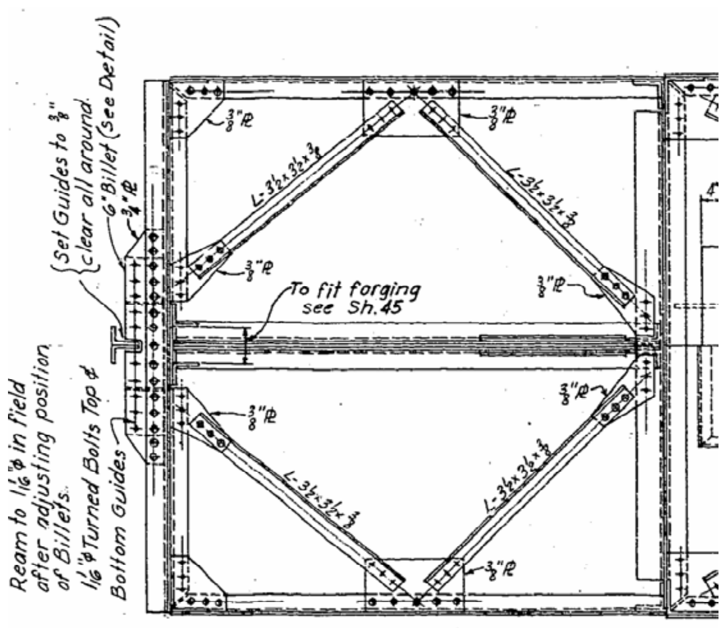

Fig. 3. Extract from Original Drawings Showing Counterweight Guide and Billet (Plan View)

Analysis revealed that ground motions for all piers were almost similar with upper bound bedrock motion at Pier W1. Pier E1, resting on piles and embedded into soil and rock to significant length, showed spectrum of motion in the free-field at the elevation of the bottom of the pile cap that was significantly larger than that at the bed rock. But, the spectrum for Pier E1 was for free-field condition that ignored the soil-pile-pile cap interaction effect on the motion that the pile cap would experience. Figure 4 shows the comparison of the longitudinal component of spectra at the base of the footing on rock and at free field cap-bottom elevation of Pier E1 for a 2,500-year event. In order to conclude whether to use spectrum with single motion with three components or multi-support excitation SSI analysis using the motions appropriate for each pier foundation location, it was decided to perform a three-dimensional seismic analysis of the foundation of Pier E1 to compute the motion at the bottom of the massless pile cap and compare that outcome with the upper bound bedrock motions at Pier W1.

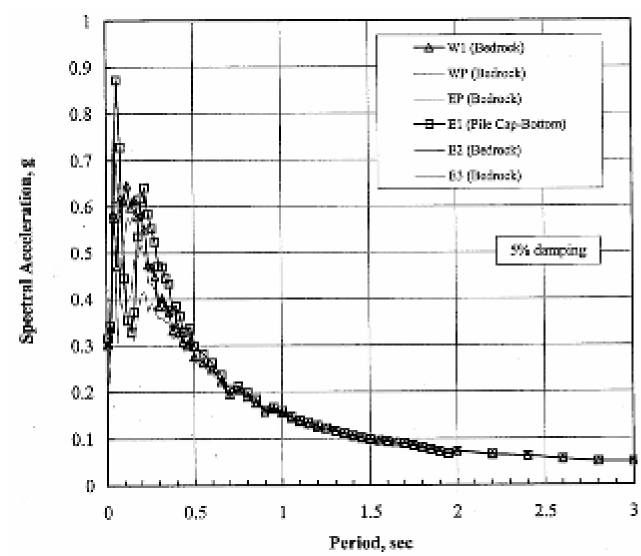

Fig. 4. Comparison of Spectra at Base of Footing on Rock and at Free-field Cap-bottom elevation of Pier E1 for 2,500-year Event, L-Component

The three-dimensional seismic analysis of the soil-pile-pile cap system was performed using the computer program SASSI. Figure 5 shows the details of the soil profile and the pier foundation used in the analysis. The shear wave velocities shown in the figure correspond to the strain-compatible values that were computed from the ground motion analysis of the soil profile at Pier E1. Both longitudinal and transverse components of the bed rock motion at Pier E1 (computed from PROSHAKE) were used as input in the program SASSI, and the motions at the bottom of the massless cap were computed. The analysis result revealed that the spectra of the bedrock motion at Pier W1 (which envelop the spectra of all the other piers on bed rock) plot very closely to those of the cap-bottom motions at Pier E1. Hence, it was reasonable to use the bedrock motion of Pier W1 (in three components) as a uniform input motion at all the supports of the Roosevelt Island Bridge.

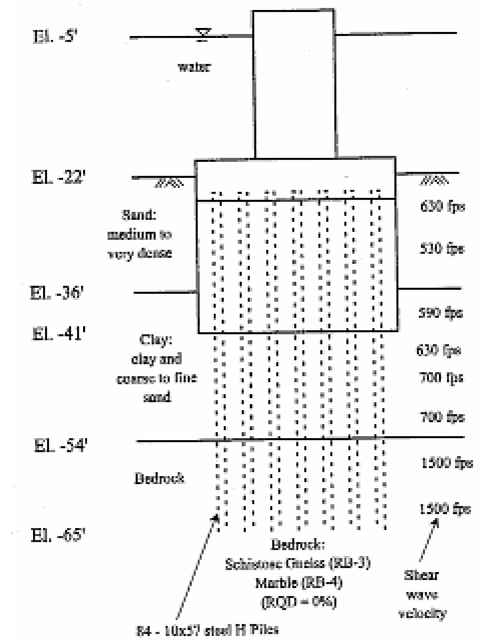

Fig. 5. Soil and Pile Foundation Details used in Seismic Analysis of Pier E1

The seismic analysis of the bridge was performed for two levels of ground motions corresponding to 10% and 2% probabilities of exceedance in 50 years (approximately 500-year and 2,500-year events). Figure 6 shows 2,500-year time history ground motions used for the analysis of the Roosevelt Island Bridge.

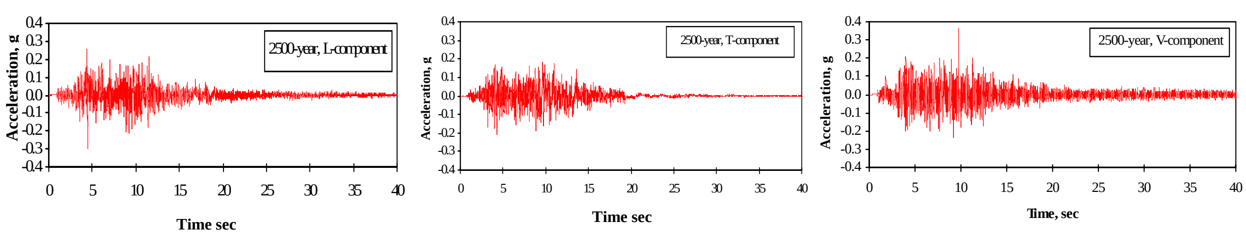

Fig. 6. 2,500-Year Time History Ground Motions

Soil Structure Interaction

In order to have the realistic results of dynamic analysis from the computer model, the soil-structure interaction effects were introduced by placing springs and dashpots to represent the soil-foundation systems. The stiffness and damping coefficients (foundation impedances) that depend on the foundation details, soil properties, soil strain levels induced by the seismic loads, and the frequency of excitation, were generated by developing full soil-structure continuum models. These coefficients were computed for all piers and the abutment, and were incorporated into the computer model using a 6x6 spring matrix.

Seismic Evaluation

To assess the performance of the lift bridge, the non-linear model was subjected to two levels of ground motions. The bridge response was evaluated for both 500-year seismic events and 2,500-year seismic events. The response quantities of interests were forces in abutment and foundations, compression and tension forces in tower and lift span members, bearings, tower displacements and the force in nonlinear gap element.

During the seismic evaluation, the seismic demand and the structural capacity were compared. The results were computed in form of factor of safety for footing stability and Capacity/Demand (C/D) ratio for other elements of the bridge.

The evaluation revealed that the foundations of the main towers and the East Abutment were found to be adequate to survive both the 500- and 2,500-year seismic events. The foundation to Pier E1 was found to be adequate for both the 500- and 2,500-year events. Scour of the Pier E1 foundation and the stability of soil between Piers E1 and E2 were both concerns that would affect the future performance of Pier E1. The foundations to Piers E2 and E3 were adequate for the 500-year event, but failed in overturning for the 2,500-year event. The foundation to Pier W1 was adequate for the 500-year event and was marginal in the 2,500-year event. If the resistance from bedrock was included, it was adequate. All of the piers were structurally adequate for both the 500- and 2,500-year events.

All superstructure elements were adequate for both the 500- and 2,500-year events, except for the counterweight guides, the top struts, and the lift span floor bracings.

The rocker bearings supporting the flanking spans were functionally obsolete, and their use is prohibited by AASHTO. The steel-on-steel sliding bearings used on the approach spans were functionally obsolete and unable to accommodate the maximum displacements in the 2,500-year event. The fixed bearings to the approach spans were unable to accommodate the forces in the 2,500-year event and failed in pintle shear. Along with structural components, some important mechanical components including trunnion shafts, counterweight sheaves, counterweight sheave bearings and guides, were evaluated and found to be adequate for both seismic events.

Displacement and force in the gap element are shown in figure 7. The force in gap element is from the counterweight on to the tower because of the small gap. It can be seen from these graphs that the force in the element is zero when the displacement is less than 3/8-in., and large peaks of force develop when the gap eventually closes.

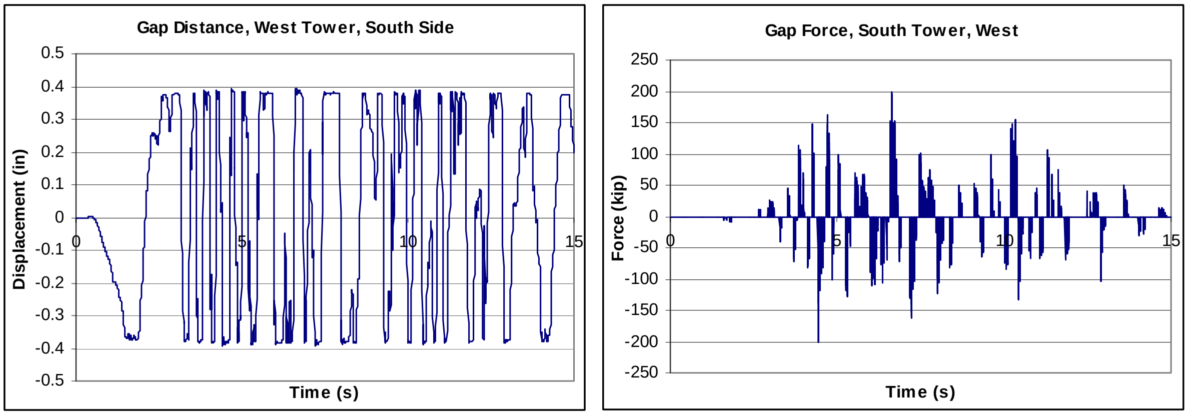

Fig. 7. Displacement and Force in Gap Element

The criteria allowed repairable damage to superstructure and foundation elements to respond beyond their elastic capacity, provided limits on maximum strain and displacement were met.

Retrofit Recommendations

Seismic safety of this bridge is a very important issue since this bridge is a critical transportation link and the only road link to Roosevelt Island. Malfunction of this bridge following an earthquake could result in tremendous loss to society and economic activities. Realistic assessment of the seismic performance of such complex bridges is a challenge.

The foundation, substructure, superstructure, and bearings of the Roosevelt Island Bridge were systematically evaluated for both 500- year and 2,500-year earthquake levels, and retrofit schemes were developed that address the inadequacies found based on the results from the nonlinear dynamic analysis.

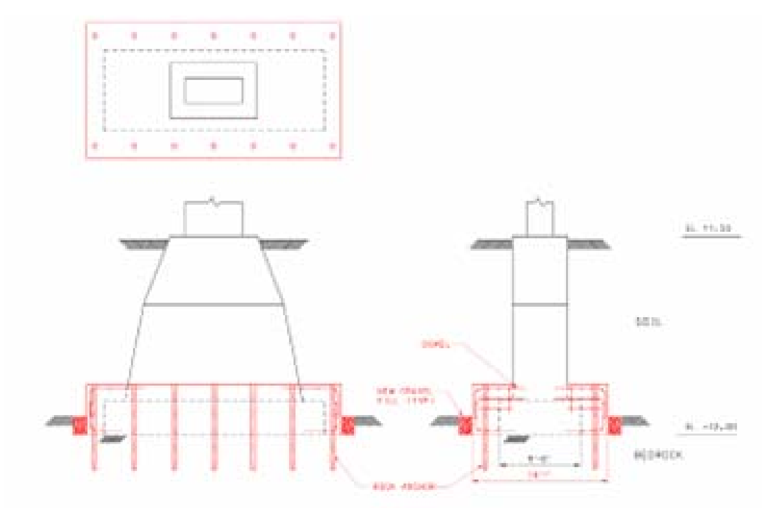

Fig. 8. Retrofit Scheme for Footing at Pier E2

The foundations to Piers E2 and E3 were adequate for the 500-year event, but failed in overturning during the 2,500-year event. These foundations should be retrofitted with rock anchor set into the bedrock. Retrofit schemes for footing of Piers E2 are shown in figure 8. The counterweight guide and the top tower struts failed in both the 500- and 2,500-year events, however, no retrofit is required for these elements. The failure of these elements prevents the transfer of very high lateral loads to the adjacent structure and protects the rest of the structure; normal operation of the bridge will not be affected. The floor bracing to the lift span failed in the 2,500-year event, and should be replaced with larger members.

The rocker bearings supporting the flanking spans are functionally obsolete, and should be replaced with elastomeric bearings. The steel sliding bearings to the approach spans are functionally obsolete and should be replaced with elastomeric bearings. Also the bearing seats of steel-on-steel sliding bearings are not in accordance with AASHTO requirements and should be retrofitted. The fixed bearings to the approach spans should be replaced with elastomeric bearings because they will fail in the 2,500-year event. Scour and slope stability of the soil near Pier E1 will effect the future seismic performance of the pier. The retrofit scheme for the bearing seat at the approach span at Pier E1 and the retrofit summary for elements of the bridge vulnerable to seismic event are shown in figures 9 and 10, respectively.

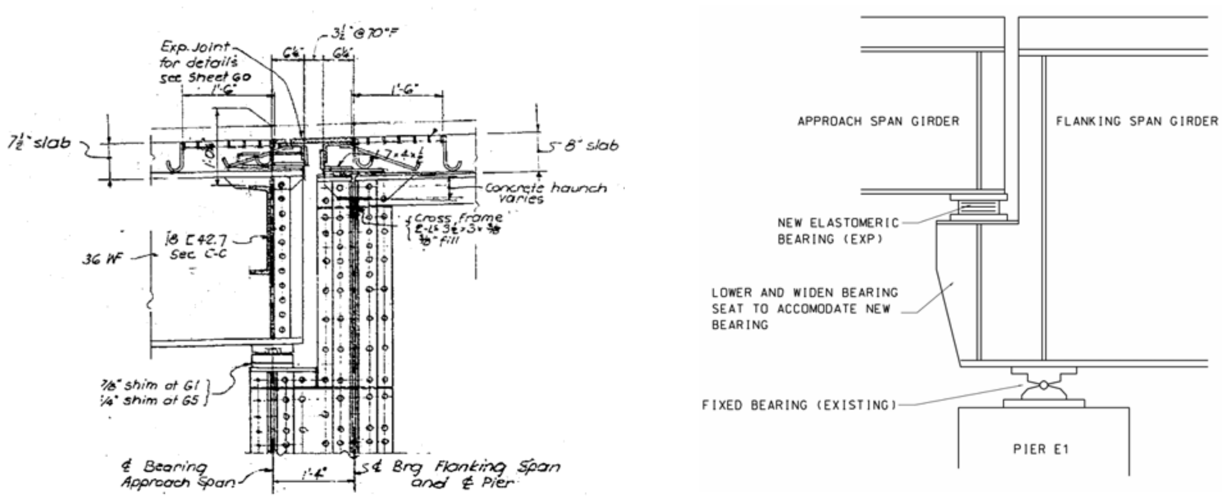

Fig. 9. Extract from Original Drawings and the Retrofit Scheme for Bearing Seat at Pier E1

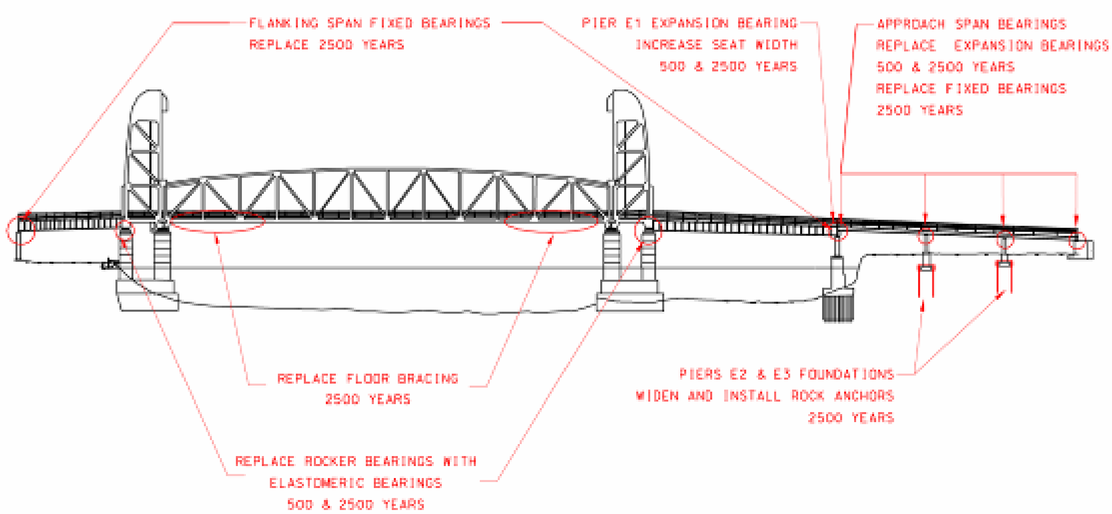

Fig. 10. Retrofit Summary for Components of the Bridge Vulnerable to Seismic Event

The computed seismic response possesses uncertainty, which had to be included in the retrofit design. Movable bridges are complex systems, and failure of one structural or mechanical component leads to the failure of other elements. To avoid successive failures and in the interest of cost effective solutions, adequate redundancy and fail-safe design were introduced in the vulnerable structural elements.

Further Study

Further study of the seismic response of the Roosevelt Island Bridge is now being planned to model more accurately the soil structure interaction at Piers E2 and E3, and also to investigate isolation bearings.

As previously noted, these piers require retrofit as originally modeled, and by taking into account the embedment of the piers into the soil, additional overturning capacity will be generated. Geotechnical software SASSI will be used for this task.

The *New York City Seismic Guidelines*, due to be updated in September 2004, will be incorporated into the final analysis.

Acknowledgements

The authors wish to thank and acknowledge Hardesty & Hanover engineers Rasmin Kharva, Michael Ludvik and Shizhong Yao for their contributions and participation in preparation of this paper. The authors would also like to acknowledge the New York City and New York State DOTs for their support and cooperation during this project.

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