Seismic Soil-Foundation Investigation of the Brooklyn Bridge

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ABSTRACT: A comprehensive seismic investigation of the Brooklyn Bridge was completed to assess its potential retrofit needs. The Brooklyn Bridge, built in 1883, has become a national treasure and architectural and engineering marvel. To ensure that the seismic retrofit needs of the bridge were based on a rational framework, avoiding overconservatism that would potentially lead to unnecessary retrofit and impacting negatively on the architecture of the bridge, advanced engineering investigations of the condition of the bridge and its seismic response were made. Specifically, the seismic investigation of the main bridge was performed following two approaches, referred to as the **global** and **local analyses**. In the global analysis, the entire main bridge with its foundation caissons was modeled, and the effects of soil-foundation interaction were incorporate through the use of foundation impedances. In the local analysis, each bridge tower with its caisson and the surrounding soils was investigated with a model using solid finite difference and slip and gap interface elements. The local analyses of the towers were performed to confirm quality of the motions and foundation impedances used in the global analysis, and to ensure that the conclusions regarding the potential need for foundation retrofitting was realistic and essential. This paper presents the details of the two seismic evaluation approaches, and compares the bridge foundation responses from both analyses. It also demonstrates the benefits of local analysis in the seismic evaluation of long-span bridges.

INTRODUCTION

The Brooklyn Bridge is the oldest of the East River Bridges in New York City. When completed in 1883, it was the world's first steel suspension bridge and had a center span more than 40% longer than other bridges. A photograph of the bridge is shown in Figure 1. The Bridge has become one of world's most recognizable and nationally celebrated historic landmarks. A comprehensive seismic evaluation of the bridge was recently completed to assess the potential retrofit needs. The scope of the seismic investigations included the Manhattan and Brooklyn masonry and steel approach structures, the ramps, and the main bridge. The main bridge is supported on massive cable anchorages and towers constructed of granite and limestone blocks.

Fig. 1 Photograph of the Brooklyn Bridge.

The main bridge has a center span of 486.3 meters (1595.5 ft) and side spans of 284.4 meters (933 ft). The cable anchorages are founded on a 1.2 meter (4 ft) thick timber grillage constructed of 0.3 meters (12 in) by 0.3 meters (12 in) Southern Pine. The size of the grillage is 36.4 meters (119.5 ft) by 40.2 meters (132 ft). Figure 2 shows elevations of the cable anchorages.

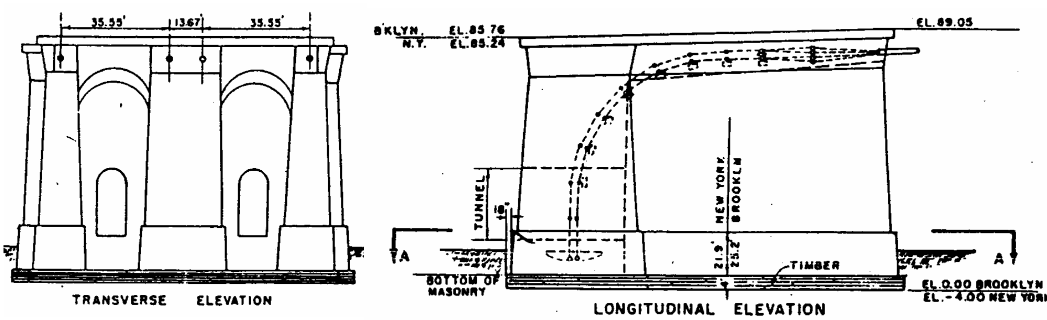


Fig. 2 Elevations of the Manhattan and Brooklyn Cable Anchorages.

The two towers of the Bridge are supported on caissons, which include a timber grillage 6.7 meters (22 ft) thick at the Manhattan Tower caisson, and 4.6 meters (15 ft) thick at the Brooklyn Tower caisson. Figure 3 shows elevations of the Manhattan and Brooklyn Towers. The Manhattan Tower caisson is entirely in the East River and is founded generally at elevation -23.8 meters (-78 ft) on an approximately 2.1 meters (7 ft) thick layer of very dense gravel, cobbles, and boulders overlying bedrock. The Brooklyn Tower caisson is on land and there is a bulkhead that laterally holds 12.2 meters (40 ft) of fill adjacent to the tower foundation. The base of the caisson is at elevation -13.7 meters (-45 ft), in a sand and gravel layer, overlying a 9.1 meters (30 ft) thick till layer over bedrock. The bedrock at the tower locations is slightly weathered.

The Brooklyn Bridge is an unusual structure with foundations constructed of massive limestone blocks, unreinforced concrete, and timber grillage. Its seismic evaluation warranted the applications of the most advanced and rigorous engineering evaluations to ensure that the assessment of seismic retrofit is made on rational and realistic scenarios, avoiding overconservatism that may lead to unnecessary retrofit and potential negative impact on the architecture of the bridge.

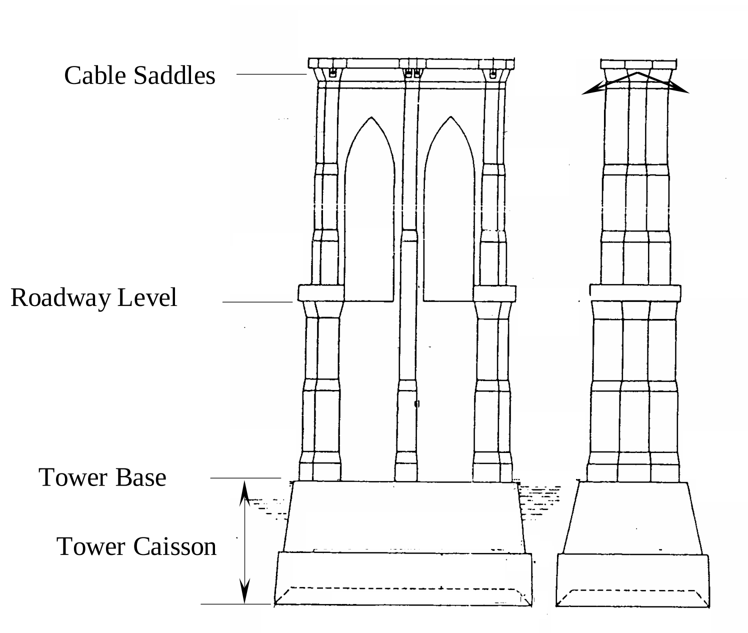


Fig. 3 Elevations of the Manhattan and Brooklyn Towers and their caissons.

This paper describes two advanced seismic analysis approaches that were utilized to assess the main bridge's retrofit needs. In the first approach, referred to as **global analysis**, the entire main bridge with its foundations was modeled in a single model in ADINA. Rigid elements were used to model the cable anchorages. The soil-caisson interaction was included through the use of foundation impedances. A spine model from beam elements and rigid links was used to represent the tower caissons. Non-linear springs with gap features along with dashpots represented the soil-structure interaction effects. Kinematic motions (motions influenced by the presence of the foundation caissons) were then applied to the foundation springs and dashpots. In the second approach, referred to as **local analysis**, each bridge tower, its caisson, and the surrounding soils were modeled in the computer program FLAC. In the analysis, the tower, caisson, and the soils were modeled using solid elements. The potential slip and gapping along the soil-caisson interfaces were modeled through the use of interface elements. The program uses the finite difference numerical technique to solve the static and dynamic response of the continuum consisting of the bridge tower, its caisson, and the surrounding soils. The purpose for using two soil-foundation-bridge interaction analysis approaches was to confirm the quality of the kinematic motions and foundation impedances, validate the analytical results from both models, and ensure that the final conclusions regarding the potential need for retrofitting, especially the bridge foundations are realistic and essential.

This paper presents descriptions of the global and local analysis approaches and demonstrates how these two analysis techniques complement each other in the seismic evaluation of the foundations of long span and critical bridges.

SOIL-FOUNDATION-BRIDGE ANALYSIS (GLOBAL ANALYSIS)

A global analysis of the bridge was performed using the computer program ADINA. The model of the bridge included the super- and sub-structures as well as foundation caisson elements. Figure 4 shows the global model of the bridge. The cable elements of the bridge were modeled with non-linear beam elements and the suspended structure and the towers with linear beam elements. Non-linear springs were included to account for cracking of the towers at specific locations. This cracking was identified using a detailed model from solid elements that was developed in the computer program ABACUS and the material properties of ANACAP-U material model 3 (2003). For the needs of the global analysis, the caissons were modeled in a manner that captured potential gapping and slipping along the caisson-soil interfaces.

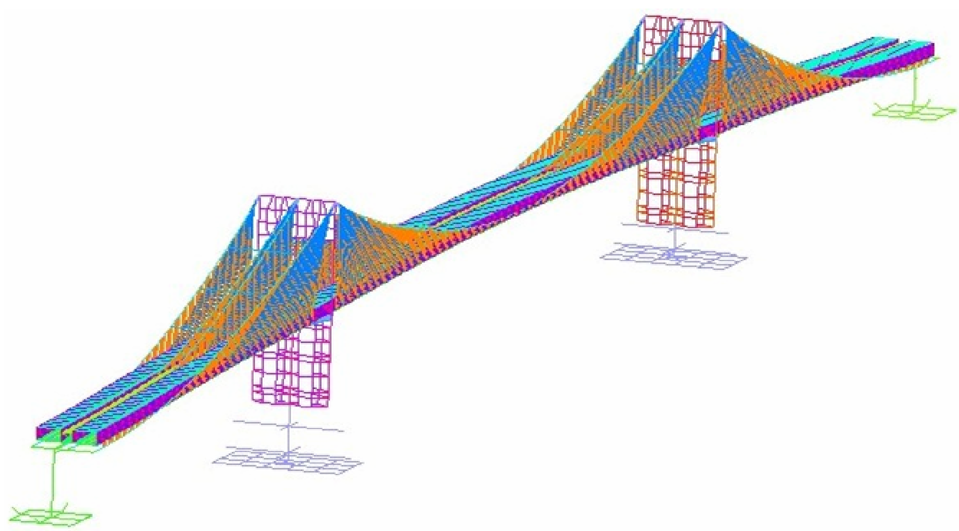


Fig. 4 Global analysis model of the Brooklyn Bridge.

The model consisted of three-dimensional elastic beam elements representing the spine of the caissons, rigid links, dashpots and truss elements with elasto-plastic hysteretic material properties and gapping features. In particular, the caisson base, which is a rigid surface 51.2 meters (168 ft) long by 31.1 meters (102 ft) wide, was modeled with rigid link elements and twenty-five non-linear truss elements in a configuration that facilitated the incorporation of the soil-caisson interaction at the base and calculation of peak soil stresses. This representation assumed rigid body motion of the base, which followed the deformations of the truss elements. The twenty-five elasto-plastic truss elements were connected at the other end to a rigid boundary surface, which was excited by the ground motions. This model was supplemented by two traction elements, one for each horizontal direction, to simulate the friction behavior between the caisson base and the soil. The horizontal elements were similar to the twenty-five vertical elements except that they represented the behavior of the entire base.

The interaction between the caisson walls and surrounding soils was modeled in a similar fashion as the base of the caissons. Following the limits of the soil strata and caisson configuration, the vertical walls were divided into several zones. Outrigger rigid link elements were used from the centerline (spine) of the caissons to the walls and elasto-plastic truss elements with similar properties as those at the caissons' base. Traction elements with elasto-plastic multi-linear material properties were also used at each outrigger to represent friction in the tangential and vertical directions.

The geotechnical input to the global analysis consisted primarily of ground motions and foundation impedances. Extensive field geotechnical and geophysical testing programs were implemented to characterize the site conditions and obtain reliable estimates of the shear and compression wave velocities of the soils, bedrock, foundation timber grillage and limestone blocks.

Cable Anchorage Motions and Foundation Impedances

In the global model of the Bridge, the soil-foundation effects were incorporated through the use of distributed springs and dashpots. Figure 5 shows typical locations of the springs and dashpots for the Brooklyn Cable Anchorage. Similar springs and dashpots were used for the Manhattan Cable Anchorage.

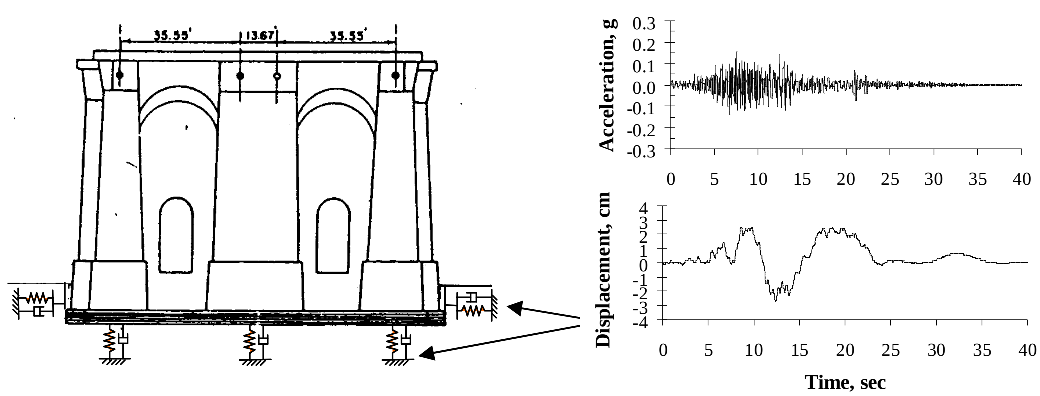


Fig. 5 Transverse elevation of the Brooklyn Cable Anchorage showing the locations of foundation springs and dashpots, and the kinematic displacement record used in the global analysis.

Springs and dashpots were placed at nine locations within the base and four locations along the sides of each anchorage. The kinematic motions of the anchorage that needed to be applied at each of the spring and dashpot locations were computed using the computer program SASSI. Figure 6 shows the SASSI model of the Brooklyn Cable Anchorage.

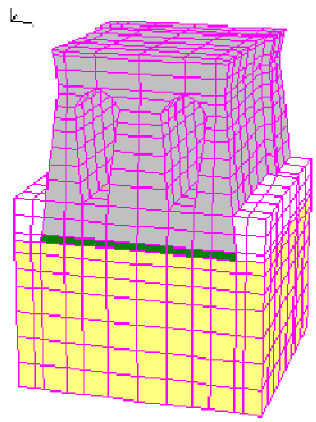


Fig. 6 SASSI model used in the kinematic motion and foundation impedance calculations for the Brooklyn Cable Anchorage.

Several rock motion time histories were selected from the set of records that the NYCDOT released in 2004 for analysis of its bridges. The appropriate records were selected and modified to represent the spatial variability of the motions and the rock condition at each of the bridge foundation locations. This paper presents motions corresponding to the 2500-year event.

The kinematic motions (motions ignoring the mass of the anchorage) at the base and along the sides of the cable anchorage were computed using the strain compatible shear moduli obtained from initial applications of one-dimensional site response analyses. To account for the effect of the spatial variability of the motions along the longitudinal axis of the bridge, the global analysis was performed using multi-support excitation, in which displacement time histories were specified at all foundation springs and dashpots, representing the interaction between foundations of the bridge and the soils. These displacement records for the cable anchorage caissons were obtained from the acceleration records calculated from SASSI after making the appropriate baseline corrections.

Figure 5 shows a typical displacement record that was specified in the global analysis at the base and sides of the Brooklyn Cable Anchorage. Similar records were computed for the Manhattan Cable Anchorage. Figure 7 shows a comparison of the response spectra of the motions at baserock, and at the bottom and sides of the Brooklyn Cable Anchorages.

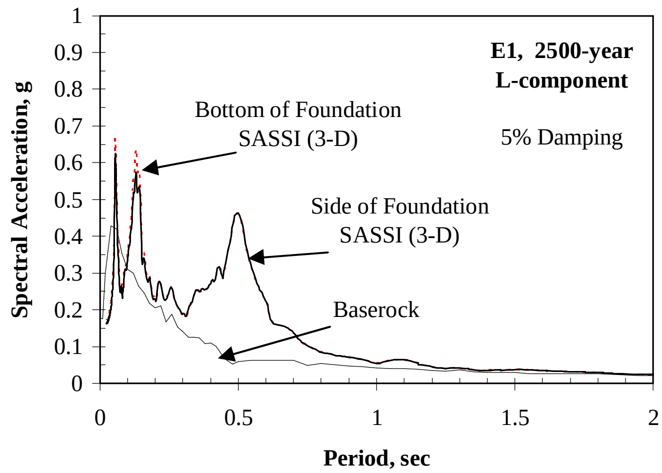


Fig. 7 Comparison of spectra of the bottom and sides of the Brooklyn Cable Anchorage foundation computed using three-dimensional SASSI analysis.

It is noted that the 11.3 meters (37 ft) thick stiff soil layer present below the caisson base has a large amplifying effect on the motion in the period range (0.47 seconds) of the stiff cable anchorage. Also, the motion along the side of the caisson is almost identical to that of the base, thus indicating that because of the large base dimensions of the caisson, it does not have a tendency to rock. Figure 8 shows comparisons of the spectra of the kinematic motions computed in the three-dimensional SASSI analysis with the spectra of the motions computed in the more conventional way of assuming a one–dimensional wave propagation (SHAKE analysis) without considering the presence of the caisson (free-field motion). As shown in Figure 8, clearly, the simplified one-dimensional analysis would have overestimated the intensity of the motion in the period range of the cable anchorage (0.47 seconds) by as much as 35%.

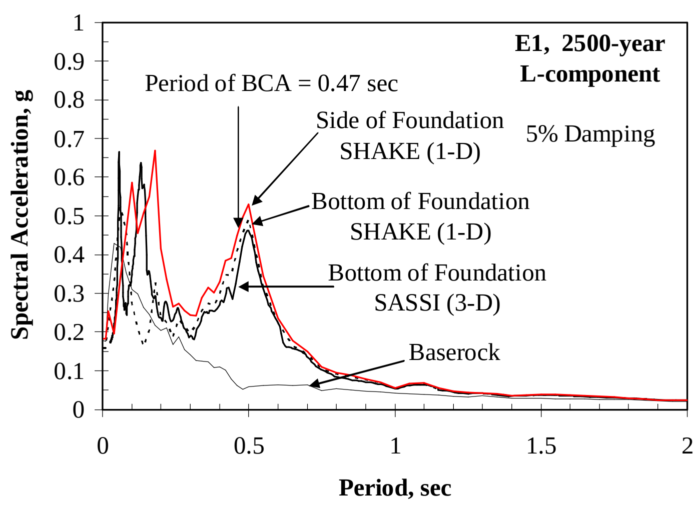
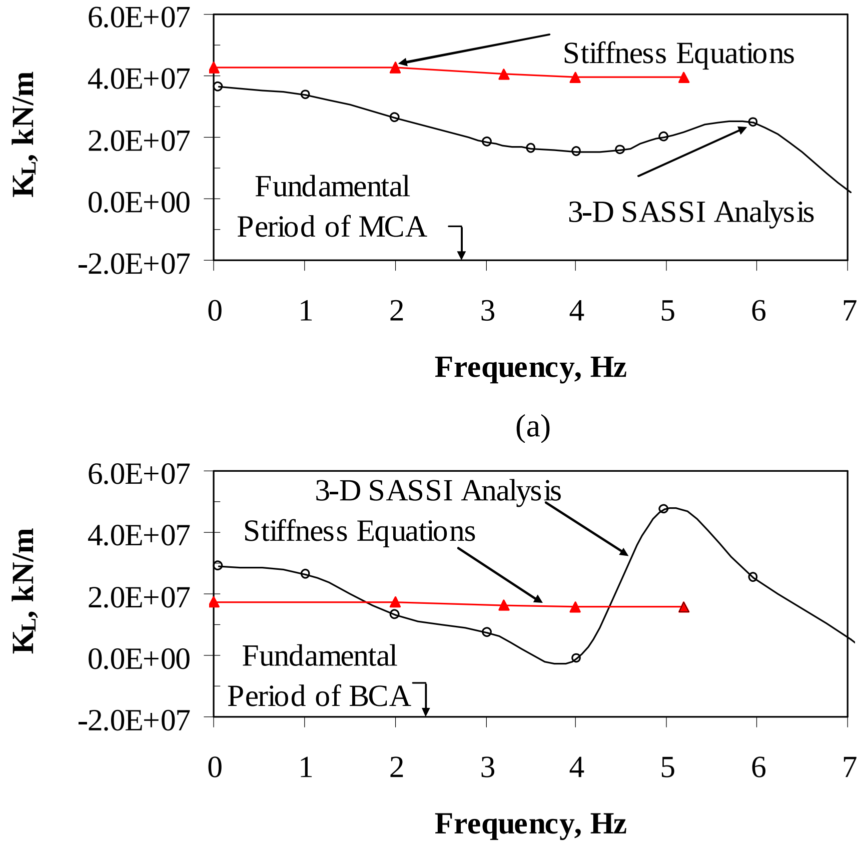


Fig. 8 Comparison of spectra from three-dimensional SASSI analysis with spectra obtained from one-dimensional SHAKE analysis.

The coefficients of the foundation springs and dashpots representing the soil-caisson interaction of the cable anchorages were computed using SASSI. The computed frequency-dependent stiffness coefficients in the longitudinal direction of the bridge for the Manhattan and Brooklyn Cable Anchorages are shown in Figure 9.



(b)

Fig. 9 Stiffness coefficients a) Manhattan Cable Anchorage, MCA, b) Brooklyn Cable Anchorage, BCA, obtained from SASSI, compared with stiffness coefficients obtained from stiffness equations for shallow foundations.

Included in Figure 9 are the stiffness coefficients computed based on simple stiffness equations for shallow foundations suggested by Gazetas (1991). Typically, within the frequency range of relevance to the anchorages (greater than 2 Hz), the simple equations overestimate the stiffness coefficients for the two cable anchorages. For example, the stiffness coefficient from the simple equations, for the Brooklyn Cable Anchorage with a fundamental period of 0.47 seconds (frequency of 2.2 Hz), is higher (by 60%) than the value computed from SASSI. This difference is not very large considering the approximations inherent in the formulations of the stiffness equations. However, because of the very stiff nature of the cable anchorage and the high frequency content of the earthquake motions, such overestimation of the stiffness would have resulted in the underestimation (by 40%) of the intensity of the motion experienced in the global analysis at the Brooklyn Cable Anchorage location, as is shown in Figure 10.

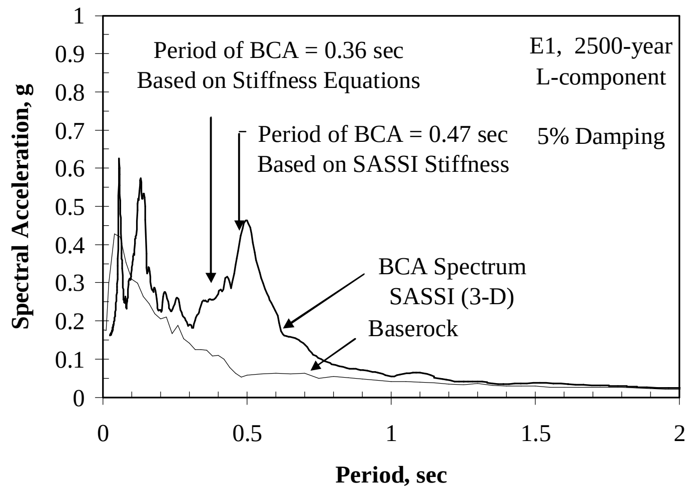


Fig. 10 Comparison of spectral accelerations for the Brooklyn Cable Anchorage computed based on stiffness coefficients from the three-dimensional SASSI analysis with stiffness coefficients from the equations for shallow foundations.

Through the comparisons made above, it is evident that simplified analysis procedures compared to more rigorous approaches may under- or over-estimate the dynamic responses of a structure depending on the characteristics of the soil and the foundation. Realistic estimates of seismic responses, especially for a critical structure such as the Brooklyn Bridge, warranted the application of advanced analytical procedures.

Table 1 presents a summary of the total stiffness and damping coefficients and damping ratios in the six modes of vibrations of the Brooklyn Cable Anchorage. These coefficients were then distributed to the nine springs and dashpots placed along the base and four along the sides of the caisson. The distribution of the total stiffness and damping coefficients to individual springs and dashpots was made by ensuring that the sum total horizontal and rocking stiffness and damping matched with the total values computed from SASSI. Note that because of large embedment of the Brooklyn Cable Anchorage caisson, the translational and torsional damping ratios are quite large due to radiational loss of energy away from the caisson. Similar calculations were made for the Manhattan Cable Anchorage caisson.

Table 1 Summary of SASSI soil-foundation dynamic impedances for the Brooklyn Cable Anchorage caisson.

Mode of Vibration	Approx. Natural Frequency, Hz.	Stiffness Coefficient K		Damping Coefficient C		Damping Ratio D
Longitudinal, L	2.3	K_L (kN/m)	1.2E+07	C_L (kN-sec/m)	6.1E+05	0.37
Transverse, T	2.3	K_T (kN/m)	1.2E+07	C_T (kN-sec/m)	6.1E+05	0.37
Vertical, V	3.8	K_V (kN/m)	3.2E+07	C_V (kN-sec/m)	6.7E+05	0.25
Rotation about L	3.3	K_{RL} (kN/m/rad)	8.1E+09	C_{RL} (kN-m-sec/rad)	6.5E+07	0.08
Rotation about T	3.4	K_{RT} (kN/m/rad)	9.5E+09	C_{RT} (kN-m-sec/rad)	9.5E+07	0.11
Rotation about V	4	K_{RV} (kN/m/rad)	6.8E+09	C_{RV} (kN-m-sec/rad)	3.5E+08	0.65

Tower Motions and Foundation Impedances

In the global analysis of the bridge, the soil-tower caisson interactions were considered through the use of springs and dashpots similar to those described for the Brooklyn Cable Anchorage. Figure 11 shows a longitudinal cross section of the Brooklyn Tower foundation depicting the locations of the springs and dashpots that were used in the global analysis model.

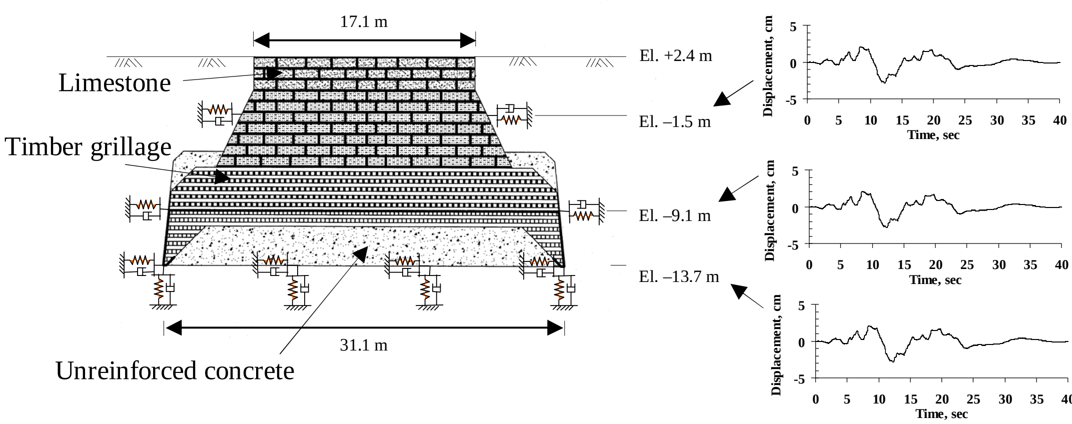


Fig. 11 Transverse elevation of the Brooklyn Tower foundation showing the locations of foundation springs and dashpots, and the kinematic displacement records used in the global analysis.

The kinematic motions applied at the locations of the springs were computed using the computer program FLAC. Figure 12 shows the FLAC model of the Brooklyn Tower foundation.

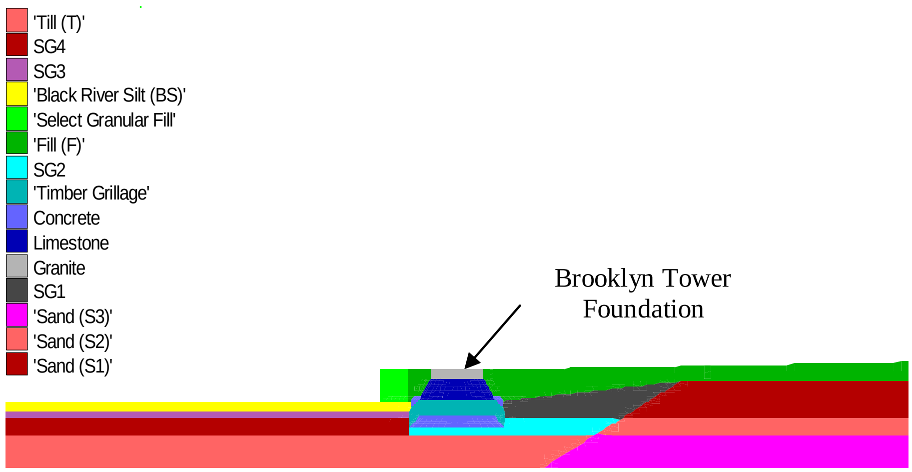


Fig. 12 FLAC model used to compute kinematic motions and impedances for the Brooklyn Tower foundation.

In the kinematic motion calculations, the foundation of the tower was given rigid properties and the mass of the structure was excluded. The baserock motion used in FLAC was computed using the rock outcrop motion appropriate for the Brooklyn Tower location and one-dimensional site response analysis. The dynamic soil-caisson interaction analysis performed by FLAC utilized a hysteretic soil model in which at every step of time integration, the soil moduli and damping ratios were adjusted according to appropriate normalized moduli reduction and damping ratio versus shear strain relationships.

As mentioned earlier, the global analysis of the bridge was performed using variable support excitation. In such an analysis, the input motions are specified at each foundation spring and dashpot as displacement time-histories. Typical computed displacement time histories along the base and sides of the Brooklyn Tower caisson are shown in Figure 11. Similar calculations were made for the Manhattan Tower caisson. The acceleration response spectra of the computed motions for the three elevations shown in Figure 11 are compared in Figure 13. It is evident that the Brooklyn Tower caisson, because of its large base and stiff foundation soils, has little tendency to rock, and hence, all the translational motions along the sides of the caissons are very similar to the motion at its base. These displacement records were subsequently used as input in the global analysis of the bridge.

The foundation impedances for the Brooklyn Tower caisson were initially computed, as a function of frequency and estimated loads on the caissons, using FLAC and the hysteretic soil constitutive model. Typical force-displacement and moment-rotation hysteresis loops at two levels along the side and at the base of the caissons were computed by applying sinusoidal forces and moments at the center of gravity of the caissons. The amplitudes of the forces and moments, as well as the frequency of excitation, were varied to capture the effect of soil non-linearity and frequency dependency of the caisson responses.

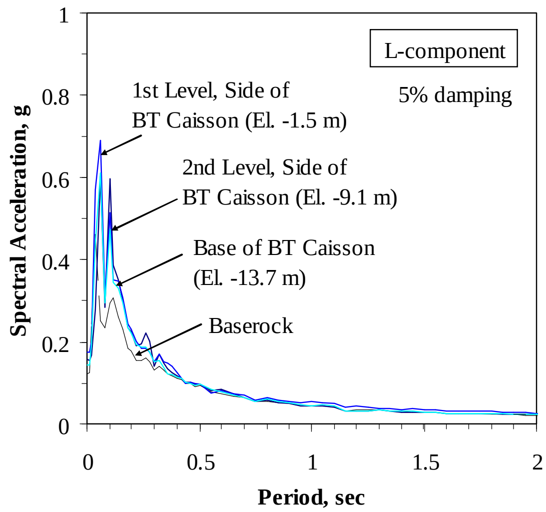


Fig. 13 Comparison of the spectra of the motions computed by FLAC for the sides and base of the Brooklyn Tower caisson.

Figure 14 shows typical results where an estimated seismic force of 391,424 kN (88,000 kips) was applied with a frequency of 10 Hz. The results show that the primary resistance to lateral inertial forces from the tower and caisson come from the base of the caisson (El. -13.7 m).

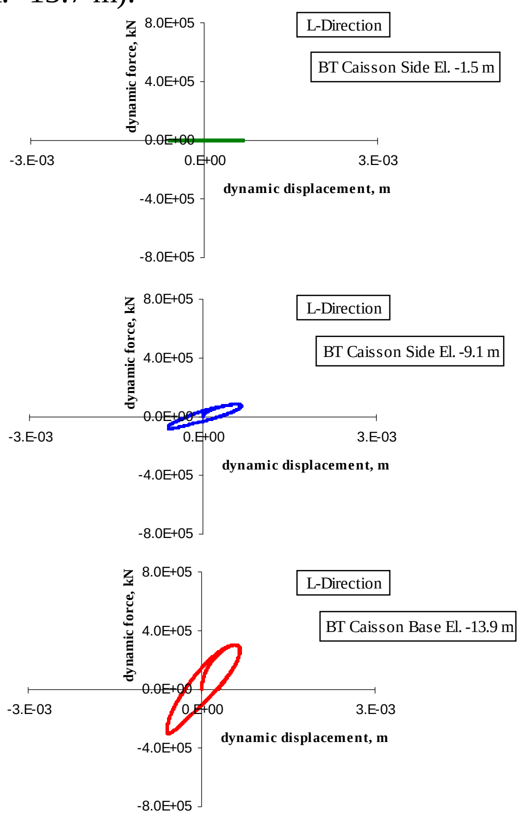


Fig. 14 Brooklyn Tower caisson force-displacement loops along the sides and base of the caisson for an estimated caisson longitudinal inertial force of 391,424 kN (88,000 kips).

Figure 15 shows the total foundation impedances of the Brooklyn Tower caisson under two levels of lateral forces. The backbone curve of the hysteresis loop shows only slight effect of soil non-linearity up to an estimated upper bound shear force induced in the caisson of 778,400 kN (175,000 kips).

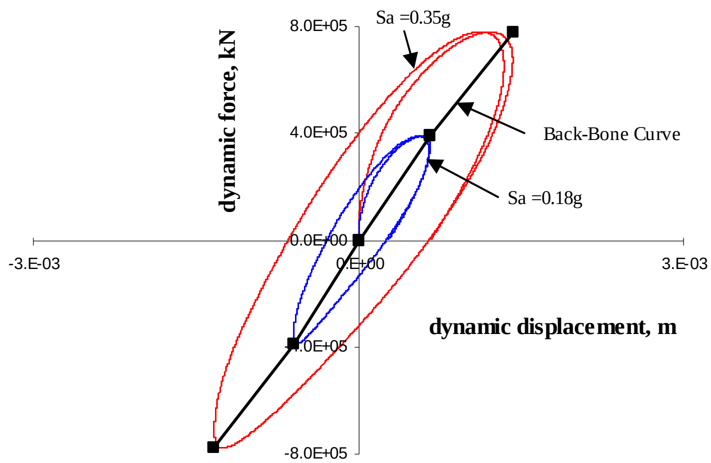


Fig. 15 Brooklyn Tower caisson total force-displacement loops for two levels of caisson inertial forces.

Using such hysteresis loops along the sides and base of the tower caisson, equivalent stiffness and damping coefficients were calculated. Table 2 summarizes the total stiffness and damping coefficients for the Brooklyn Tower caisson.

Table 2. Summary of soil-Brooklyn Tower caisson dynamic impedances, computed using SASSI.

` of Vibration	Applied Load on Caisson	Stiffness Coefficient K	Damping Coefficient C	Damping Ratio
Longitudinal, L	391,424 kN			
Side of caisson El. -1.5 m		K_L (kN/m) = 1.07E+07	C_L (kN-sec/m) = 8.70E+04	
Side of caisson El. -9.1 m		K_L (kN/m) = 1.07E+08	C_L (kN-sec/m) = 8.70E+05	
Base of caisson El. -13.7 m		K_L (kN/m) = 4.19E+08	C_L (kN-sec/m) = 3.39E+06	
Total		5.37E+08	4.35E+06	0.25
Transverse, T	391,424 kN			
Side of caisson El. -1.5 m		K_T (kN/m) = 2.23E+07	C_T (kN-sec/m) = 4.05E+04	
Side of caisson El. -9.1 m		K_T (kN/m) = 1.07E+08	C_T (kN-sec/m) = 1.94E+05	
Base of caisson El. -13.7 m		K_T (kN/m) = 3.67E+08	C_T (kN-sec/m) = 6.66E+05	
Total		4.96E+08	9.00E+05	0.06
Vertical, V	222,400 kN			
Side of caisson El. -1.5 m		K_V (kN/m) = 1.42E+07	C_V (kN-sec/m) = 1.46E+03	
Side of caisson El. -9.1 m		K_V (kN/m) = 2.85E+07	C_V (kN-sec/m) = 2.92E+03	
Base of caisson El. -13.7 m		K_V (kN/m) = 1.38E+09	C_V (kN-sec/m) = 1.42E+05	
Total		1.42E+09	1.46E+05	0.00
Rotation about L	2.60E+7 kN-m	K_{RL} (kN-m/rad) = 4.15E+11	C_{RL} (kN-m-sec/rad) = 2.78E+08	0.02
Rotation about T	2.60E+7 kN-m	K_{RT} (kN-m/rad) = 1.75E+11	C_{RT} (kN-m-sec/rad) = 6.89E+08	0.12

These total coefficients were subsequently distributed to 25 springs along the base of the caisson and 20 springs at each of two elevations along the sides of the caisson. The distribution was made ensuring the cumulative total stiffness and damping of the individual springs along the sides and base of a caisson matched the total stiffness and damping coefficients shown in Table 2. These foundation impedances were used in a preliminary seismic global analysis of the bridge to estimate the inertial loads on the caissons of the towers, and to make a preliminary assessment of the retrofit need of the tower caissons.

To account for the effect of potential sliding and tilting of the tower caissons on the non-linear soil-caisson response, the FLAC analyses were repeated using models that included the entire tower, and the tower caisson with its surrounding soils. In these models, slip and gap elements were included along the soil-caisson interfaces. The analysis involved first applying gravity to compute the initial stresses within the interface elements. Then, forces and moments were applied on the caissons, one direction at a time (pushover analysis) and the displacements and rotations of the caisson were computed.

Figure 16 shows the entire model of the Brooklyn Tower and its caisson in which slip and gap elements were included.

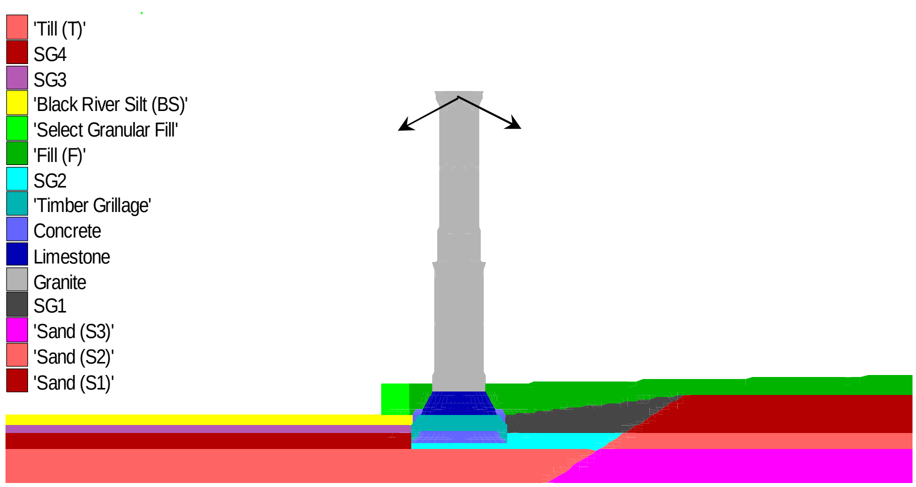


Fig. 16 Longitudinal model of the Brooklyn Tower and its foundation used in FLAC to compute non-linear force-displacement and moment-rotation relationships for the caisson, which included interface elements along its sides and base.

Also included in the model were static and equivalent dynamic cable forces and deck loads on the tower that were computed by the initial global analysis of the bridge. The properties of the interface elements included: the friction angle of the cohesionless soils, the undrained shear strength of the clay, and the normal and shear stiffness of the interface elements, which were based on the shear modulus of the soil and the dimensions of the soil elements adjacent to the interface elements.

The moduli of the timber grillage and the limestone of the tower foundation were measured in the field using the geophysical technique of shear and compression wave tomography. Figure 17 shows the results obtained using two boreholes drilled through the Brooklyn Tower foundation. On average, the shear and compression wave velocities of the timber grillage were 823 mps (2700 fps) and 1737 mps (5700 fps), respectively. The corresponding values for the limestone and granite blocks were 2652 mps (8700 fps) and 4145 mps (13,600 fps), respectively.

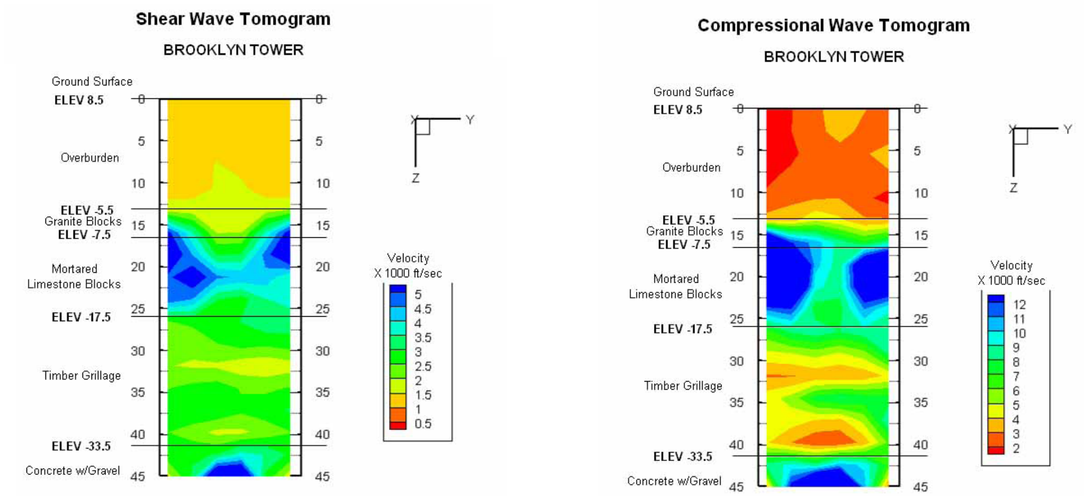


Fig. 17 Shear and compression wave tomography results of the timber grillage, and the limestone and granite blocks of the Brooklyn Tower foundations.

Figure 18 shows a typical force-displacement curve obtained for the Brooklyn Tower caisson, in the longitudinal direction. The solid curve represents the total stiffness of the caisson. The dashed curves show the relative contributions from the sides and base of the caisson.

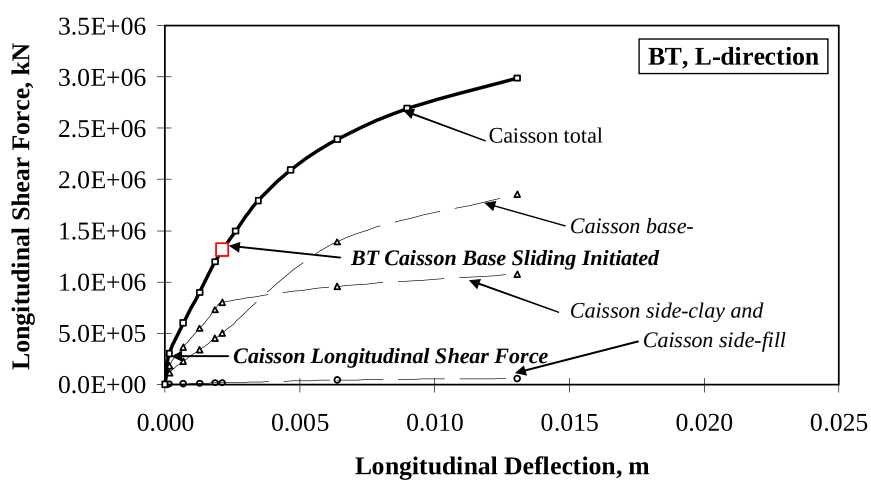


Fig. 18 Non-linear force-displacement relationship in the longitudinal direction for the Brooklyn Tower caisson, computed from the pushover analysis of the caisson, using FLAC with interface elements.

A number of observations are made from the results shown in Figure 18. Sliding of the caisson is initiated at a total shear force of about 1,334,400 kN (300,000 kips) acting at the center of gravity of the caisson. This is far larger in magnitude than what was computed 146,784 kN (33,000 kips) from the initial global analysis using foundation impedances. Hence, sliding of the caisson under the design event is not likely to occur, a conclusion based on the non-linear force-displacement relationships and the results from the global analysis. This conclusion is later confirmed by the results from the local analysis. Also, at small levels of shear force, the total stiffness is linear and the primary resistance to the caisson inertial forces comes from its base, conclusions that are consistent with those arrived at from the frequency dependent impedance analysis of the caisson (Figures 14 and 15).

Figure 19 shows the transverse moment versus rotation curve of the Brooklyn Tower caisson. Again, the results show that gapping will be initiated along the base of the caisson only if the transverse moment exceeds 7.5×10^6 kN-m (5.5×10^6 kip-feet), which is much greater than what was computed from the initial global analysis 4.7×10^6 kN-m (3.5×10^6 kip-feet) using frequency dependent foundation impedances. Hence, the Brooklyn Tower caisson is not expected to separate from its base during the 2500-year event, a conclusion later confirmed using the local analysis of the tower and its foundation. Similar calculations using the Manhattan Tower caisson led to the same conclusions, that the caisson is safe against sliding and separation.

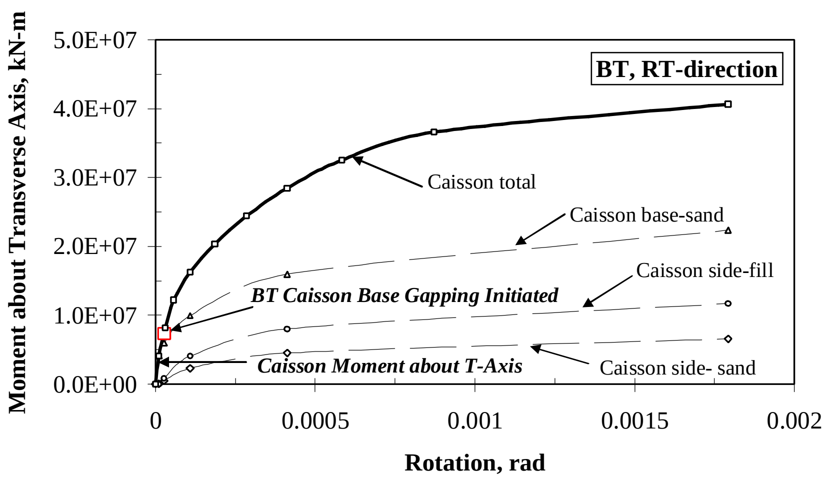


Fig. 19 Non-linear moment-rotation relationship about the transverse axis for the Brooklyn Tower caisson, computed from the pushover analysis of the caisson, using FLAC with interface elements.

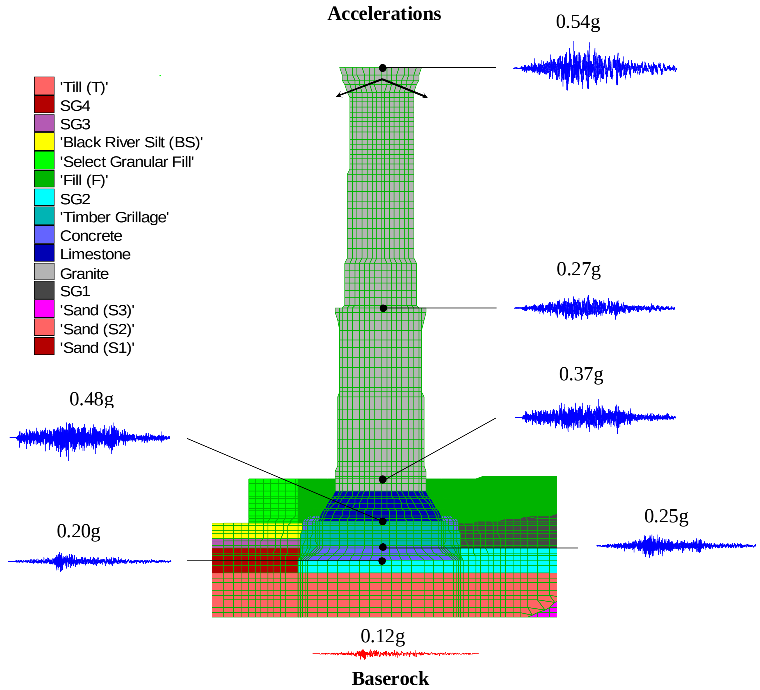
A total of three translational force-displacement and three moment–rotation curves were generated for each tower caisson using FLAC and interface slip and gap elements. These total stiffness curves were then distributed to the base and side springs and the global analysis was repeated for the final results. In ADINA, these curves were used as initial loading backbone curves for base shear tractional and for normal contact springs. Unloading of tractional springs was considered through the use of full Masing hysteresis, and of normal contact springs through the use of initial tangent stiffness. Later in this paper, selected results from the global analyses will be compared with corresponding results from the local analysis, which is described in the next section.

SOIL-FOUNDATION-TOWER ANALYSIS (LOCAL ANALYSIS)

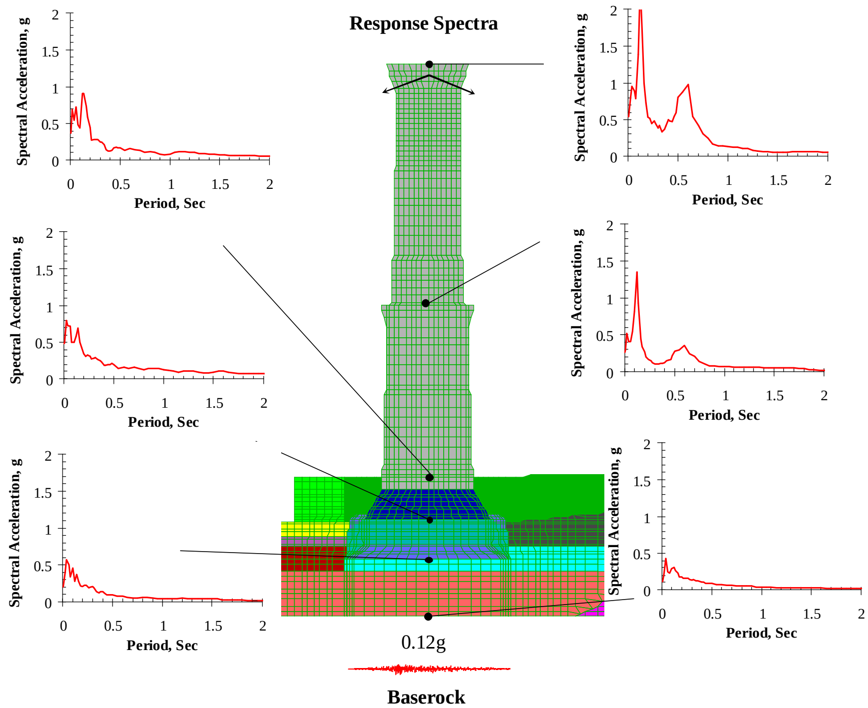
The Brooklyn Bridge towers are massive and rigid, while its superstructure is flexible in comparison with the towers. Furthermore, the design rock motions are rich in high frequencies and have little energy in the low frequency range. Therefore, it is quite reasonable to expect that the dynamic inertial loads from the deck and the dynamic component from the cables will make only a small contribution to the seismic response of a tower and its caisson. This expectation was clearly observed in the global analysis of the bridge. Hence, it was of interest to perform a local seismic analysis of each of the two towers with their caissons and surrounding soils using FLAC. Such an analysis avoided the various assumptions made in the calculations of the kinematic motions and foundation impedances and provided added benefits of including initial stresses, more accurate modeling of the soil non-linear behavior, and computing the stress distributions within the caisson as well as along its sides and base.

The Brooklyn Tower caisson model shown in Figure 16 was further used to investigate the vulnerability of the caisson. It also included static and equivalent dynamic cable forces, and hydrostatic effects. For the soils the hysteretic soil model was utilized in which the soil moduli and damping ratios were adjusted at every step of time integration based on parameters that approximated appropriate normalized moduli versus shear strain curves. The soil-caisson-tower model first was subjected to gravitational loads and all the initial total and effective normal stresses were calculated and saved within FLAC. The model was then subjected to the 2,500-year baserock horizontal and vertical motions used earlier in the calculations of the kinematic caisson motions. The time histories of acceleration, displacement, shear stress, and vertical normal stress were computed at various nodes of interest including at the top and bottom of the interface elements. The results were then processed to evaluate the response of the tower and its foundation.

Figures 20a and 20b present summary plots of the accelerations and response spectra at various nodes within the longitudinal model of the Brooklyn Tower and its foundation, under longitudinal earthquake excitation. Figures 21a and 21b present similar results within the transverse model.

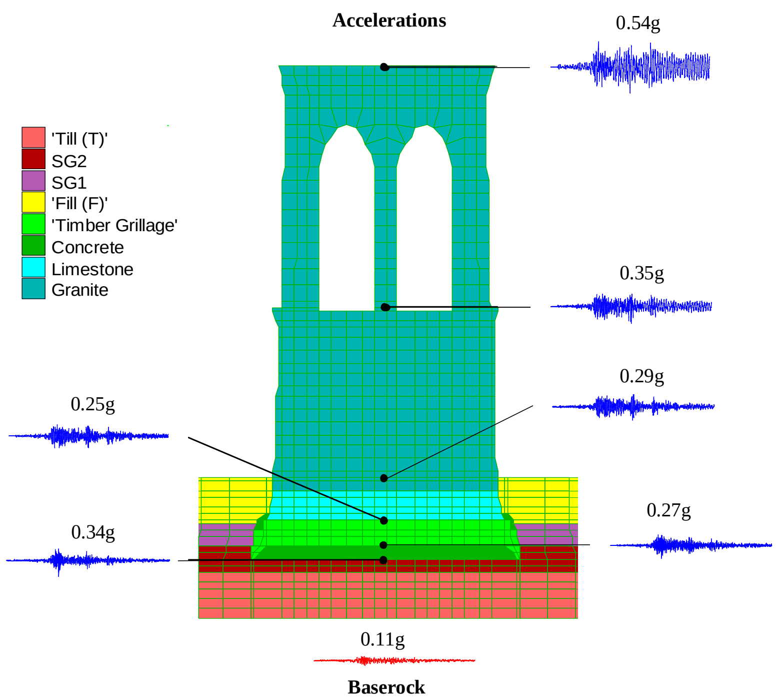


(a)

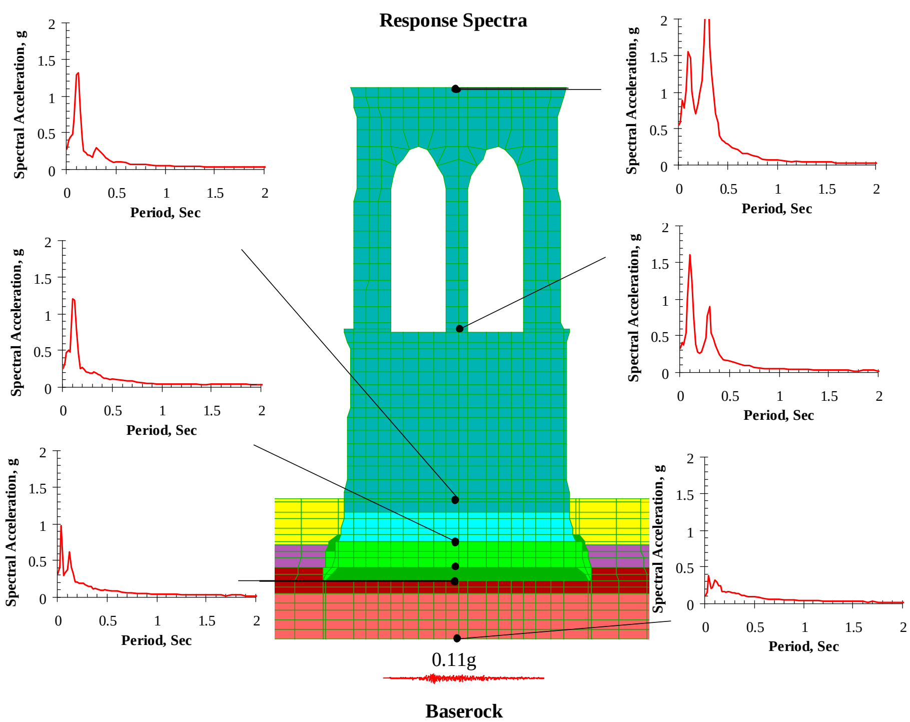


(b)

Fig. 20 Longitudinal responses of the Brooklyn Tower and its foundation, a) acceleration time histories, b) corresponding response spectra, under the 2,500-year event.



(a)



(b)

Fig. 21 Transverse responses of the Brooklyn Tower and its foundation, a) acceleration time histories, b) corresponding response spectra, under the 2,500-year event.

These results show that the baserock motion amplifies as it propagates through the structure. Figure 20b shows amplification at two particular periods, 0.15 seconds and 0.7 seconds. Figure 22 shows spectral acceleration ratios from the longitudinal model obtained by dividing the spectra at various elevations within the tower and its caisson with the spectrum at the base of the caisson.

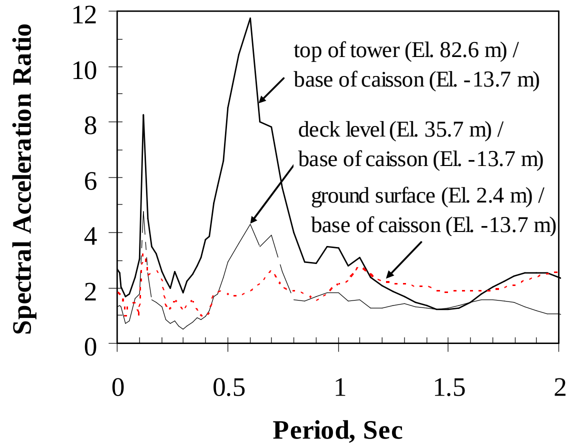


Fig. 22 Ratios of the longitudinal spectral accelerations, relative to the caisson base.

Two modal frequencies can be seen clearly to occur at about 0.12 seconds and 0.6 seconds. Simple calculations of the horizontal and rocking periods of the tower and its caisson confirmed that the first period corresponded to the longitudinal period of the tower and the second is most likely associated with the rocking period about the transverse axis of the bridge. These modal periods were also within the period ranges that were observed through ambient vibration measurements of the bridge.

Figures 23a and 23b show the horizontal and vertical displacement histories at the top and bottom of the interface elements at the base of the caisson, respectively under the longitudinal earthquake excitation of the 2500-year event. The top and bottom displacements are exactly identical, indicating that the interface elements do not exhibit slip or gapping along the base of the tower.

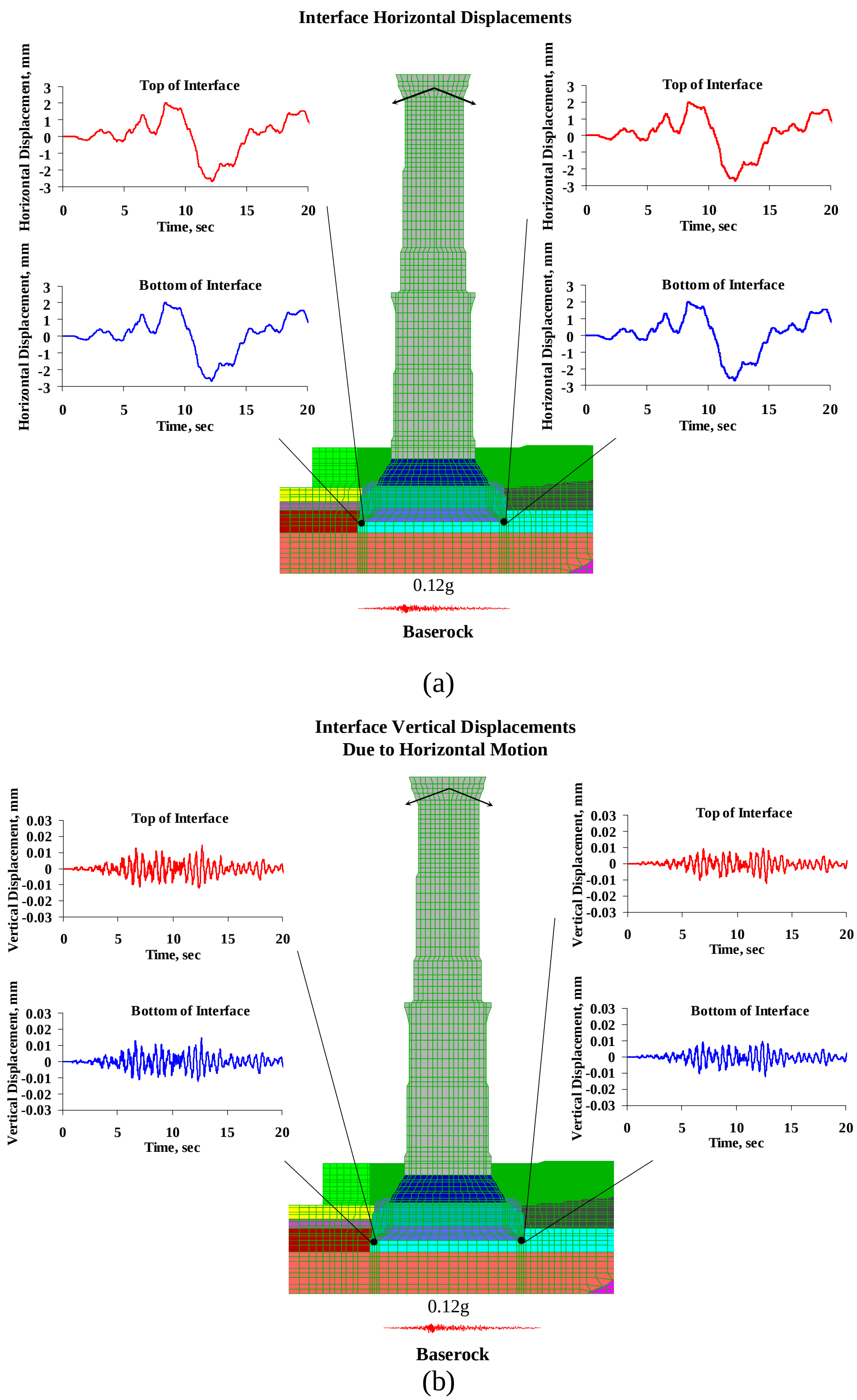


Fig. 23 Displacements at the top and bottom of the interface elements along the base of the Brooklyn Tower caisson, under the 2,500-year event, a) horizontal displacement, b) vertical displacement.

Figure 24 displays the vertical displacements at the top and bottom of the caisson base interface elements under the combined horizontal (longitudinal) and vertical motions. Even under this most severe condition, when the vertical motion can potentially reduce the base stresses, the vertical displacements at the top and bottom of the interfaces are exactly the same, indicating that there is no gapping (loss of contact) along the base of the caisson.

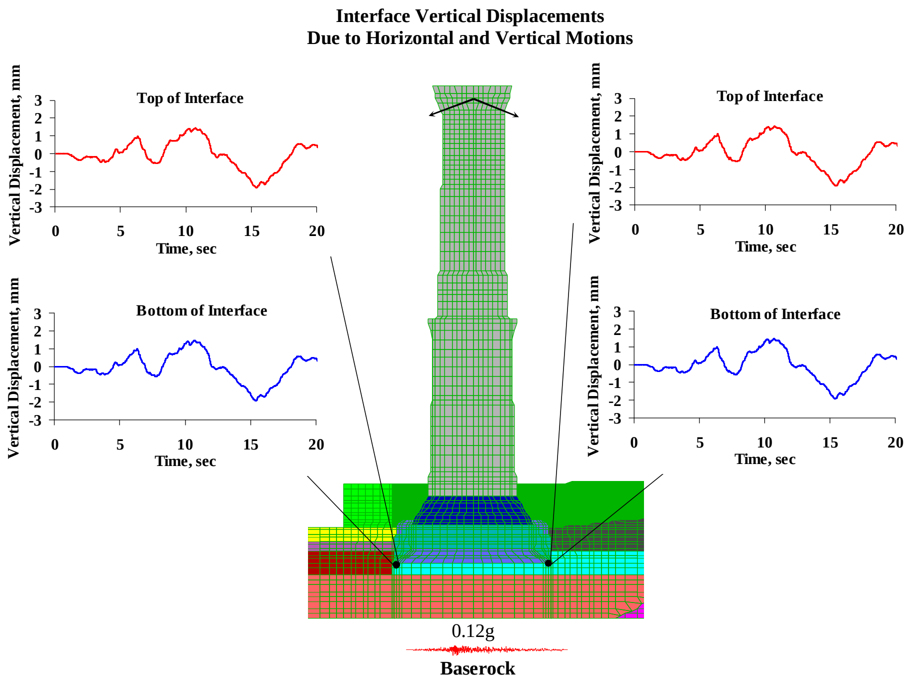


Fig. 24 Comparison of vertical displacements at the top and bottom of the interface elements along the base of the Brooklyn Tower caisson, induced by the combined longitudinal and vertical motions of the 2500-year event.

Figure 25 shows a summary of the initial static and dynamic shear stresses along selected cross sections within the Brooklyn Tower and its caisson. The maximum shear stress in the concrete of the caisson is about 310 kPa (6.5 ksf) and in the timber grillage is about 345 kPa (7.2 ksf). These values are significantly smaller than the shear capacities that were measured in the laboratory for the concrete and timber specimens.

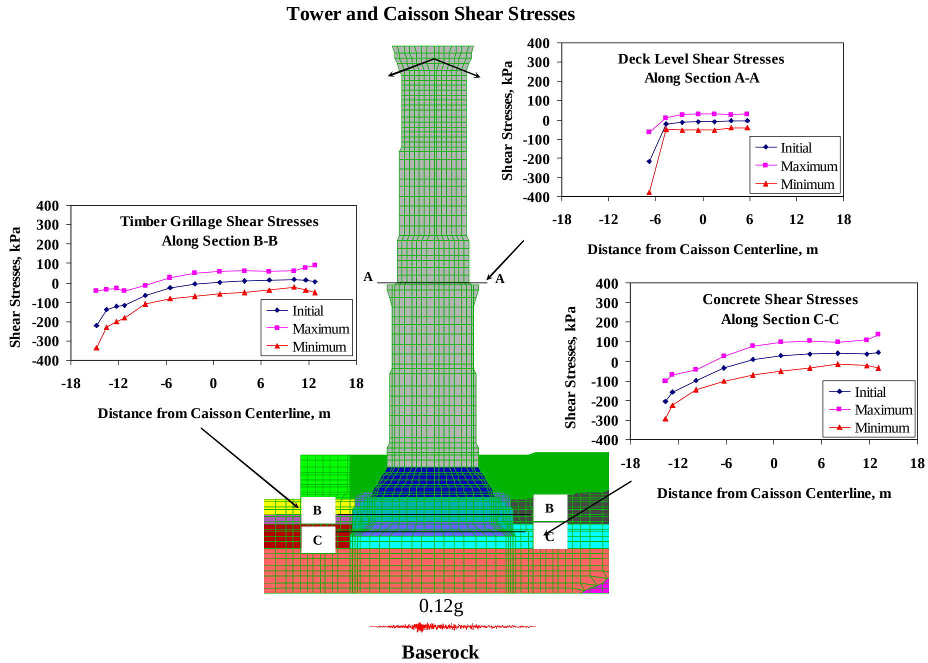


Fig. 25 Longitudinal shear stresses computed using the FLAC model of the Brooklyn Tower and its foundation, under the 2500-year event.

The total shear force time history along a cross section through the middle of the caisson (Section B-B) was computed by integrating the shear stress time histories along the cross section. The result is shown in Figure 26. This total shear force time history, obtained from the local analysis, is compared later in this paper with the results from the global analysis.

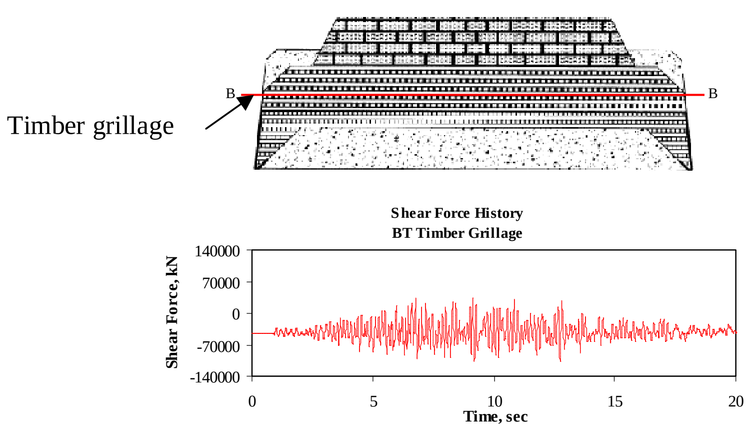


Fig. 26 Total longitudinal shear force history along cross section B-B in the middle of the timber grillage of the Brooklyn Tower caisson.

The maximum value of the total shear force within the caisson is about 88,960 kN (20,000 kips) as shown in Figure 26. Under such a magnitude of shear force, the caisson is not expected to slide or tilt as was demonstrated through the use of the force-displacement and moment-rotation curves described in the previous section of this paper.

Figure 27 shows the shear and effective normal stresses along the base of the caisson induced by gravity and the horizontal earthquake motion, without the vertical component of excitation.

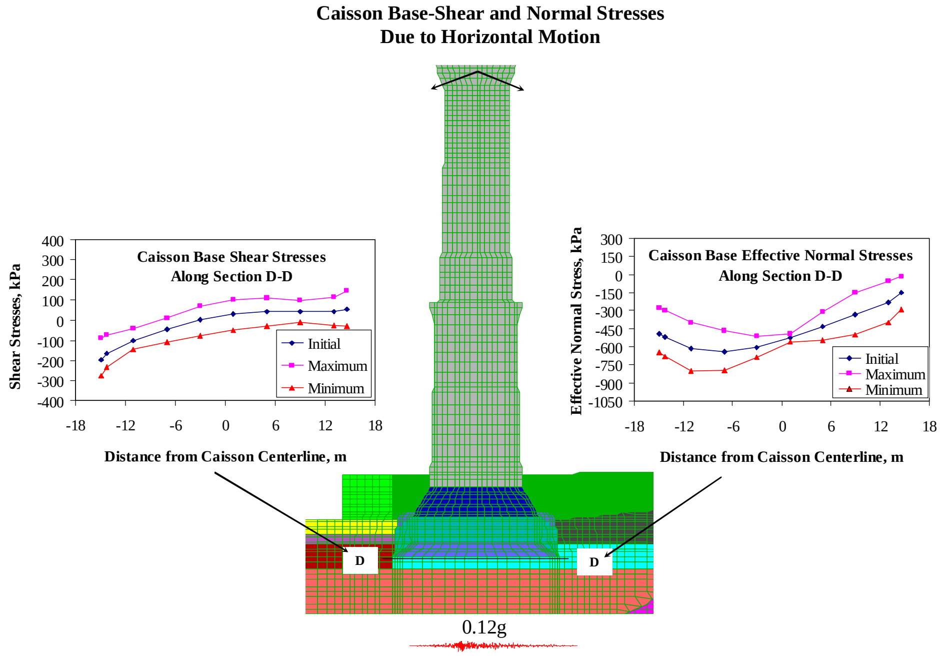


Fig. 27 Brooklyn Tower caisson base shear and effective normal stresses induced by the longitudinal motion of the 2500-year event.

Initially, under gravity, the effective normal stresses are negative (compression) with values larger to the right of the centerline of the tower due to the larger cable forces in the direction of the center span of the bridge (to the left when looking at Figure 27). Under the transverse moment induced by the longitudinal horizontal excitation, the maximum compression increases to about 827 kPa (17 ksf) to the left of the tower centerline and decreases to about 0 kPa to the right of the tower centerline.

Figure 28 shows the shear and effective vertical stresses under the combined horizontal and vertical excitations. Under this load combination, the maximum effective normal stress slightly increases to about 862 kPa (18 ksf) and the minimum effective normal stress remains at about 0 kPa.

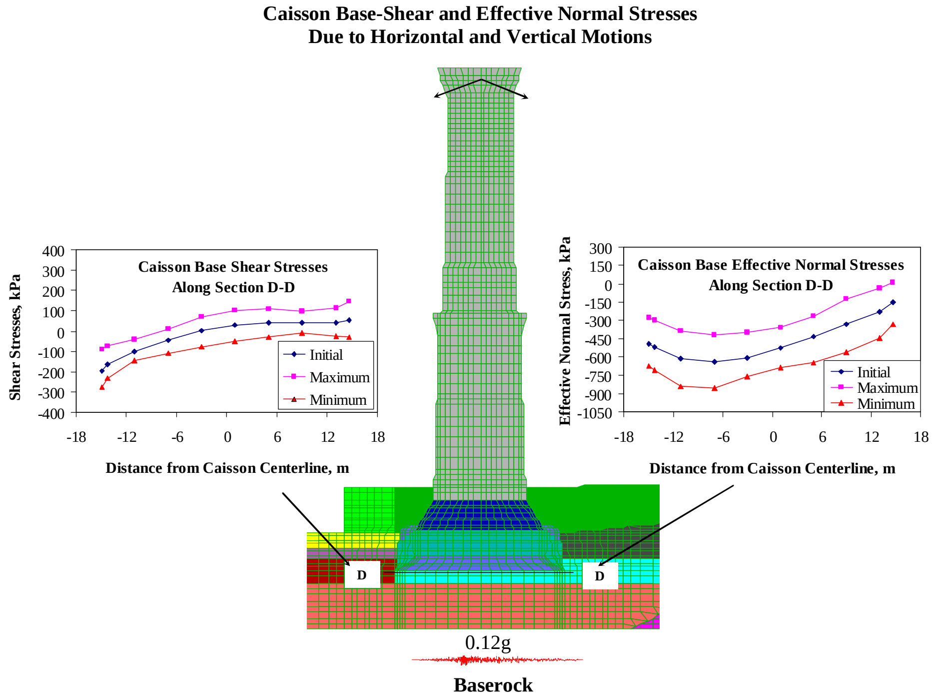


Fig. 28 Brooklyn Tower caisson base shear and effective normal stresses induced by the combined longitudinal and vertical motions of the 2500-year event.

In summary, seismic longitudinal and vertical analyses of the Brooklyn Tower caisson and the surrounding soils led to the conclusion that the effective normal stresses at the bottom of the caisson are small 8.62 kPa (18 ksf) relative to the ultimate capacity >4,788 kPa (100 ksf). Additionally, the caisson is safe against sliding and will not lose contact with its base soils under the longitudinal and vertical components of the 2500-year earthquake.

Similar investigations of the Brooklyn Tower and its caisson were made considering seismic excitation in the transverse direction. The conclusions were identical to those inferred from the longitudinal analyses. A typical result is shown in Figure 29 which depicts the caisson base shear and effective normal stresses under the combined horizontal and vertical motions. Under this load combination, the maximum effective normal stress is about 758 kPa (15.8 ksf) and the minimum effective stress is at about 69 kPa (1.4 ksf) leading to the conclusion that the caisson is safe against sliding and bearing capacity type failure, and will not lose contact with its base soil under the transverse and vertical excitations induced by the 2500-year event.

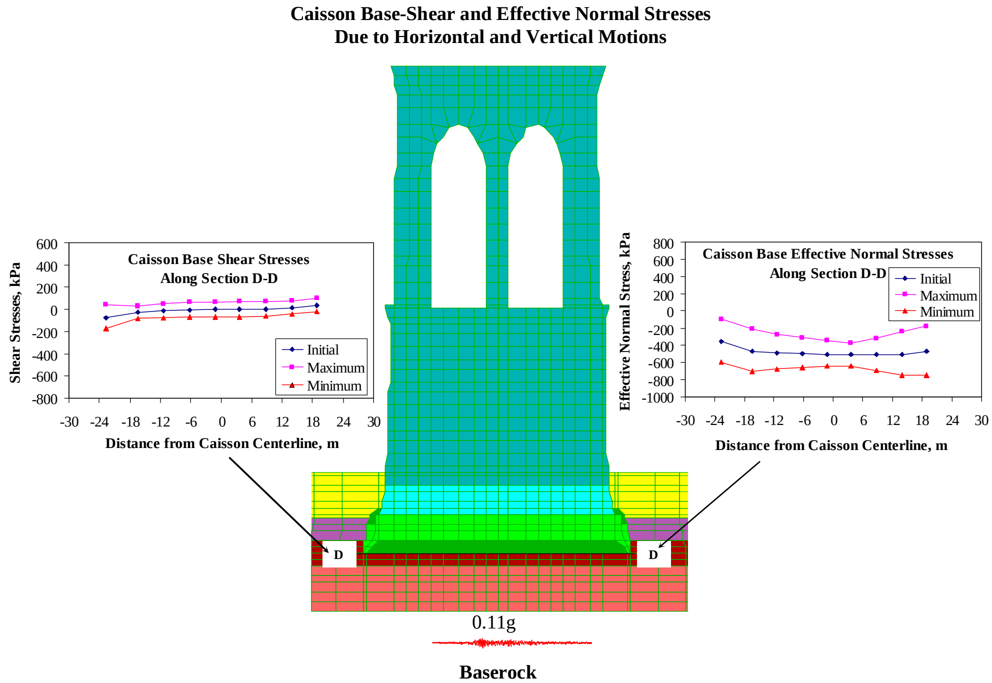


Fig. 29 Brooklyn Tower caisson base shear and effective normal stresses induced by the combined transverse and vertical motions of the 2500-year event.

COMPARISONS OF GLOBAL AND LOCAL ANALYSIS RESULTS

As described earlier, the global analysis of the Brooklyn Bridge incorporated the entire bridge including the towers, cables, suspended structure and foundations. Thus, it provided the means to consider the cable effects and the masonry tower potential for cracking. The caissons were modeled using beam elements, which permitted the calculation of stresses at only a few selected locations where the springs were placed.

The local analysis that involved the investigation of the seismic interaction of the bridge tower with its foundation and surrounding soils permitted more accurate considerations of the non-linear soil caisson interaction as well as the direct consideration of the potential slip and gapping around the caissons. The local analysis also provided a more detailed distribution of stresses that included initial effective vertical normal stresses, and the shear stresses within the tower and its foundations.

It is of interest to compare selected results from both the global and local analyses. Figure 30 shows a comparison of the total shear force time history in the longitudinal direction along cross section B-B of the Brooklyn Tower caisson. The results from both analyses are quite comparable both in intensity and general frequency content.

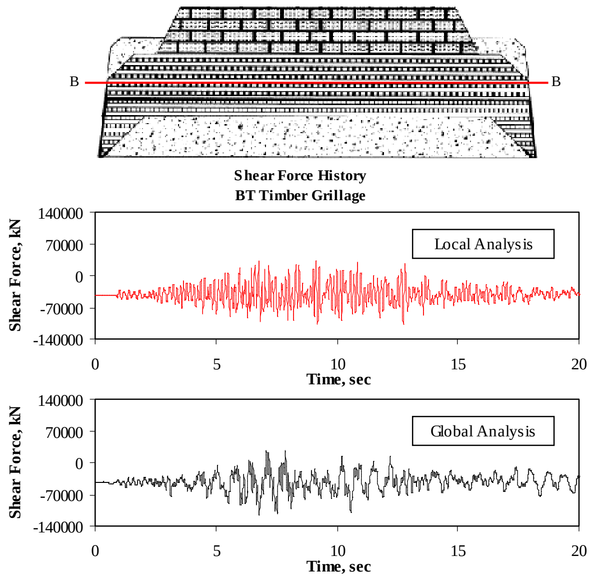


Fig. 30 Comparison of longitudinal total shear force time histories along cross section B-B of the BT caisson, from the local and global analyses.

In Figure 31, a comparison is made of the effective vertical normal stress along the base of the Brooklyn Tower caisson obtained from the global and local analyses. Again, the agreement is quite good considering the wide differences in the analysis approaches.

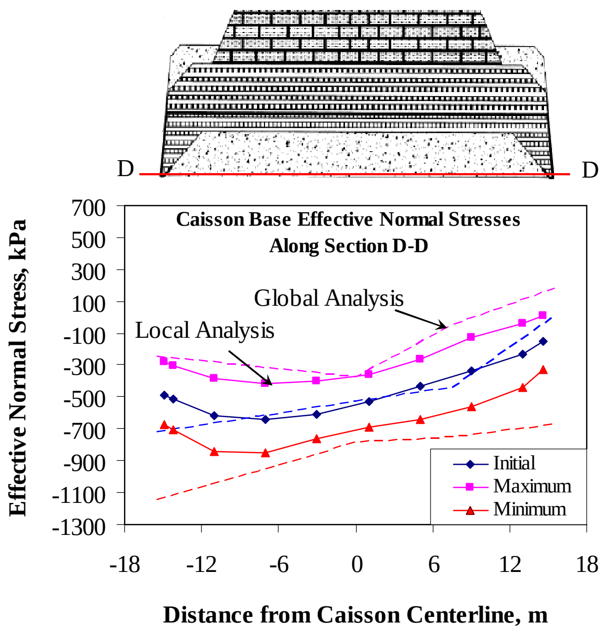


Fig. 31 Comparison of effective vertical normal stresses along the base of the Brooklyn Tower caisson obtained from the local and global analyses.

Finally, in Figure 32 a comparison is made of the drift of the Brooklyn Tower normalized with respect to the displacement at the base of the tower caisson.

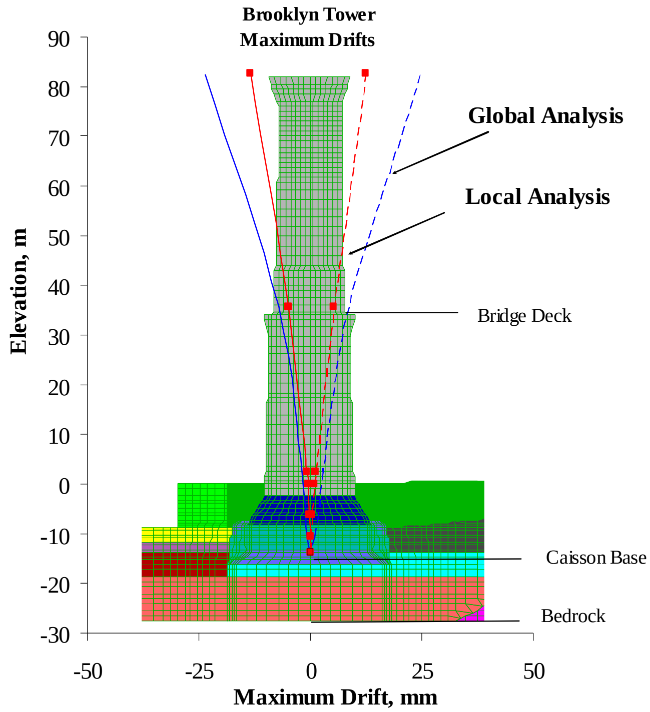


Fig. 32 Comparison of drifts along the Brooklyn Tower caisson and its foundation, normalized with respect to the drift at the base of the caisson, from local and global analyses.

It is noted that these drifts are maximum values and do not necessarily occur at the same time. The drift values from the local and global analyses are small and comparable with the global analysis results.

Comparisons of the results obtained from the analyses of the Manhattan Tower and its foundation led to the same conclusion, that the local and global analyses yield similar results and the main tower foundations are adequate to safely resist the 2500-year event without experiencing sliding or lift-off along its base, nor bearing capacity failure, and hence do not require retrofitting.

CONCLUSIONS

Seismic investigation of the historic Brooklyn Bridge was performed to assess its potential need for retrofitting. The bridge serves a critical transportation need in New York City, and very importantly is a national landmark and a world recognized architectural and engineering achievement. The seismic assessment of the bridge was completed using the most advanced engineering investigations to ensure that the evaluation of retrofit needs were based on a rational framework and avoided "pitfalls" (as described by Peck, 1977) of overconservatism, including implementation of unnecessary retrofit schemes which may negatively impact the architecture of the bridge.

Two approaches were followed to determine the soil-foundation-bridge interaction, namely, **global** and **local analyses**. In the global analysis model, the soil-foundation interaction was introduced through the use of non-linear hysteretic springs with gapping features and dashpots. In the local analysis, each of the towers with their foundation caissons and the surrounding soils were investigated. The local analysis models included hysteretic soil behavior as well as interface slip and gap elements. Comparisons of various results obtained from the global and local analyses showed satisfactory agreement and led to the same conclusion, that the foundations of the Brooklyn Bridge under the 2500-year design event do not require retrofitting.

Based on extensive seismic evaluations of the Brooklyn Bridge, using global and local analytical approaches, the following observations and conclusions are drawn.

1. The Brooklyn Bridge is a long span bridge with massive cable anchorages and towers. The superstructure contributes very little to the tower dynamic responses. The global analysis has shown that, for the level of seismic loads in the New York City metropolitan area, very little cracking of the masonry towers is anticipated. Thus, the local analysis of the towers with their foundation caissons yielded dynamic responses of the towers and the caissons very similar to those obtained following the current state of practice of seismic analysis for critical bridges (global analysis).

2. The agreement in the results between the global and local analyses is a confirmation of the quality of the kinematic motions and the foundation impedances used in the global analysis of the bridge as well as a confirmation of the validity of the caisson modeling approach in the global analysis.

3. Quality kinematic motions and foundation impedances were computed following advanced soil-structure interaction analysis procedures. Such procedures, considered the three-dimensional kinematic effect of the foundations on the ground motions, and the non-linear force-displacement and moment-rotation stiffness relationships that included the effect of potential slip and gapping along the sides and bases of the caissons of the towers. These non-linear stiffness curves were obtained by performing pushover analyses of the foundation caissons. If ground motions and foundation impedances were computed using more simplified analytical procedures, the caisson and tower responses computed by the global analysis would not have been in agreement with those obtained from the local analysis.

4. The local analysis provided the advantage of considering the effect of the initial static tower and soil stresses, accounting for the non-linear soil response directly in the computations, modeling more accurately the potential

sliding and gapping of the tower caissons, and computing caisson static plus dynamic internal and external stresses as well as the stresses in the tower structure.

5. In suspension bridges, seismic response of bridge towers with large foundation caissons can be reliably evaluated following a local analysis in which the bridge support components with the soil continuum are considered together in a single model.

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ACKNOWLEDGMENTS

The authors acknowledge the contributions of Thomas Thomann of URS to the geotechnical field and laboratory investigations, and Paul Fisk of NDT to the geophysical field investigations. Also acknowledged are the following personnel from the New York City Department of Transportation: Henry Perahia, Deputy Commissioner, Division of Bridges, Seth Solomonow, Assistant Commissioner, Press Office, Walter Kulczycki, Project Manager, East River Bridges, Division of Bridges, and Jaktar Khinda, Project Seismic Engineer, Division of Bridges.