SOIL-FOUNDATION-STRUCTURE INTERACTION AND DESIGN OF DEEP FOUNDATION AGAINST LATERAL SPREAD

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Abstract

This paper describes the analytical models used to account for the soil-foundation-structure interaction effects for two major long span bridges in the US. The first one is a recently completed cable-stayed bridge founded on 10-ft (3 m) diameter drilled shafts embedded as deep as 200 ft (61 m) below the mudline. The second one is an existing suspension bridge with a main span of 4,260 ft (1,299 m) supported on deep gravity type caissons. Both bridges are the longest in their respective category in the North America. In both cases, the soil-foundation system were explicitly modeled in the inelastic time history analysis with spatially varying ground motions applied not only at different pier foundation locations but also at varying elevations along the vertical axis of each foundation. The design approach used for the drilled shaft foundation against liquefaction-induced lateral spread movements is also presented.

Introduction

It is well recognized that the dynamic characteristics of a foundation system (including its stiffness and capacity) influence the magnitude of the dynamic response and hence the earthquake load demands on the bridge structure. For pile or drilled shaft foundations, there are several conventionally used analytical procedures to account for the soil-foundation-structure interaction effects, including (a) 6x6 stiffness matrix method with near surface free-field ground motions as seismic input, (b) equivalent cantilever method with near surface free-field ground motions as seismic input, and (c) uncoupled substructure method using 6x6 impedance matrix and effective support motions derived from soil-foundation interaction analysis. In these methods the total structure is divided (uncoupled) into two substructures: the superstructure and the foundation, with an interface introduced between the two at the top of the foundation to represent the foundation stiffness.

To provide a realistic design (or retrofit), it generally requires a more detailed and higher level of analysis, particularly for important bridges. When significant non-linear behaviors in soils or inelastic deformations in the foundation elements are expected, it is not sufficient to use a simple linear stiffness/impedance matrix to represent the foundation characteristics. Instead, it would be most desirable to develop a fully coupled analytical model of the entire system including the superstructure, piers/columns, and the detailed soil-foundation system in a time-history analysis. As a minimum, such a model

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should account for the spatially varying and depth varying soil springs as well as ground motions to properly capture both the inertial and kinematic interaction effects. It is also important to include the non-linear load-displacement characteristics of the soil springs and individual foundation elements (e.g., piles/drilled shafts) so as to capture not only the hysteretic damping behavior, but also the influence of mobilization of foundation capacity on bridge performance (if inelastic deformations of the foundation elements are allowed). Two case studies of the fully coupled model are presented herein to illustrate the explicit incorporation of soil-foundation-structure interaction in the performance-based seismic design approach. The foundation systems used in the two bridge cases are completely different. The new Cooper River Bridge, located in Charleston, South Carolina, is founded on large diameter drilled shafts, while the 40-year old Verrazano-Narrows Bridge in New York was built by using deep gravity type caissons to support its two towers and large footings for its two anchorages.

Another area of importance is the lateral movement of liquefied soil against piles or drilled shafts. Liquefaction-induced lateral spread was considered in the design of the drilled shaft foundations for the new Cooper River Bridge. The design approach used for this case study as well as that recommended by a recent research program NCHRP 12-49 (MCEER/ATC, 2003) will be discussed in this paper.

Soil-Foundation-Structure Model of Cooper River Bridge

The recently completed Cooper River Bridge in Charleston South Carolina is located within one of the most seismically active regions in the eastern US. The design safety evaluation earthquake is a 2,500-year return period event with a moment magnitude of 7.3 and seismic shaking intensity similar to that in portions of California (a Peak Ground Acceleration of 0.65g).

The bridge has a main span of 1,546 ft (471 m) and a cable-supported span length of 3,296 ft (1005 m), with a 10,441 ft (3,182 m) of high and low level approach structures. To address traffic and shipping demands, the new bridge will provide a 1,000 ft (305 m) horizontal and 189 ft (58 m) vertical clearance over the main channel. The two 572.5 ft (174.5 m) high diamond shaped towers support a 126 ft (38.4 m) wide deck carrying 8 traffic lanes and a 12 ft (3.7 m) pedestrian walkway/bikeway on the south side. The main span utilizes a composite concrete deck with I-shaped steel edge girders. To achieve a cost-effective foundation system high capacity drilled shafts were used to minimize the number of substructure units. Each of the main piers of the cable-stayed bridge is founded on eleven 10-ft (3 m) diameter drilled shafts, extending down to 230 ft (70 m) below the mean sea level. The piers of the high and low level approach structures are generally supported on only two columns, with a single 8-ft (2.4 m) or 10-ft (3 m) diameter drilled shaft foundation for each column.

The site is underlain by about 50 to 65 ft (15 to 20 m) of recent deposit and 250 to

300 ft (76 to 91 m) of Cooper Marl. The recent deposits consist primarily of loose (liquefiable) to medium dense sand or very soft organic marsh deposit with shear wave velocity as low as 250 ft/sec (76 m/sec). The underlying Cooper Marl is characterized as stiff to hard calcareous silty or sandy clay or clayey sand and silt (Castelli, 2004). Based on in-situ seismic testing, the upper half of the Cooper Marl has a shear wave velocity between 1,200 and 1,500 ft/sec (366 to 457 m/sec), increasing to about 2,000 ft/sec (610 m/sec) in the lower portion of the Marl stratum.

Due to the difficult soil conditions (very soft) and the expected ground motion intensity (very high) at the project site as well as the nature of the foundations (many single pier drilled shafts – very flexible and lack of redundancy), it was determined to model the superstructure and the foundation as a complete system (i.e., coupled analysis), where every essential elements of the foundation such as all the drilled shafts, footing cap, piers, and the non-linear soil springs are included in the bridge model. This model, representing about 10,000 linear feet (3,000 m) of bridge as shown in Figure 1, was supported by 68 pier columns and more than 100 drilled shafts (mostly 10-ft (3 m) diameters), and excited by about 600 sets of spatially varying ground motion time histories.

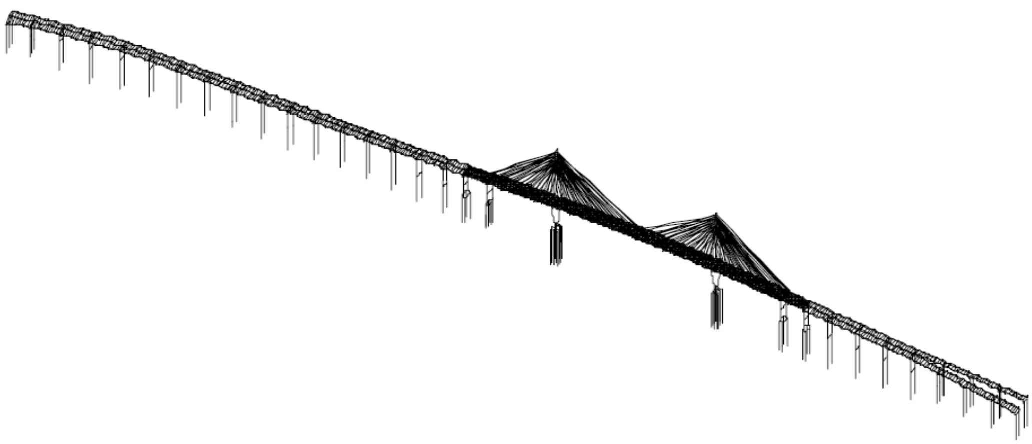

Figure 1 Global ADINA Finite Element Inelastic Time History Analysis

An inelastic time history analysis using records from three different events as part of the final design was accomplished using computer program ADINA, with post-processors to track the moment curvature behavior of those sections of the structure that may potentially undergo inelastic behavior during a seismic event. The soil-structure interaction was considered by explicitly modeling the drilled shafts as moment curvature elements supported by discrete non-linear springs. Due to the long length of the structures as well as the highly variable subsurface conditions along the bridge alignment, the effects of spatial variations of ground motions were considered for the bridge structures to account for the following effects: (a) wave-passage, (b) wave scattering/incoherency, and (c) local site response (by SHAKE analysis). Figure 2 illustrates how the spatially varying ground motions (varying in elevation as well as in horizontal direction) were applied to the support ends of the non-linear springs to account for the kinematic interaction effect.

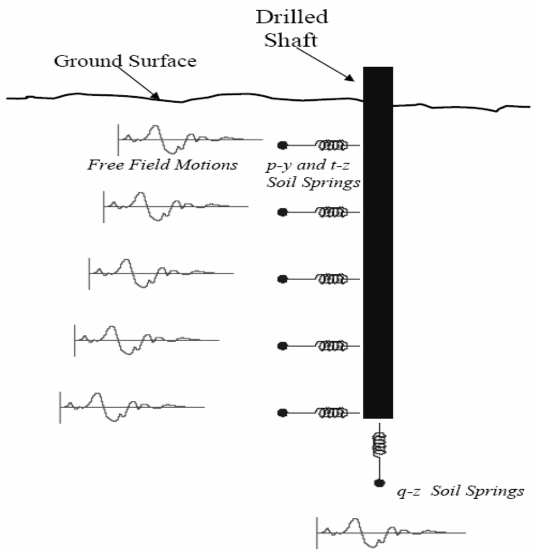

Figure 2 Kinematic Interaction with Spatially Varying Ground Motions (for Drilled Shaft Foundation)

Based on load test data, non-linear plasticity-based (hysteretic) truss elements were used for the horizontal (p-y) springs, nonlinear elastic springs for skin friction (t-z) and non-linear elastic compression only springs for the tip resistance (q-z), as shown in Figure 3.

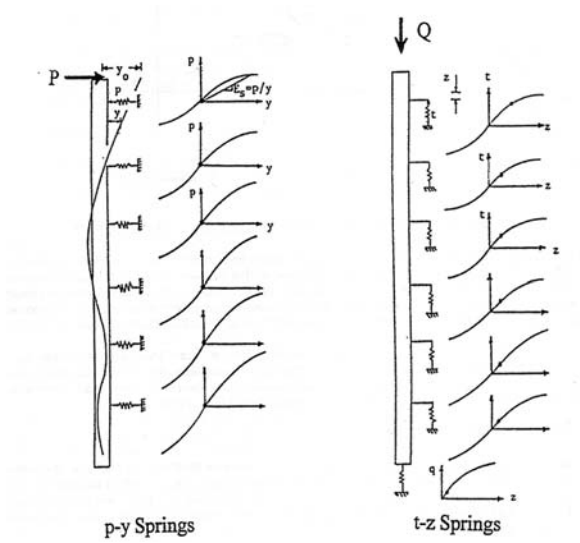

Figure 3 Plasticity-based p-y springs and Non-linear t-z and q-z Springs

Because of the importance of the bridge, design criteria required that hinging be prevented below grade in the foundations elements. The design of the drilled shafts (particularly for those single-pier drilled shaft) has in general been controlled by the plastic hinge capacity of the columns above the drilled shafts (Bryson, et al., 2003). Since most of the shafts are embedded in approximately 50 to 60 ft (15.2 to 18.3 m) of soft soils, the shafts have had their maximum moment at the level of the stiff marl just below the soft upper soil layer, particularly in area where the surficial soils liquefy (Wang, et al., 2004). This has resulted in having more reinforcing steel at the mid portion of the drilled shaft than at the top, a somewhat unusual arrangement but one dictated by the site conditions.

Soil-Foundation-Structure Model of Verrazano-Narrows Bridge

The Verrazano-Narrows Bridge spans the Narrows in New York City connecting the boroughs of Staten Island and Brooklyn. The suspension sections of the bridge total 6,690 ft (2,040 m). With a main span length of 4,260 ft (1,299m), the Verrazano-Narrows Bridge is the longest suspension bridge in North America, exceeding by 60 ft (18 m) the San Francisco Golden Gate Bridge. The bridge was completed in 1964, and is currently carrying a total of twelve lanes of traffic on two decks (Figure 4)

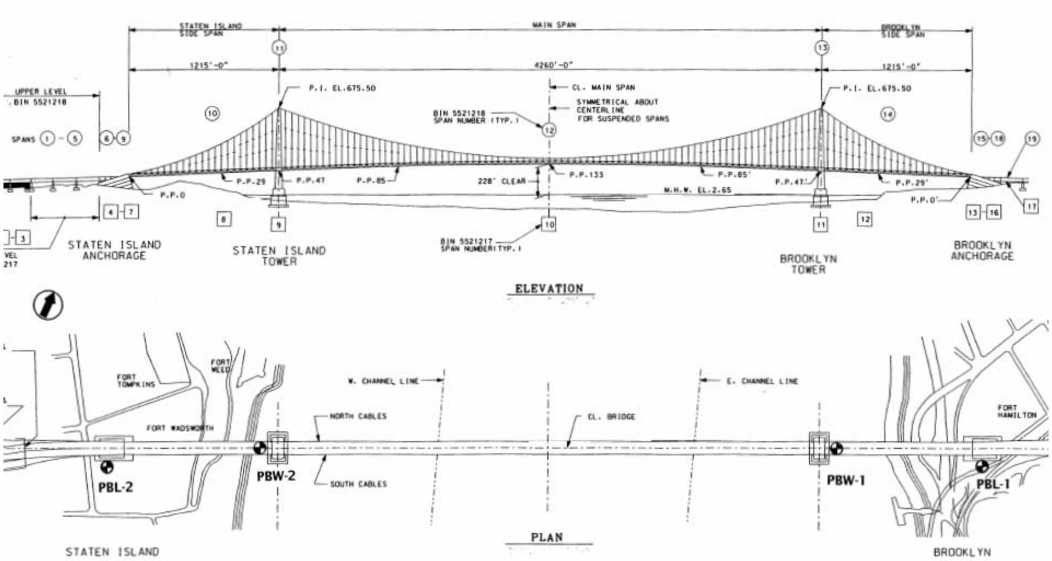

Figure 4 Elevation and Plan of Verrazano-Narrows Bridge

The foundations for the two main piers consist of gravity type deep open concrete caissons resting on firm soil above rock. The Staten Island caisson (length x width x depth: 70m x 39m x 31m) has an invert elevation of 195 ft (32 m) below the mean sea level (MSL), while the Brooklyn caisson (70m x 39m x 50m) has an invert elevation of approximately 170 ft (52 m) below the MSL. The foundations were constructed by dredging the overburden to depth and concurrently sinking the caissons in incremental lifts. The two anchorages are massive concrete blocks resting on footings embedded about 70 to 80 ft (21 to 24 m) into firm soils.

The Verrazano-Narrows Bridge is classified as a critical bridge by NYCDOT and therefore two seismic hazard levels were considered in evaluating the seismic vulnerability of the bridge: the safety evaluation level of a 2,500-yr return period event and the function evaluation level of a 500-yr return period event.

The seismic evaluation of the bridge utilized ADINA. The main cables were modeled with truss elements, with one element spanning between each set of suspenders. The suspenders were modeled with a single truss element each, using a nonlinear material only able to carry tension. The model of the bridge deck included all members of the stiffening trusses, including the “floor frames” spanning between the stiffening trusses and the lateral bracing systems. For the most part, the towers of the bridge were modeled with elastic beam elements. Areas of potential inelastic response were modeled with nonlinear moment-curvature beam elements.

Conventionally, the soil-foundation-structure interaction effects for bridges supported on large gravity type caissons have been considered by using the substructure methods (i.e., the uncoupled approach). In this method, the soil-foundation system is treated as an elastic or equivalent-elastic system in the frequency domain (i.e., an elasto-dynamic analysis). This is in general considered acceptable because most gravity type caissons (particularly if they are deeply embedded in competent soils) are characterized by high frequency vibratory characteristics and whose dynamic response is very different from the long period motions of the long span bridges. Reasonably accurate results can be derived from the elasto-dynamic approach because of the distinctly different modes of vibrations between the bridge and the caissons.

However, if the stability of the caisson itself needs to be addressed in the analysis, either due to the large lateral loads, caissons being in weak soils, or dominating rocking mode, then the elasto-dynamic approach may not provide reliable results. This is particularly the case when gapping (due to the uplift) takes place at the base of the caisson. In this case, a soil-foundation model with allowance for base separation and soil yielding should be incorporated into the analysis to capture the geometric as well as soil non-linearity effects.

For the Verrazano-Narrows Bridge project, a comprehensive soil-foundation-structure model subjected to spatially varying ground motions (i.e., multiple-support motions) was developed and incorporated into the ADINA model (Figure 5). The soil stiffness and damping was modeled by horizontal and shear springs with hysteretic force-displacement relationship to represent the non-linear inelastic behavior as well as dissipated energy in the soils surrounding the foundations. The interfaces between the foundation and the surrounding soils (both at the base and along the sides) were explicitly modeled to allow separation (i.e., gap elements) and slip, in the event that the shear/tensile strengths were exceeded during the simulated dynamic soil-foundation-structure analysis. One of the advantages of this complete model is that the dynamic earth pressures induced along the exterior surfaces of the caisson wall can be directly derived from the response analysis of the bridge to assess the structural capacity of the caisson foundation.

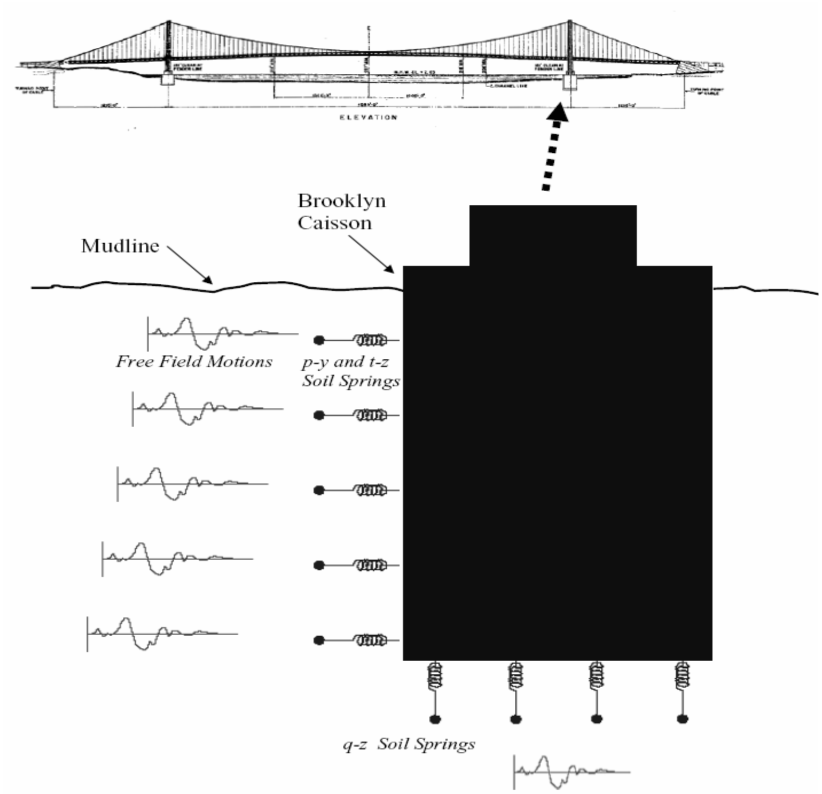

Figure 5 Coupled Soil-Foundation-Structure Interaction Model Subject to Spatially Varying Ground Motions (for Gravity Caisson Foundation)

There are two critical issues in employing the coupled soil-foundation-structure interaction model (Figure 5). They are:

- How to reliably derive non-linear inelastic soil springs for large gravity caissons?
- How to validate the soil-foundation model used in the coupled bridge response analysis?

Non-linear soil springs (p-y, t-z, and q-z) have been widely used and accepted in practice for analyzing piles or drilled shafts subject to lateral and axial loads. There are sufficient testing data generated from many research and construction projects to allow reasonable estimates of soil spring values. For large gravity caissons, however, very little data exists for such purpose. Most of the empirically or analytically derived soil spring data (primarily in the vertical direction) have been for spread footings or slabs on grade, based on over-simplifying assumptions. For deep and large gravity caissons embedded in stratified soils there are no generally accepted procedures in estimating the non-linear p-y, t-z, or q-z springs. To overcome this, an analytical procedure has been proposed for the soil-foundation system of the Verrazano-Narrows Bridge. This procedure is outlined as follows:

1. Create a 3-dimensional continuum model of the soil-foundation system, such as the one shown in Figure 6 (by FLAC3D). The caisson can be modeled as plate or solid/brick elements depending on the nature of its construction. The soils were modeled as solid/brick elements with an appropriate constitutive law for each stratum of different characteristics (such as Mohr Coulomb or Von Mises model with associated stress-strain relationships and failure criteria). The soil models and properties (particularly the modulus values) should be consistent with those used in the site response analysis in deriving the free-field spatially varying ground motions. In addition, the soils can be assumed to have no tensile strength to effectively allow tensile separation between the soil and the caisson (at the base and along the exterior surfaces of the caisson wall).

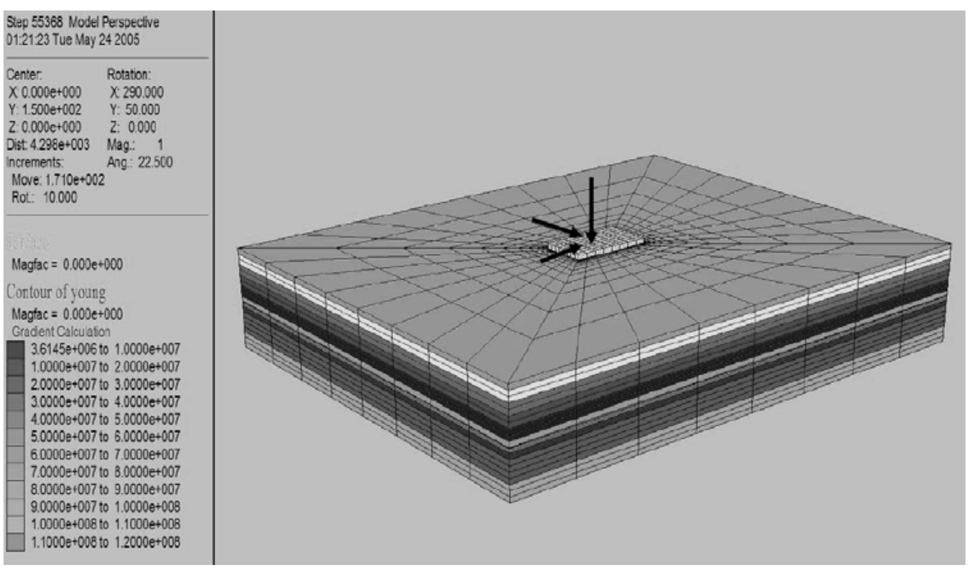

Figure 6 3-D Continuum Finite Difference Soil-Caisson Model (FLAC3D)

2. Three separate loading cases were applied to derive their respective non-linear springs. The first loading case was to apply increasing vertical load at the center point on top of the caisson. By doing this, the non-linear

springs in the vertical direction (including the vertical q-z normal springs at the bottom of the caisson and the vertical t-z shear springs along the vertical wall) can be derived at each individual nodal point based on the non-linear relationship between the stress and displacement at that point.

3. The second loading case was to apply increasing horizontal load in the longitudinal direction to derive the longitudinal non-linear springs at the soil-caisson interface (including the longitudinal p-y springs normal to the walls running in the transverse direction and the longitudinal horizontal t-z shear springs at the bottom of the caisson and along the walls running in the longitudinal direction). Then the process was repeated for the third loading case by applying increasing horizontal load in the transverse direction to derive the transverse non-linear springs. Figure 7 presents an example of derived p-y springs in the longitudinal direction. It is to be noted that the p-y springs display different characteristics at different elevations, reflecting different properties associated with various soil strata.

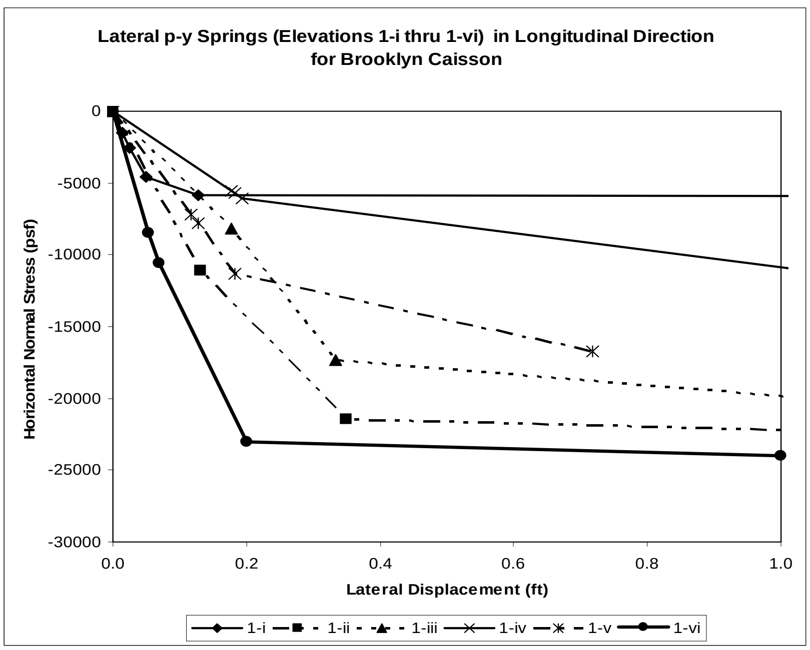

Figure 7 Non-linear p-y Springs Derived from 3-D Continuum Soil-Caisson Model

To validate the soil springs derived from the continuum soil-caisson model, a horizontal push over analysis was performed for both the continuum soil-caisson model (FLAC3D) and the soil spring model (ADINA), which was used in the bridge response analysis. In the push over analysis an increasing lateral load was applied at the center of the top of the caisson. The results from the analysis are presented in Figures 8 and 9. As indicated, the lateral displacement curves agree reasonably well, validating the use of the discrete soil spring model for the global bridge response analysis.

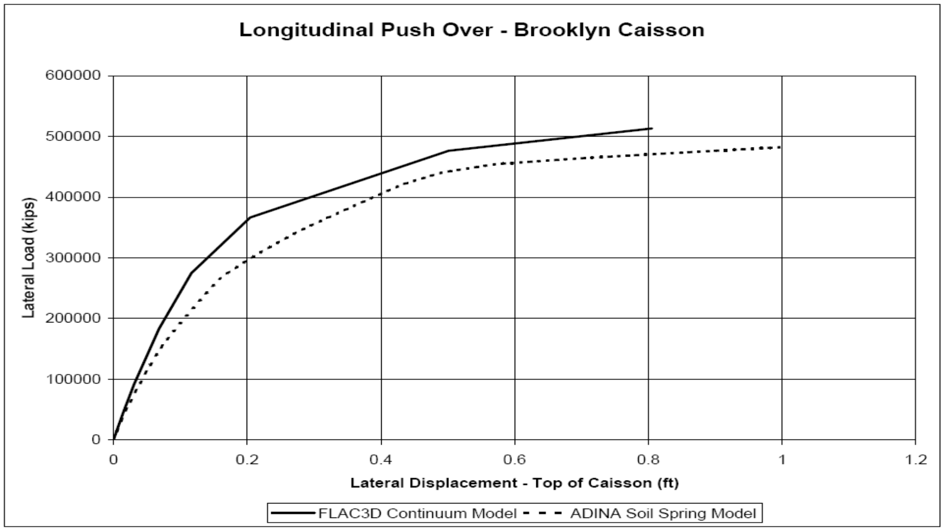

Figure 8 Lateral Load-Displacement Curve at Top of Brooklyn Caisson (Longitudinal Push Over Analysis)

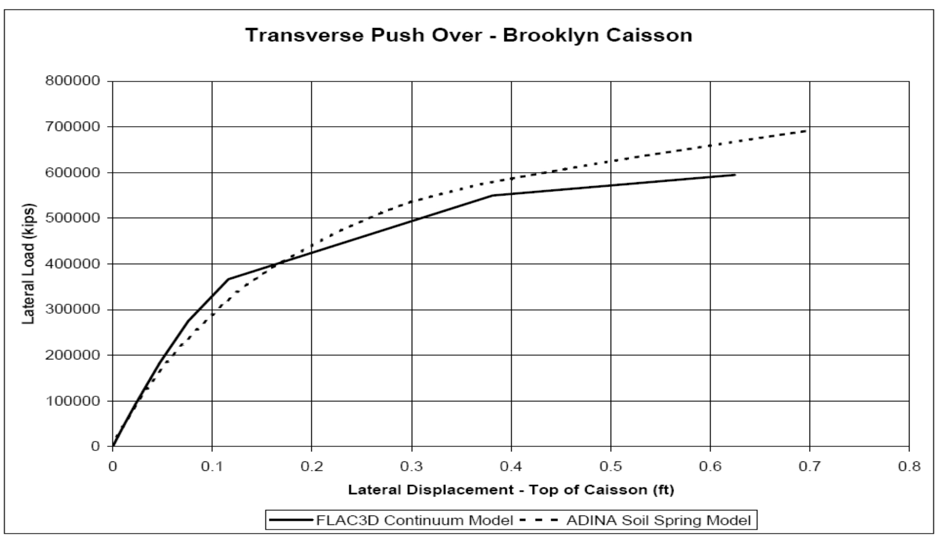

Figure 9 Lateral Load-Displacement Curve at Top of Brooklyn Caisson (Transverse Push Over Analysis)

Piles/Drilled Shaft Design Against Lateral Spread – Cooper River Bridge

Lateral spread can result in the imposition of significant lateral demands on the pile or drilled shaft foundations. Figures 10 and 11 (MCEER/ATC, 2003) depict two cases of loading conditions resulting from lateral spread movements that are relevant to the design of some of the drilled shaft foundations for the Cooper River Bridge. Along the high level approach on the Mt. Pleasant side, the 10-ft diameter drilled shafts on the water side are subject to about 3 ft (0.91 m) of lateral spread soil movement resulting from liquefaction within the upper 40 ft (12.2 m) of soil deposit, using the empirical procedure developed by Youd et al. (1999) . The extent of liquefaction is anticipated to reach the mudline (i.e., without non-liquefied crust on the top). In this case (Figure 10) the liquefied soil is likely to flow around the drilled shafts. An L-PILE analysis was performed by prescribing a 3-ft (0.91 m) ground displacement profile within the upper 40 ft (12.2 m) of the soil, pushing toward the drilled shaft via soil p-y springs. The p-y springs within the liquefied layer were assigned a residual strength and a reduced stiffness. The results presented in Figure 12 show that the maximum deflection at the top of the 10-ft (3 m) diameter shaft is merely a little more than an inch, suggesting little adverse impact even under the Safety Evaluation Earthquake.

The more critical case, however, is when there is a non-liquefied crust riding on top of the liquefied layer, as shown in Figure 11. In this case the piles/drilled shafts tend to move along with the soil, especially for small diameter piles or drilled shafts. The subsurface conditions near the shoreline on the land section suggest that a non-liquefied crust condition (about 10 ft (3 m) thick) is likely to occur during the earthquake.   An initial trial L-PILE analysis using a 5-ft (1.5 m) diameter drilled shaft subject to 3-ft (2.91 m) lateral spread movement shows that the deflection at the top of the shaft will exceed 2.5 ft (0.76 m) and is obviously unacceptable. The final design called for a minimum shaft diameter of 8 ft (2.4 m) in this area. Using the 8-ft (2.4 m) diameter shaft the resulting maximum pile deflection is reduced to about 5 inches (0.13 m) as shown in Figure 13. The response of superstructure to the foundation deflections (both lateral and rotational) was considered acceptable under the safety evaluation earthquake. With adequate reinforcements these drilled shafts can be designed to behave within the elastic range and therefore satisfying the design criteria. Using large diameter drilled shafts in this case has avoided costly ground improvement measures, which otherwise would probably be required.

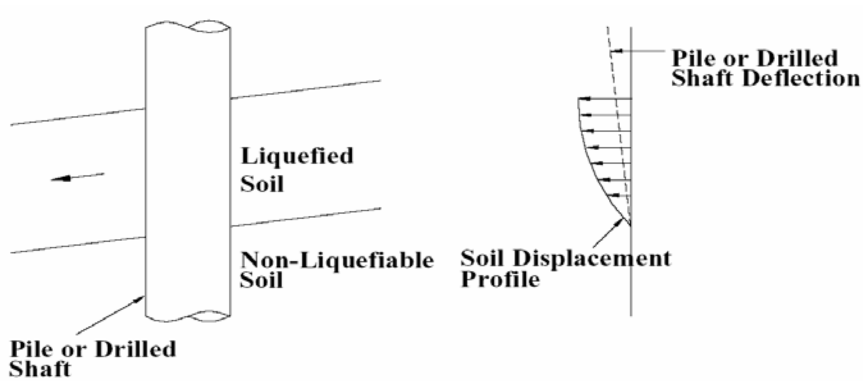

Figure 10   Soil Loading on Drilled Shafts – Liquefied Soil Without Crust

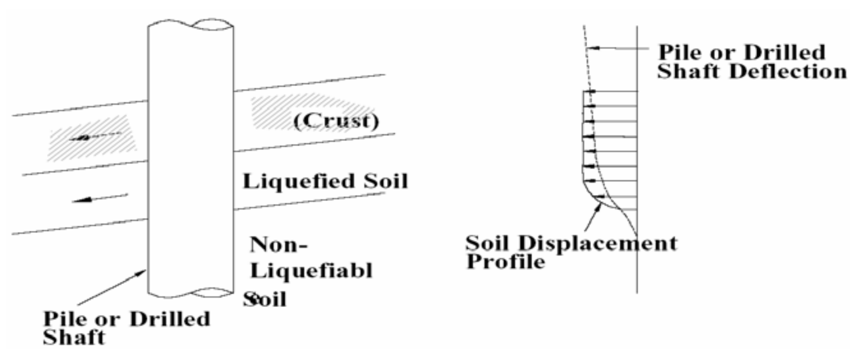

Figure 11   Soil Loading on Drilled Shafts – Liquefied Soil With Crust

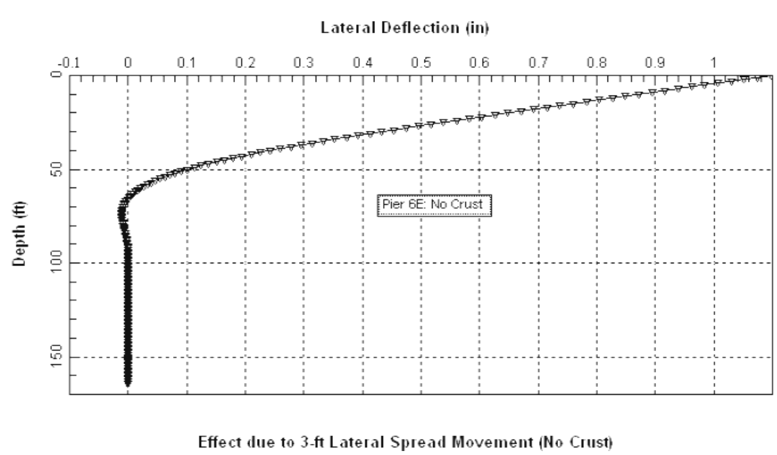

Figure 12 Response of 10-ft (3 m) Dia. Drilled Shaft - without Crust

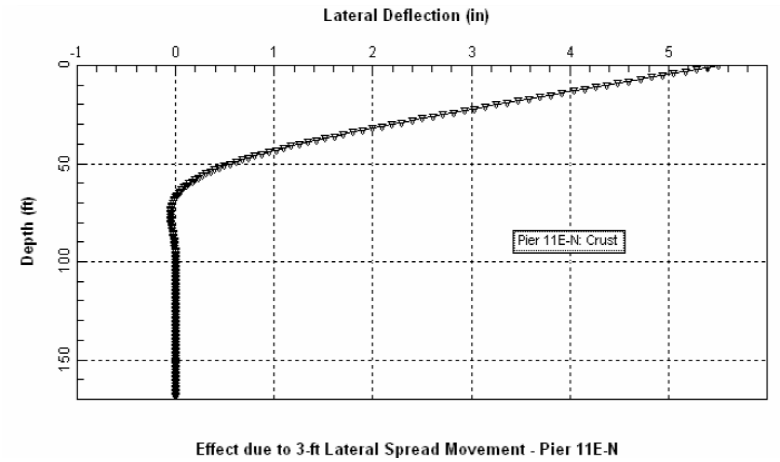

Figure 13 Response of 8-ft (2.4 m) Dia. Drilled Shaft - with 10-ft (3 m) Crust

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