Evaluation of Soil Structure Interaction Effects for Two Major Bridges in Turkey

A. Giannakou¹, W. Chen², and J. Chacko³

ABSTRACT

The soil structure interaction evaluations performed for two major long span bridges in Turkey are presented. The development of design ground motions incorporating soil-structure interaction effects and the assessment of fault rupture induced deformations on both the foundation and the superstructure are discussed. A substructuring method was adopted for the global dynamic structural bridge models. Nonlinear foundation springs were developed together with an approach that incorporated the significant coupling effects between displacements and rotations of deep caisson foundations for the spring implementation in the global model. Numerical analyses were performed to develop design ground motions at the base of the piers incorporating the effects due to the presence of the foundations. Additionally, due to the very close proximity of one bridge to the North Anatolian Fault, advanced numerical models were developed to evaluate the fault rupture-induced demands on a bridge pier foundation in terms of displacements and rotations.

Introduction

Two major long span bridges are being constructed in Turkey, the Izmit Bay bridge and the 3rd Bosphorus bridge, located about 2- and 40 km from the North Anatolian Fault (NAF), respectively (Figure 1). The Izmit Bay bridge is a ~3-km-long suspension bridge that crosses Izmit Bay with a 1.3-km long South Approach Viaduct consisting of 11 piers located within the secondary deformation zone to the north of the primary trace of the NAF (Figure 2). The 3rd Bosphorus Bridge crosses the Bosphorus strait separating the European and Asian continents and its main span is supported by two land-founded towers approximately 1.4 km apart.

A substructuring approach was adopted for the dynamic time history analyses of both the Izmit Bridge South Approach Viaduct and the 3rd Bosphorus Bridge. Kinematic motions and foundation springs were developed for use in dynamic structural analyses of the bridge superstructures. Since the foundations of both bridges consist of deep caissons an approach that incorporated the significant coupling effects between displacements and rotations of deep caisson foundations was developed for the implementation of the foundation springs in the structural analyses.

Since the Izmit Bridge South Approach Viaduct was located within the NAF secondary fault zone the viaduct and its foundations also had to be designed for fault rupture. Advanced numerical models and state-of-the-art methodologies were adopted for the foundation design of the viaduct.

¹Amalia Giannakou, Fugro, Istanbul, Turkey, agiannakou@fugro.com
²Wei Yu Chen, Fugro, Oakland, USA, wychen@fugro.com
³Jacob Chacko, Fugro, Istanbul, Turkey, ichacko@fugro.com
This paper presents soil-structure interaction evaluations for the development of design ground motions and springs at the base of the bridge piers, and the assessment of fault rupture passing through the pier foundation.

**Substructure Approach: Development of Foundation Springs and Kinematic Motions**

**Design Approach**

A substructuring approach was adopted for the dynamic time history analyses of the two bridges. A global model of the bridge was developed for the dynamic SSI evaluations. Foundation springs were developed to replace the foundation system under the pier in the global model. Three dimensional finite element models of the bridge foundations and the surrounding soil were built and nonlinear foundation springs were developed. The nonlinear springs were used in the global model to represent the load-deflection characteristics of the foundation and surrounding soil and to capture damping through hysteresis. A Masing rule was used to capture unloading and reloading. Because the bridge foundations were caisson-type systems, the significant coupling effects between displacements and rotations had to be incorporated in the implementation of the springs in the structural model. To account for this in the structural model the nonlinear springs were connected to the bridge piers through a rigid vertical link. The rigid link was introduced to capture the effects of the embedment depth of foundation and to capture the coupling between rotational and translational stiffness of the springs. The length of the rigid link varied for different foundation depths.
The presence of the foundation tends to “resist” and, hence, modify soil deformations generated by the passage of propagating seismic waves. As a result the motion at the foundation (kinematic motion) differs from the free-field ground motion. Numerical models (without a superstructure) were developed at each foundation location to simulate the propagation of the ground motion through the soil to the top of the foundation. The numerical models simulated the soil with nonlinear solid elements and the foundation with elastic soil elements. The approach is illustrated schematically on Figure 3.

![Figure 3. Schematic Illustration of Approach for Dynamic Structural Evaluations of the Bridges: (a) Step 1: Development of Kinematic Motions and Foundation Nonlinear Springs, (b) Step 2: Incorporation in Global Structural Model of the Bridge through a Rigid Link.](image)

**Development of Nonlinear Foundation Springs**

The foundation system of the Izmit SAV consisted of caisson-type foundations made from slurry walls with a plan area on the order of 8 by 20-meters and lengths varying from 10 to 20 meters. Soil conditions under the Izmit SAV foundations consisted of stiff to hard overconsolidated clays with undrained shear strength varying from about 50 kPa near the surface to about 200 kPa at 50 meters depth. The 3rd Bosphorus bridge Tower foundations consisted of 20-m deep caissons with a diameter of 20 meters. The site conditions consisted of moderately strong to strong agglomerate with unconfined strength values of about 30 to 40 MPa.

Three dimensional finite element analyses were conducted to develop the backbone curve representing the nonlinear load-deflection relationships (Figure 4a). The backbone curves were developed by numerically "pushing" the foundation in all three directions and calculating the resistance offered by the surrounding soils or rock. The foundation and soil were modeled with solid elements. The soil nonlinear behavior was modeled with the Mohr Coulomb model while the rock nonlinear behavior was modeled with the Hoek-Brown model. The foundation was modeled with elastic solid elements. For the Izmit SAV slurry wall foundations due to the disturbance of the soil at the base of the slurry wall during construction, a 1-meter thick “weak” layer ($S_u=15$ kPa) was introduced at the base of the slurry wall in the bearing capacity estimates and the numerical models to limit the axial
resistance from the base bearing capacity of the system. Additionally, an interface was introduced between the slurry wall and the surrounding soil limiting the side friction between the slurry wall and the surrounding cohesive soils to 0.55 of the undrained strength. No contact was assumed between the cap and the soil and a 1-meter thick gap was introduced between the cap and the soil inside the slurry walls. For the 3BB caisson foundations interface elements were used around the foundation (Chen et al 2014).

For the development of vertical load-deflection curves the model was first brought to force equilibrium under gravity. The foundation was then pushed vertically to generate the load-deflection curves. During dynamic loading, changes in loading in the vertical direction are expected to be rapid and the effective stresses in the soil are assumed to not change due to generation of pore pressures during the transient and rapid axial loading. Similarly, if the foundation is pushed up, suction is anticipated to be developed due to negative pore pressure generation. Therefore, the behavior of the sand layers, where present, was modeled as undrained while pushing the footing. The soil layer may behave partially drained depending on its permeability, but those effects were considered to be insignificant for the soil conditions on site. Cavitation forces were considered in the analyses and suction was limited to 1 atmosphere in addition to the in situ pore pressure at a given depth. Similar to the vertical response, the lateral response of the foundation was estimated using the 3D models. The model was brought to force equilibrium, and pushed laterally to obtain the lateral load-deflection curves.

Caisson type foundations exhibit significant coupling effects between displacements and rotations which cannot be neglected. This means that the application of horizontal shear force at foundation head results in both horizontal displacement and rotation and the application of bending moment at foundation head results in both rotation and horizontal displacement. For the implementation of the nonlinear springs in the global model a simplified foundation model was developed that includes the nonlinear horizontal and rotational springs (in the form of backbone curves with a Masing rule, Figure 4d) in combination with a vertical rigid link (Figure 4b). To estimate the length of the rigid link the foundation stiffness components were linearized targeting the expected range of shear force and bending moments (Figure 4c). More details on the development of the rigid link element can be found in Khosravifar et al (2015).

**Development of Kinematic Ground Motions**

Numerical models were developed including the foundation and the surrounding soil and the ground motion was propagated from the base of the models to the foundation top. Where soil was present the soil elements were modeled using nonlinear constitutive laws that capture the finite strength of the soil as well as its nonlinear stress-strain relationships. Where rock was present, an elastic stress-strain relationship was assumed. No superstructure was included in these analyses.

For the Izmit SAV due to the high levels of shaking at the project site, significant non-linearity and relatively high shear strains (in excess of 1 percent) are induced in the soil by the ground motion propagation. Accordingly, the soil behaves according to a nonlinear stress-strain law implemented through a "hysteretic" model that allows for the modeling of the reduction of the shear modulus with increasing shear strain and corresponding increase in hysteretic damping. The foundation elements were modeled as elastic.
The motions that were developed from Probabilistic Seismic Hazard Assessment (PSHA) analyses (Chacko et al. 2014) were input at the base of the finite element models and propagated upwards through the soil to the top of the foundation. Motions were input as “outcrop” motions at depth. A compliant base was used at the bottom of the finite difference model and “outcrop” motions were used to excite the base of the model. Kinematic acceleration time histories were developed in the two horizontal directions at each foundation location for seven motions and three design events.

For the Izmit SAV, numerical models were constructed at the different pier locations that included the variation in soil stratigraphy at each pier and the variation in foundation slurry wall length at each pier. The Fault Normal (FN) and Fault Parallel (FP) components that resulted from PSHA, were rotated to take into account the curvature of the SAV (left illustration, Figure 5). The rotated horizontal ground motions were input at the base of the numerical models as described above and propagated upwards. The ground motions are altered by site response effects, as well as by interactions with the foundation block. Due to variations in the soil conditions at each of the foundation locations, differential site response effects were observed in the transient time histories at each of the pier locations analyzed. The differential responses occur due to variations in the soil conditions, as well as different levels of nonlinearity that develop within each of the soil profiles. Kinematic acceleration time histories were developed in the two horizontal directions (longitudinal and transverse to the bridge axis) at each pier foundation and at the south abutment. The kinematic horizontal displacement time histories estimated at the top of two consecutive pier foundations were subtracted to estimate the peak transient relative displacements in the Fault Normal and Fault
Parallel direction after they were rotated to also accounting for the curvature of the SAV structure (right illustration, Figure 5). The resulting peak relative transient displacements were 0.25 and 0.1 meter, respectively, for the 1000-year event (Figure 5). The SAV structure had to be able to accommodate these types of relative displacements within the performance requirements.

![Image](image.png)

**Figure 5. Relative Horizontal Displacement Time Histories Between Adjacent Piers of the SAV Fault Normal Direction, 2475-Year Return Period**

**Evaluation of Fault Rupture Induced Demands on a Bridge Foundation**

**Design Approach**

Recent research efforts combining field studies, centrifuge model testing, and numerical modeling have resulted in the development of a validated methodology for analysis and design of foundation–structure systems against surface fault rupture (Anastasopoulos et al., 2008; Loli et al 2011). Fugro applied this methodology in order to evaluate the demands on the Izmit SAV foundations from a secondary fault rupture that passes through one Pier location. The analysis of the bridge–foundation system subjected to faulting–induced deformation is conducted in two steps (Figure 6). In Step 1, the response of a single bridge pier foundation subjected to fault rupture deformation is analyzed where a detailed 3D model of the structure and surrounding soil is subjected to fault rupture induced displacement at its base. In Step 2, a global structural model is subjected to the computed displacements and rotations of Step 1.

**Fault Rupture – Soil – Foundation Interaction**

The Izmit SAV design criteria specified that the structure had to withstand free field displacements of 0.7 meter (horizontal) and 0.3 meter (vertical) anywhere along its length for the 1,000-year event and 1 meter and 0.5 meter for the 2,475-year event (Travasarou et al. 2013).

3D nonlinear numerical analyses were performed to evaluate the demands on a pier foundation in terms of displacements and rotations due to fault rupture (both strike slip and dip slip with dip angle of 80°).
Fault rupture propagation through the soil will induce large shear strains, therefore consideration of the post-peak strain softening behavior of soils is essential (Bray et al., 1994; Anastasopoulos et al., 2007). A Mohr Coulomb failure criterion that allows for strain softening was used to model the soil layers. The ability of the numerical model to capture the fault rupture propagation through soil was verified against centrifuge experiments (Giannakou et al, 2012). Parametric analyses were performed with respect to the relative position of the foundation to the fault rupture outcrop, the dip angle of the dip-slip fault-component and the fault rupture orientation relative to the foundation. Figure 7 presents analyses results in the form of a deformed mesh considering different positions of fault offset relative to foundation centerline. Since the foundation system is stiff and continuous, it forces the fault rupture to divert around the footing, although rotation and torsion of the footing does occur. The horizontal displacements of the foundations were found to vary between 0.9 to 1.0 meters, the vertical displacements between 0.5 to 0.6 meters and the rotations between 0.2 to 1 degree for the 2475-year event. These displacements and rotations were used in a static analysis to distort one of the foundation elements of the global model (displacements applied directly to the footing) to evaluate the consequences of the resulting displacements on the superstructure and deck elements that are supported.

Figure 6. Overview of Design Approach Against Fault Rupture: (a) Step 1: Detailed Soil Foundation Model, (b) Step 2: Detailed Superstructure Model.

Conclusions

The design approach adopted for the evaluation of soil structure interaction effects for two major bridges in Turkey was presented in this paper. Complex phenomena that include strong ground motions, soil nonlinearity, and influence of foundation embedment depth on development of foundation springs have been considered in the design. Additionally,
advanced numerical models were developed to evaluate the effects of fault rupture through a bridge foundation. The methodologies adopted for the seismic design of the two bridges offered an improved characterization of the bridge dynamic behavior and identified controlling mechanisms which led to design optimization, efficiency and significant cost reductions. Similar methodologies can be adopted for quantifying seismic hazards for important projects.

Figure 7. Results in the Form of Deformed Mesh of Parametric Analyses of Different Position of Fault Offset relative to Foundation Centerline.

Acknowledgments

We would like to thank Nurol and Ictas-Astaldi JV for their support and input in adopting and applying the approaches presented here.

References


