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CONTINUOUS HORIZONTAL REINFORCEMENT DIAPHRAGM WALL APPLICATION

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ABSTRACT

The aim of this paper is to give detailed information about design, construction and testing of a box type foundation system composed of a 100 cm thick diaphragm wall used to support foundations of the South Approach Viaduct; which is located south of Izmit Bay Bridge Project. The on-going Izmit Bay Bridge Project; that is composed of a 3-km-long suspension bridge and a 1,4-km-long South Approach Viaduct; is located at one of the most seismically active places in the world. The site, which has the potential to experience significant earthquakes associated with the relative motion accommodated on the North Anatolian Fault, is underlain by deep deposits of soft soils, and areas of unstable and liquefiable soils. Therefore; special foundation approaches were implemented for the construction of box type foundation system of the South Approach Viaduct.
The paper also outlines relevant aspects of the diaphragm wall design with particular reference to aspects concerning overall stability and design loads; under static and seismic loading conditions.
Moreover, the construction methodology and application stages of project in order to maintain continuous horizontal reinforcement; which is quite different from the traditional methods is presented. The success of this technology in achieving and maintaining the required design loads has paved the way for further applications within the seismically high region.

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Continuous Horizontal Reinforcement Diaphragm Wall Application

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ABSTRACT

Diaphragm walls are implemented as a box type deep foundation under the South Approach Viaduct Pier Foundations; which is located south of İzmit Bay Bridge and a part of Gebze-Orhangazi – İzmir Motorway Project. However, since the project location crosses North Anatolian Fault and it lies on the secondary fault zone, different approaches are implemented for the construction of these diaphragm walls. This paper outlines the construction methodology and application stages of project in order to maintain continuous horizontal reinforcement; which is quite different from the traditional methods.

1. Introduction

The Izmit Bay Bridge Project; that is composed of a 3-km-long suspension bridge and a 1,4-km-long South Approach Viaduct is planned to be constructed in one of the most seismically active places in the world. The bridge will connect the Diliskelesi peninsula to the North with the Hersek peninsula on the south. The proposed project site spans the plate boundary between the Anatolian plate on the south and the Eurasian plate on the north and will experience significant earthquakes on the North Anatolian Fault Zone (source of the 1999 Magnitude Mw 7.4 Izmit and Mw 7.2 Düzce earthquakes).
The bridge is the critical link of the 420km Gebze–İzmir Motorway awarded through a Build Operate Transfer (BOT) model to the NÖMAYG Joint Venture in 2009. Nurol İnşaat ve Ticaret A.Ş. was awarded by NÖMAYG for the design and construction of the on-going South Approach Viaduct of the Izmit Bay Bridge (Figure 1). The structural design was carried out by Wiecon; geotechnical, geological, and seismological evaluations of the proposed project region was performed by Fugro and diaphragm wall construction works were carried out by Kasktaş A.Ş.

2. Description of the Project

2.1 Project Area

The South Approach Viaduct (SAV) is located along the western margin of Hersek peninsula, within 5 km from the North Anatolian Fault and within a zone of secondary deformation around the primary trace of the North Anatolian Fault. The alignment crosses the western Hersek peninsula shoreline approximately 500 meters south of the Northern tip of the peninsula, and continues south with the centerline within about 80 meters of the shoreline.

Figure 1. Location and Alignment of the South Approach Viaduct (SAV)

The South Approach Viaduct of the Izmit Bay Bridge brings the bridge down from on the order of El. 60 meters at the South Anchorage to an elevated embankment approximately 1,4 km farther south. The proposed viaduct consists of 11 Piers and 10 standard intermediate spans with lengths varying between 136 m and 100 m and two bank spans one of 125 m, attached to the main bridge and the other of 72 m attached to the south embankment (Figure 2). The South Approach Viaduct is located between km 7+084.26 (Pier P0) and km 8+462.43 (Pier A12). The SAV piers are numbered consecutively starting from Pier P1 which is located south of the South Anchorage of the main Bridge to Pier P11 and terminating at the south embankment A12, located south of Pier P11. The interface between the main bridge and the SAV is at the South Anchorage of the main bridge (Pier P0).

Figure 2. General Layout of South Approach Viaduct Footings

2.2 Subsoil Conditions and Geology

Prior to the construction of the foundations of the South Approach Viaduct, a detailed geophysical and geotechnical survey program was executed between 2011 and 2012 by Fugro Sial. The site investigation consisted of 10 no. of boreholes down to 60 m depth; 126 no. of Menard Pressuremeter tests in all boreholes; 48 no. of CPT; and laboratory tests. Since the site is underlain by deep deposits of soft soils, and areas of unstable and liquefiable soils, characterizing the geological, seismological and geotechnical setting, foundation soil conditions, fault locations, as well as developing an appropriate design criterion was the most critical component for the project. Therefore; a sophisticated and extensive site investigation program at the Izmit Bridge was carried out by Fugro in order to provide geotechnical engineering services for the final design of the proposed viaduct.

The soil layers encountered at the site are presented below:

Loose to Medium Dense Sand Layer: The thickness of the layer changes between 2m and 5m and it is prone to liquefaction. The color is grey and SPT N value is between 5~10; γ_n=18~18.8 kN/m^3; Ø=25~35°.

Stiff Clay: The thickness of the layer changes between 2m and 5m and in yellowish brown color. SPT N value is between 10~20; Cu= 75~200 kPa; γ_n=18.8~19.0 kN/m^3.

Very stiff to hard clay: The thickness of the layer changes between 18m and 35m and in greenish gray to dark grey color. SPT N value is N>30; $\gamma_n \approx$19.0 kN/m^3, Cu ≥200 kPa

The ground water depth is about ~2 m. The idealized soil profile according to the existing boreholes is shown in Figure 3 below:

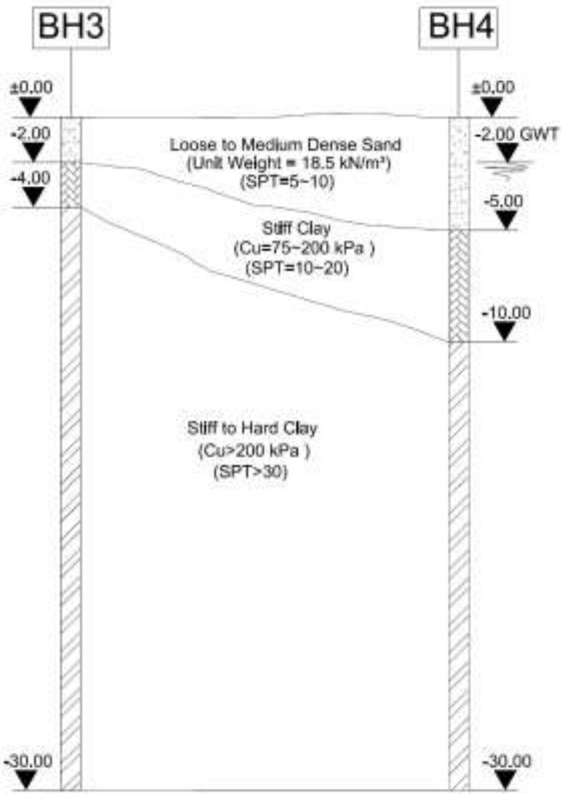

Figure 3. Generalized Soil Profile

Having evaluated the results of the shallow water geophysical and geotechnical survey data shows presence of several secondary faults within the area and consequently the location of the south anchorage of the main bridge was moved approximately 150 m north from the originally proposed location to an apparent area of no recent faulting. This shift to the North necessitated the extension of the South Approach Viaduct by about 150 meters to the North into an area of identified secondary faults. Piers P01 and P02 of the Viaduct are located within number of active fault traces of North Anatolian Fault (Figure 4)

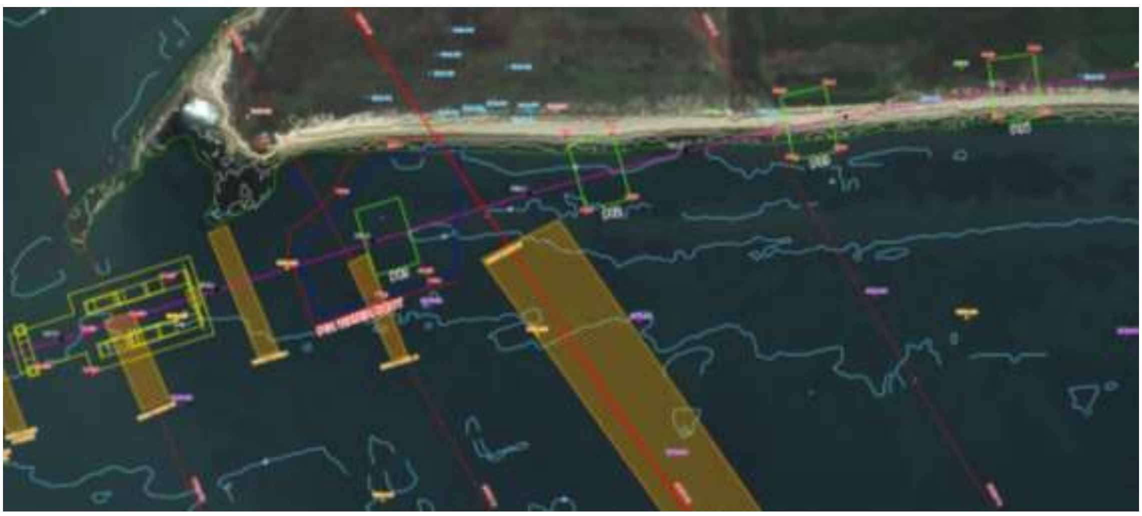

Figure 4. Secondary Faulting Zone between P01-P02 footings of SAV

3. Foundation Concept

3.1. Foundation Concept Performance Evaluation

Analyses were conducted for the performance evaluation of the different foundation types. The performance evaluation was focused on the following aspects:

- Foundation axial capacity
- Foundation performance under static and earthquake loads
- Foundation performance against fault rupture induced displacements

Three dimensional finite difference analyses were conducted by Fugro using the computer program FLAC (Itasca 2011) to develop the axial load-deflection curve for different foundation types and sizes. The shallow footing dimensions in plan view that were analyzed are 30 m x 30 m (longitudinal x transverse), 28 m x 33 m, 26 m x 33 m, 25 x 25 m, and 20 m x 20 m. The base of the shallow foundations is at elevation -5.5 m.
The diaphragm wall foundation system dimensions that were analyzed are 8 m x 21 m x 13 m (longitudinal x transverse x depth), 8 m x 21 m x 23 m, 13 m x 21 m x 23 m, and 15 m x 27 m x 23 m. The cap size thickness was 3 meters, with the cap bottom at -2 m.
The foundation system was modeled using the solid elements. The model was first brought to force equilibrium under gravity. The footing was then pushed vertically to generate the load-deflection curves. Figure 3 shows the FLAC 3D models for the different foundation systems.
The deformation patterns of the soil during the axial push for the shallow foundation (25 m x 25 m) and the diaphragm wall foundation (13 m x 21 m x 23 m). The axial load-deflection curves for the two foundation systems are presented in Figure 5.

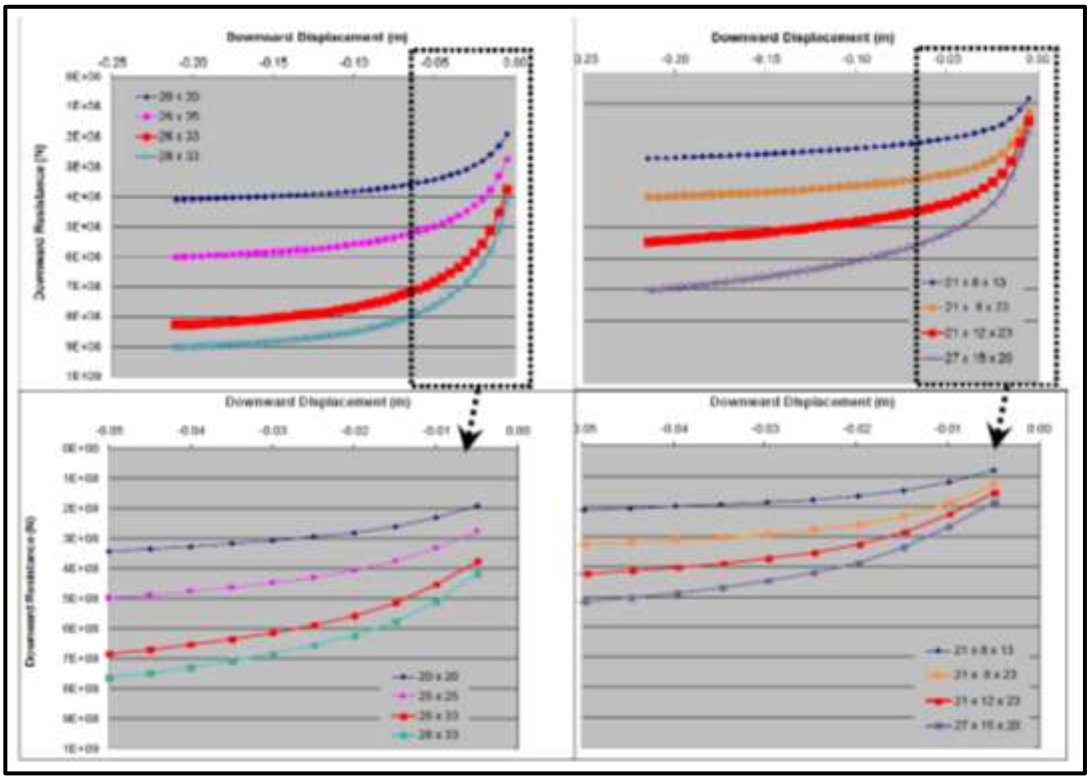

Figure 5. Axial Load-Deflection Curves - Shallow Foundation vs Diaphragm Walls

Analyses were performed to evaluate the performance of two different foundation types for the SAV piers, a shallow foundation and a diaphragm wall foundation system. Due to the severity of the design ground motions, the superstructure introduces significant moments on the foundation. For a shallow footing solution, the size of the footing is driven by the overturning resistance to the superstructure loads rather than the vertical bearing capacity. Additionally, since fault rupture through a pier foundation is the main concern for this project, the foundation system was found to play a key role in the response of structures subjected to fault induced ground movement. Structures resting on rigid and continuous foundation systems (such as a raft, or a box-type foundation) have demonstrated to be capable of achieving a very satisfactory performance, irrespective of the faulting type. As a result; a caisson-type of foundation was selected as the most suitable foundation system; which consists of four perimeter diaphragm walls, a concrete cap, and a diaphragm wall constructed along the bridge transverse direction under each Pier legs. The thickness of the diaphragm walls is 1.00 meter and the cap thickness is 3 meters. The foundation concept is shown schematically in Figure 6. A total of 14,400 m^2 diaphragm wall with a maximum depth of 23 m was executed.

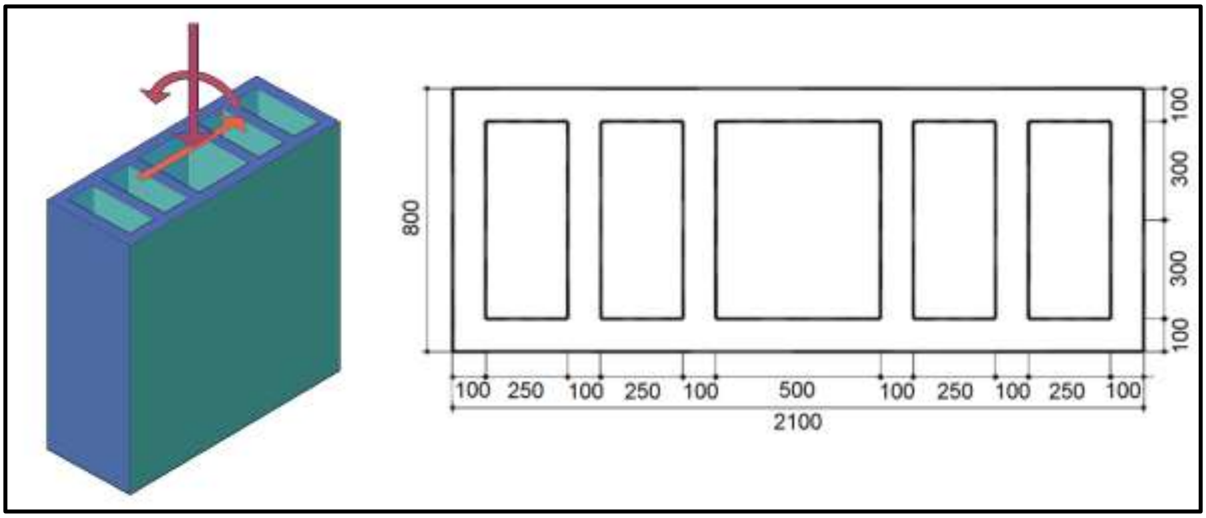

Figure 6. Box-Type Foundation Concept

4. Construction Sequences

The design of box-type foundation necessitates a job specific special diaphragm wall methodology; which requires special tools and techniques; in order to maintain continuous horizontal reinforcement; which is quite different from the traditional methods. The success of this application in achieving and maintaining the required design loads has paved the way for further applications within the region with a high seismicity.

4.1 Test Panels

By bearing on mind the fact that the selected diaphragm wall methodology was not only the first one to be applied in Turkey but also the number of similar projects completed worldwide was very limited. Therefore; before commencing construction, a number of trial test panels having the same features as the box-type foundation diaphragm wall panels were constructed down to the design depth to calibrate the construction procedures. These trial test panels were executed so as to verify implemented construction methodology for continuous horizontal reinforcement diaphragm wall in the vicinity of the working location.

4.2. Construction Stages

4.2.1 Excavation

Panel excavations were executed by using hydraulic grab. During the construction of test panels, instability of panels due to the uppermost loose sandy layers was observed. Consequently jet grout columns were implemented under the guide walls for improving the uppermost loose layers and eliminating the risk of instability of panels.

Panel layout plan and sequence of construction works were prepared for each footing. The diaphragm wall panels are classified according to the construction procedures as Starter (S), Intermediate (I) and Closer (C).

L and T shape starter panels (S) and intermediate panels (I) were excavated in two stages, whereas closer panels were excavated in one stage. (Figure 7) After diaphragm wall excavations were finished, bentonite samples were taken from the bottom of the excavated trench and taken to the laboratory for standard bentonite quality control tests prior to concreting. Then, reinforcement cages and special tools appropriate for each type of panel were placed in to the excavated trench.

Figure 7. Diaphragm Wall Panel Excavation

4.2.2 Preparation and Placement of Reinforcement Cages

Since the diaphragm wall construction required a special technique for providing the continuity of the horizontal reinforcement, special reinforcement cages in different shapes and dimensions for each panel type were used in accordance to the dimensions of the excavated diaphragm wall panels. (Figure 8) Besides as requested by the Designer all horizontal reinforcements of the adjacent panels are overlapped by 700 mm as shown in Figure 9.

During preparation of starter and intermediate panel reinforcement cages, steel end-plates were welded to cages to provide a barrier between the concreted and un-concreted sections.

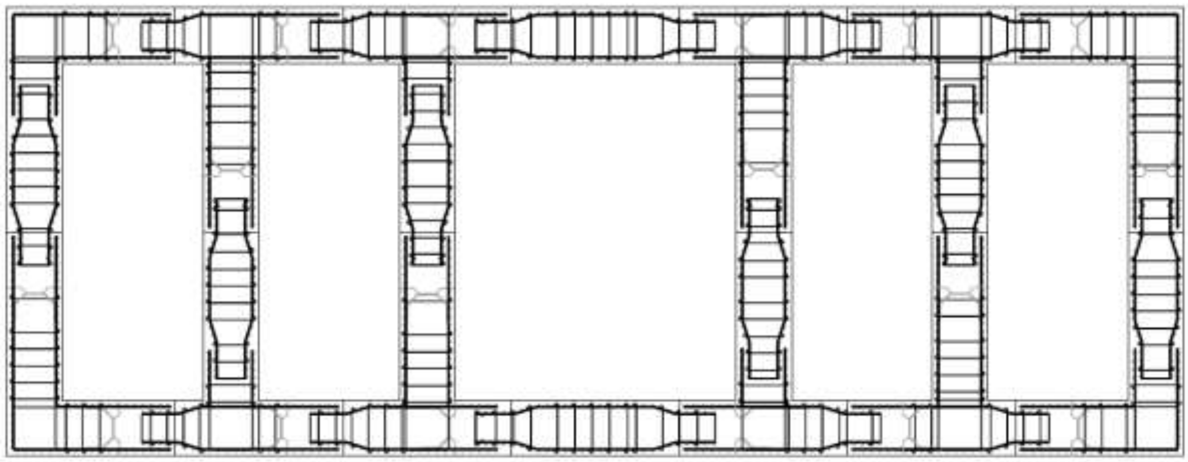

Figure 8. Layout of Diaphragm Wall Reinforcement

While the reinforcement cage was lowered into the excavated trench, concreted section of the cage was covered with geotextile at the mouth of the trench in order to prevent concrete leakage to adjacent panel and thus, to preclude the problems that would occur during construction of adjacent panel. (Figure 10) This geotextile also surrounded the bottom of the cage to provide maximum protection against leakage of concrete beyond the partition steel end-plate. The secondary panel cage (female cage) was designed to allow a proper splicing between the subsequent panel cages.

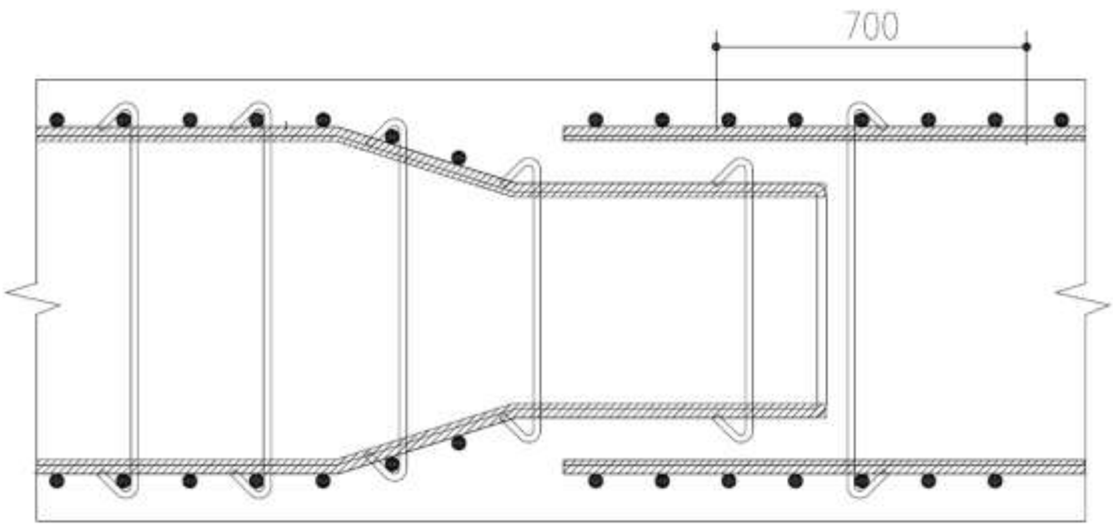

Figure 9. Typical Horizontal Reinforcement Overlap Detail

Each steel reinforcement cage was assembled horizontally on the ground in a single longitudinal section. Each cage was stiffened, using convenient stiffening elements placed between the main reinforcing bars, to provide the cage rigidity needed to avoid deformations during lifting and lowering into the trench. In addition, proper welding was performed to increase the stiffness of the cage during handling and lifting. Appropriate spacers were placed on the faces of the cage to ensure the correct concrete cover. All the cages were equipped with a number of 60-mm-diameter steel sonic pipes to measure the concrete integrity. The distance between the sonic pipes was not greater than 2.0 m.

Figure 10. Covering the reinforcement cage with geotextile

The maximum weight of each cage was about ~40 tons. The handling process for reinforcing cages was performed by two service cranes. Each cage, assembled in a horizontal position due to its exceptional shape and dimension, was lifted from its horizontal position and suspended vertically by means of the two service cranes. A special lifting frame was used to avoid any localized overstress of the assembled cage.
Once the cage was in the vertical position, crawler crane supported the cage from the top and moved it to the open trench. The cage was slowly lowered into the trench and, once down, was suspended on the guide walls through bars welded to the main longitudinal bars. Therefore, the reinforcement cage did not rest on the bottom of the trench, and the clear distance between the reinforcement and the bottom of the trench was not less than 150 mm.

4.2.3. Concreting
Concreting works were executed by using tremie method. Prior to starter and intermediate panel concreting works, gaps which existed outside of steel end plates were filled with a suitable fill material. Concreting and filling processes were performed in a simultaneous manner one after another until concrete overflowing at the head of trench was observed.

5. Quality Control Tests after Construction Works

As part of a strict quality assurance program pursued in the project, cross hole sonic logging tests were performed at each footing in 100 different points (0.8 nos/m^2) to investigate continuity of panels along their depths. This test enables information about the discontinuities and defects that could be occurred during construction. Moreover, tests were executed by analyzing the sound waves delivered between transmitter and the receiver probes placed on the steel pipes welded to the reinforcement cages while preparation of them. By examining the delivery time and magnitude of signals, continuity of the panels which are constructed can be interpreted. Cross hole logging tests did not indicate any area of poor quality concrete and joint between the panels appeared to be sound and integral within the depth of the diaphragm wall.

Conclusions

In general, when designing structures in seismically active areas, foundations of critical structures are typically located away from known fault. However, for long structures such as bridges, tunnels and pipelines, a fault maybe unavoidable, and fault rupture risk impossible to preclude. In the subject project interpreted geotechnical and geophysical data collected during site investigation revealed numerous traces of the secondary fault zone on the entire area near the south anchorage of the main suspension bridge and the south approach viaduct. Therefore; foundation design for the approach structures was optimized by performing probabilistic fault displacement hazard analyses in combination with advanced numerical soil structure interaction studies. In order to cope with the above mentioned difficulties, a box-type diaphragm wall system with continuous horizontal reinforcement was selected as the most appropriate foundation system. This paper describes a new construction method of diaphragm wall; which was successfully carried out in Turkey using a continuous horizontal reinforcement in accordance to the strict HSE and Quality Control / Quality Assurance Programs implemented in the Project. Significant data related to the construction and design of the proposed system was documented, and the technical details of the design approach were highlighted. This project might be considered as a good model in Turkey; which verifies that major iconic structures can be constructed at one of the most seismically active places in the world.

Acknowledgments

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