Seismic Soil-Structure Interaction Analysis for the Transbay Transit Center in San Francisco-Methodology

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SUMMARY:
The Transbay Transit Center (TTC) is a 5 story transportation development in downtown San Francisco with a 50 ft deep reinforced concrete trainbox below grade and a steel superstructure .The structure shall serve as the major hub for local public transportation, including the High Speed Rail service, bus services, retail areas and a 5.6 acre roof level park. The proposed structure is approximately 1,500 feet long, with a 300 foot extension planned at a later stage. The challenges to the design include the variations in substructure and superstructure profile, presence of a stiff end wall at the west end, spatial variation of soil properties, nonlinear nature of soil and structural response, presence of adjacent structures, and the high water pressure. This paper discusses the methodology for performing seismic soil structure interaction analysis as part of the design of the TTC, with more emphasis on the response of the embedded trainbox. The use of a combination of different analysis techniques to obtain the information required for the design of the structure will be described.

Keywords: seismic soil structure interaction, performance based design, seismic design

1. INTRODUCTION OF THE STRUCTURE

The Transbay Transit Center (TTC) consists of three above grade levels and two basement levels. The structure is built in two phases. Phase 1, being designed now extends from gridline 1 at the west end to gridline 36 at the east end. These gridlines are typically 42.5 ft apart. The TTC superstructure will consist of three pods; west, center and east, separated by seismic joints on gridlines 10 and 20. The substructure is a continuous concrete box approximately 1500 ft long, 180 ft wide, and 50 ft deep. The bottom of the trainbox is a 5 ft mat foundation supporting the structure. Due to water pressure, there is net uplift acting at the bottom of the mat. The current design incorporates tie-downs to resist uplift forces.

The entrance of trains to the trainbox from the west side requires an approach throat that almost doubles the width of the section at the entrance. There are west and east end walls to close the perimeter of the trainbox. The east wall will be removed when Phase 2 of the structure is to be constructed.

The superstructure is a steel structure. In the transverse (North-South) direction, moment frames carry the lateral forces acting on the structure. In the longitudinal direction, the main lateral force resisting system is a combination of eccentrically braced frames (EBFs) and bucking restrained frames. The lateral forces in the transverse and longitudinal direction are ultimately transferred to the trainbox and then to the soil.

2. CHALLENGES OF SEISMIC DESIGN

The seismic design of the TTC involves analysis and design for a myriad of issues related to dimensions and geometry of the structure, the effect of long and deep basement, effect of body as well as surface waves, variations in soil profile through length and depth, effects of high water pressure, and presence of adjacent structures. Dealing with all these issues required consideration of soil-structure interaction effects. This paper focuses on the general methodology and the range of analysis techniques used to get information on the seismic behaviour of the structure and address the various issues unique to this structure. More details of the numeric analyses results will be published in separate papers.

The large dimensions of the basement of the TTC provide challenges regarding the definition of seismic ground motions. At a length of 1500 ft, the ground motion affecting one end of the TTC is different from the other end. Similarly, the soil profile and ground motion changes significantly through the depth of the trainbox and the design should account for this variation. It is simply not an option to choose the ground motion at a given level, whether mat or ground surface or an elevation in between and design for it. Capturing the effects of these variations requires performing soil-structure interaction analysis at various sections along the trainbox. These analyses provide a good description of the seismic demands at various elevations at any section, as well as the changing response through the length.

The trainbox has end walls that can affect the response of the trainbox and the type and intensity of the motion as experienced at the base of the superstructure. Without the end walls, the cross section of the trainbox is flexible and can rack laterally. The racking stiffness of a structure as compared to the stiffness of the soil that was replaced has a significant effect on the interaction effects (Wang, 1991). Structures that are more flexible in racking can amplify the lateral distortions as compared to what would be observed in free field. On the other hand, very stiff structures can significantly limit the amplitude of distortions compared to free field motions and in effect average the ground motion in depth. At the location of an end wall, the significant lateral stiffness of the end wall will prevent noticeable racking. On the other hand, far from the end walls, the racking flexibility of the trainbox can affect its deformations as well as the motion affecting the superstructure. The heavy weight of the superstructure can also result in large base shears that can potentially result in additional racking of the trainbox section.

On the east end, the temporary wall constructed as part of Phase 1 construction will be removed in phase 2, changing the seismic behaviour. A seismic joint is constructed between the east end wall and the rest of the trainbox. Therefore, the trainbox is free to rack at the east end during both phases of construction. The west end is permanent and its effects should be considered. These effects are incorporated in the analysis by preventing racking in sections close to the end wall. In addition, a 3D model of the trainbox was analyzed as will be discussed to evaluate some of the 3D geometry effects. The effect of the trainbox at the west end is also an increase in lateral racking stiffness and was captured through the same analysis as performed for the west end wall.

There was an extensive site analysis performed by AMEC Geomatrix (AMEC, 2010) to identify the soil profile throughout the site. The soil profile changes along the length as well as across the width of the structure. The soil is typically stiffer at the west end and becomes softer going east. About the last one third of the length of Phase 1 of TTC is located east of the old San Francisco Bay shoreline. The top layers of this soil are very soft fill material. An upper layer of the soil throughout the site is susceptible to liquefaction and is expected to liquefy in a strong earthquake. The variations in soil profiles are captured in the soil-structure interaction analysis of 2D structural sections, as discussed in detail in an accompanying paper (Fernandez et. al, 2012).

One last factor that is unique to this project is the incorporation of the effects of surface waves on the seismic response. The typical seismic site response analysis involves propagation of seismic shear waves upward from the bedrock and analyzing the structural response subject to such ground motion. Even when performing soil structure analysis with 2D or 3D models of the structure, the input ground motion is the bedrock ground motion as propagated upward through shear waves. The effect of surface waves is typically ignored and the literature discussing the effects of surface waves is scarce. The response of the structure to surface waves was a concern that needed to be considered. Given the length of the structure, the question was whether the surface waves have a considerable effect, and if so, whether there is a need for a seismic joint within the trainbox itself to minimize such effects. Next, the analysis procedures are described based on the types of seismic waves that are being considered. The discussion is focused on the forces acting on the TTC trainbox.

3. EFFECTS OF SHEAR WAVES ON THE TRAINBOX RESPONSE

The vertically propagating shear walls are typically the only component of the ground motion considered in seismic design. Seismic Site Response Analysis (SSRA) techniques rely mainly on propagating the horizontal component of the bedrock motion vertically all the way to the surface through a soil column. The analysis can be performed using nonlinear or equivalent linear models of soil to produce free-field soil motions and spectra and strain compatible soil properties all the way to the surface. To analyze the soil-structure interaction, two-dimensional or three-dimensional models can be used that include soil and the foundation and possibly the rest of the structure. This analysis can also be performed using either nonlinear or equivalent linear models. The analysis quantifies the interaction effects and helps determine the response of the section. For the TTC, a combination of 2D and 3D analysis models was used to ultimately estimate the effect of seismic shear wave on the response of the structure. Each analysis technique addresses certain issues, and has its own limitations.

3.1. Two-Dimensional Soil-Structure Interaction Analysis

The most important component of the SSI analysis of the TTC is a series of 2D SSI analyses, performed at various points along the structure. The purpose of performing the analysis at multiple sections is to capture the variations in the structural geometry, soil profile, and superstructure properties. Four different sections are analyzed, each capturing the response at a certain section of the TTC. The first section is namely at gridline 6 of the structure, has been modelled as rigid in racking and is meant to capture the response very close to the end wall. Two other sections, one at gridline 15 and one at gridline 21 are analyzed. These two sections are meant to represent typical sections in the structure. Finally, one section was analyzed at gridline 31. This section was analyzed with and without a representation of the adjacent structures. The soil profile at this section was also softer than that at the other analysis sections. The analysis at the 2D sections was typically performed two ways: direct approach including all the inertial effects of soil and the structure in one model, and substructuring approach, consisting of kinematic analysis (the stiffness and inertial effects of soil and the stiffness effects of the trainbox) followed by structural evaluation using the kinematic analysis results. For substructuring approach, soil stiffness, and damping values, and scattered motions were then provided so that a model of the structure could be analyzed using a substructuring approach, with the second model having a more refined model of the structure including nonlinear effects and accurate mass distribution. The details of modelling the soil, handling variations in soil properties, and the results from the analysis are presented in an accompanying paper, (Fernandez et. al, 2012).

3.2. Three- Dimensional Model of the Trainbox

The two dimensional models provide a great deal of information about the response of the structure and are a great tool for the design. For the superstructure, the results of the analysis provide estimates of the demands on the superstructure. Such information was used to design the superstructure for appropriate seismic demands. The superstructure comprises three different sections, separated from each other above ground through seismic joints. The analysis from the four sections provided adequate information for proper design of each portion of the superstructure.

The trainbox itself is subject to two different types of seismic demands. The lateral racking of the trainbox imposes bending and shear forces in its side walls. The trainbox section is similar to a moment frame moved laterally, subject to internal forces in the same direction. The bending of the side wall results in tensions and compression on the opposite faces of the same wall and produce stresses that are principally in the vertical direction. Similarly, the shear forces in the side walls tend to be in the direction parallel to the width of the wall and would require shear reinforcement in that direction, if any.

However, there are other forces acting on the trainbox as a result of the variations in the response along the length. The various sections along the length have different racking and total displacements. The racking is only part of the transverse displacement experienced at any section. The racing is very little close to the west end wall, whereas there is tangible racking in the middle of the box. Even at location far from the west end, the variations in trainbox geometry, soil profile, and superstructure properties mean that the racking profile is not uniform. The difference in total displacement is less though and global responses of the four sections are generally in phase, as seen in Figure 1. Looking at Figure 2, it is evident that any differential transverse displacement or racking also results in bending of the trainbox about a vertical axis. These forces stress the entire trainbox cross-section acting as one large beam section. The bending moments and shear forces associated with this mode of deformation act in a direction different from what was discussed previously. The effect of the bending is axial stresses in the longitudinal direction of the trainbox. These stresses act not only on the side walls, but also on the horizontal diaphragms at the ground and basement levels, including the mat foundation. The side wall and adjacent slabs on one side would experience compressive stresses whereas the side walls and slab on the other side are subject to tensile stresses. The shear forces act in the transverse direction across the width of the trainbox, subjecting the diaphragm at the three levels to shear forces.

To capture this set of forces, the deformations as obtained from the 2D SSI analysis models are imposed on a three dimensional model of the trainbox. Since the previous analysis only gave deformation histories at four sections, the displacements at various other points along the trainbox are determined by interpolation and extrapolation from these four points. The task of imposing deformation on the model is sensitive and should be handled properly. Imposed motions on a model determine the internal forces. Discontinuities in displacement that might seem minor could result in very large locale forces and stresses when analyzed. The interpolation of the motions should produce a smooth profile to avoid such erroneous results. In addition, the imposition of such forces might result in large local external forces acting on the model. In reality, soil is a flexible material with its own limits of strength and with internal deformations when subjected to large stresses. Therefore, the soil would deform and relieve some of the forces that would otherwise form. A more realistic model is one that allows for the soil to deform. For our analysis, we imposed the motions on the far end of the springs that were attached to the sides of the trainbox. These springs represented the dynamic strain-compatible stiffness of the soil, as obtained from 2D SSI analyses. The deformations within the springs allow for a more realistic distribution of the displacement that reflects the flexibility of the soil. The deformations in the model in the springs were relatively small compared to the total amplitude of displacements. Therefore, their deformations do not distort the deformed shape of the trainbox. In addition, it would be too computationally expensive to analyze a full nonlinear model of the entire trainbox and the superstructure with proper details to capture nonlinear effects under several sets of dynamic ground motions. Rather, the intent of the analysis was to estimate the internal forces in the trainbox as a result of the variations in ground motion and establish whether there is a need for a seismic joint in the trainbox. Those ground motions were in turn obtained using proper dynamic analysis accounting for dynamic effects and the nonlinear nature of the soil and structure response. As such, the model of the trainbox was subjected quasi-statically to the ground motion, just to estimate the internal forces as a result of external deformations. The external deformations were applied to the far end of the soil springs. The results of the analysis showed bending and shear demands for the trainbox to be well within its capacity.

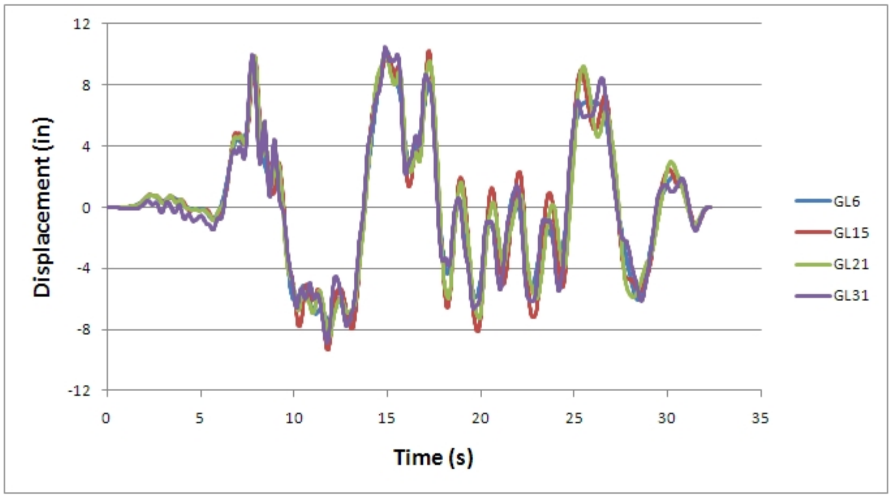

Figure 1- Displacement of Top of Trainbox under 161MX GSL2 Excitation at Different Sections

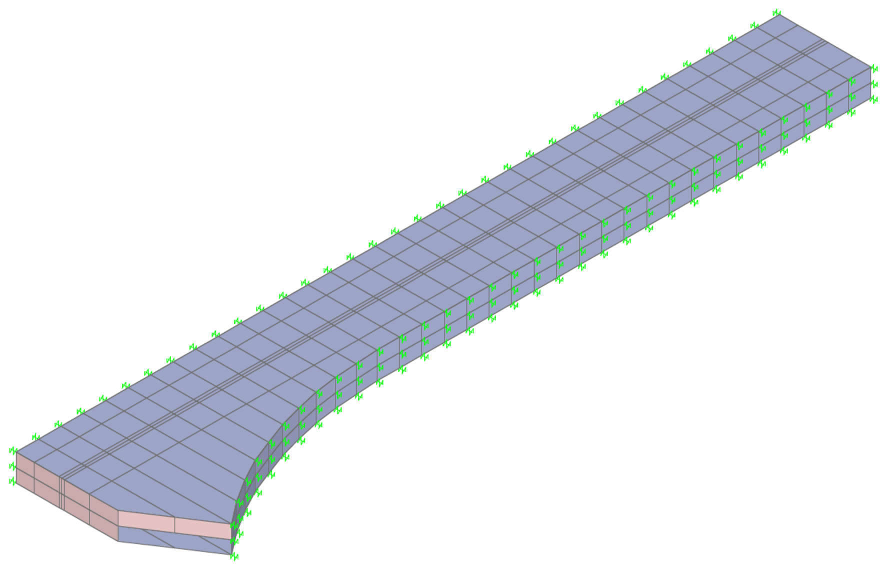

Figure 2. Model of the trainbox to study the effect of varying ground motions through its length and depth

4. EFFECTS OF SURFACE WAVES ON THE TRAINBOX RESPONSE

Decoupling of recorded ground motion into the contribution from different wave types is not a straightforward task. For sites that are far from a seismic source, there is a well-known separation in time between the moments of arrival of major P waves, S waves, and surface waves because of the difference in wave propagation speeds. However, for a site similar to TTC that is about 10 miles from the fault and is expected to experience a long duration of ground motion during strong earthquakes, the different wave types will all be simultaneously present in the record. Efforts to decouple the ground motion into various wave types require a closely spaced network of synchronized seismographs. There have been limited studies with this goal (Bolt et. al. 1982, Darragh, 1988).

To estimate the effect of surface waves on the trainbox, it was necessary to estimate the frequency content and the amplitudes associated with the surface waves. The surface waves are two types: Rayleigh waves and Love waves. Rayleigh waves result in elliptical motion of particles in a vertical plane in the direction of wave propagation. Love waves produce horizontal motions in the direction normal to the wave propagation. Each wave type is studied separately as it results in different structural actions. Figure 3 shows the different wave types present in seismic ground motion.

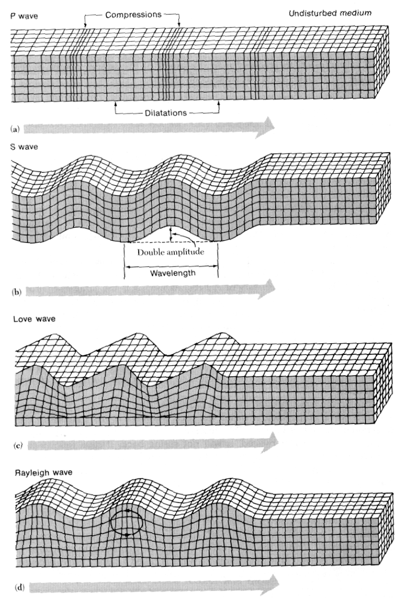

Figure 3. Different types of seismic waves (Bolt, 1993).

4.1. Estimation of Love Wave Effects

Love waves induce transverse motion similar to horizontally propagating shear waves in soil particles, but are limited to the surface layer of soil on a stiffer half-space. Simple closed-form solutions for Love waves are available (e.g., Udias, 1999). An important property of Love waves is that their speed is frequency-dependent. The lower the frequency of the Love wave, the higher its wave speed. The wave speed varies between the shear wave velocities of the half-space and the upper layer. Equation 4.1 shows the relationship between the shear elastic properties of the media, the frequency of Love wave, and its propagation speed. In this equation, G and G′, and β and β′ are the shear modulus and shear wave velocities in the half-space and surface layer, respectively. The variable *c* is the velocity of the Love wave of natural frequency ω propagating through an upper layer of thickness H. The wavelength λ is related to the wave period and wave speed by λ = c T.

Love Wave Speed Equation:

$$\frac{G\sqrt{1/c^2-1/\beta^2}}{G'\sqrt{1/\beta'^2-1/c^2}} = \tan\left(\omega H\sqrt{1/\beta'^2 - 1/c^2}\right) \tag{4.1}$$

The density for the bedrock and surface layers are 148 and 120 pcf respectively. For the half-space, bedrock shear wave velocity is established at 4265 ft/s (AMEC, 2010). For the upper layer, the shear wave velocity varies significantly with depth and along the length of the trainbox. Importantly, the shear wave velocity is also highly dependent on the level of shear strain and varies considerably during an earthquake. A range of shear wave velocities, 400 to 700 ft/s, is considered for the upper layer based on local section profiles. A representative strain-compatible shear wave velocity at the site is 400 ft/s. The depth to bedrock varies between 150 ft (45 m) and 250 ft (75 m) in the vicinity of TTC.

The relationship between Love wave period and its wavelength (based on the calculated wave speed) can be plotted using Equation 4.1 for different sets of upper layer shear wave velocity and thickness (depth to bedrock). To address the variations in the parameters affecting the Love wave characteristics, two shear wave velocities of 400 ft/s and 700 ft/s were used. The depth to bedrock was varied for the analysis between 50 ft (15 m) and 250 ft (75 m). The 50 ft represents values observed not at the site, but within a proximity of about 1000 ft. The Love wave period and wavelength corresponding to these velocity and thickness variations is shown in Figure 4. As such, waves of a given period could manifest in different wavelengths and waves of a given wavelength could have a wide range of periods. Therefore, the Love waves that would be measured at the TTC site would be substantially incoherent and their energy would be spread across a wide range of frequencies and wavelengths.

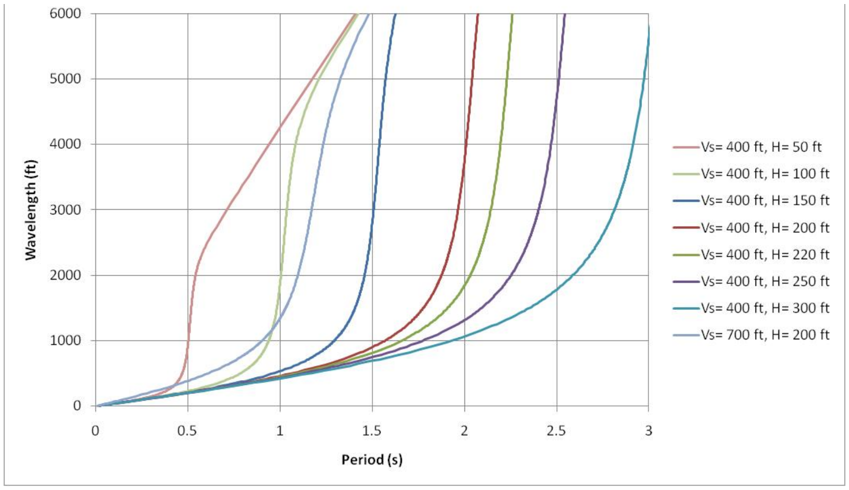

Figure 4- Wavelength of Love waves

Based on a simple analysis of the model subject to sinusoidal deformations of various wavelength, it was shown that for a fixed amplitude of motion, the maximum shear and moment imposed on the trainbox structure came from waves with a wavelength of about 1000 ft. Based on Figure 4, these waves should have periods less than 2s for the realistic range of values of layer thickness (H) and shear wave velocity at the site.

To estimate the amplitude of the motion in this period range, a set of ground motions records from the 1989 Loma Prieta earthquake at sites with similar soil condition to the TTC were studied. For each recorded motion, the displacement history was filtered in the frequency domain to compute the peak ground displacement from waves with periods of less than or equal to 2 seconds. Figure 5 shows a sample displacement records together with the corresponding contribution from periods of 2 seconds and less. The average ratio of the two-second displacement to the peak displacement for these motions was 0.3. The average displacement in the transverse direction from the SSI analysis for the design earthquake was about 10 inches. Therefore, the motion for periods less than 2 second based on all types of waves was estimated at 3 inches. Within that period range, the contribution of surface waves is likely less than 10-20% of the total displacement (Abrahamson, 2011). Conservatively assuming that a 0.2*3=0.6 amplitude Love wave at the critical wavelength of 1000 ft was acting on the trainbox, the resulting internal moments and shear forces acting on the trainbox were found to be roughly 10% or less of the capacity. Therefore, the trainbox was considered strong enough to sustain the effect of Love waves with no damage and without the need for a seismic joint in the trainbox.

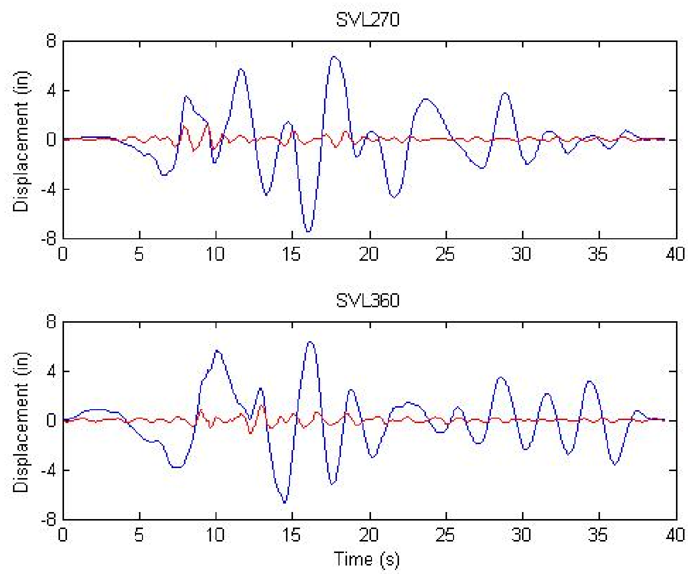

Figure 5- Displacement records at Sunnyvale station and the corresponding component for periods less than 2 second

4.2. Estimation of Rayleigh Wave Effects

Rayleigh waves induce elliptical motions in soil particles, as seen in Figure 3. , with a vertical component and a horizontal component that is in the direction of wave propagation. The vertical component of the motion is prevented at the TTC by tie-downs. For waves propagating parallel to the trainbox, the horizontal motion is parallel to the trainbox, which cannot cause any transverse and racking motion in the trainbox. Rayleigh waves propagating normal to the box will reach the box at the same time over its length and will tend to move the trainbox evenly over its length. For the waves arriving from random angles, the very high stiffness of the box in its longitudinal direction will prevent horizontal motions in that direction and the inclined angle of attack will result in higher apparent wave speed and apparent wavelength, reducing the effect of the imposed motion. Furthermore, the amplitude of all the surface waves including the Rayleigh waves is limited, as shown in the previous section. The combination of the effect of the tie-downs and the large longitudinal stiffness of the trainbox disrupts the necessary vertical and horizontal motions for the Rayleigh waves, hence minimizing their effect on the trainbox.

5. CONCLUSIONS

This paper presented the general framework of the analysis for the seismic soil-structure interaction effects at the Transbay Transit Center. Different analysis techniques and model were used for this task. Each model can address certain features of the seismic response more properly. For the TTC, several 2D SSI analyses were performed to analyze the effect of the variations in soil profile and geometry and other factors such as the presence of adjacent structures. These results were then imposed on a 3D model of the trainbox to have an estimate of the effects of 3D geometry changes acting on the trainbox. Then, in an analysis unique to this project, closed form equations of surface waves were analyzed using parameters from the soil at the site to obtain the relationship between the wave periods and wavelength. The critical wavelength affecting the trainbox was separately identified using a simple analytical model. The data form past earthquakes was then processed to determine the relative contribution of the waves from certain wavelengths. Finally, the result of previous research was used to estimate the relative contribution of the surface waves to the seismic ground motions in a given period range. All these pieces of information were combined to establish that the effect of surface waves on the trainbox is within acceptable limits. While the discussion of the various analyses in this paper was kept to a minimum, it is expected for a structure of this level of complexity to use a combination of analysis models to get a full picture of the parameters of the structural response in order to design the structure properly.

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